METHODOLOGY FOR THE OPERATIONAL PERFORMANCE ASSESSMENT OF TWO-LANE HIGHWAY FACILITIES

By

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Traffic engineers have indicated that a facility-based evaluation methodology for two-lane highways would be much more useful and practical to them than just the individual segment and point analysis methodologies. A facility level analysis will allow the various features (e.g., isolated intersections, continuous grades, passing lanes) that are typical to an extended length of two-lane highway to be addressed in a combined analysis with a single performance measure and level of service value resulting.

Currently there is not any operational analysis methodology to address two-lane highways with different segment types at the facility level. The scope of analysis provided in the *Highway Capacity Manual* (HCM) *2000* for two-lane highway is limited to separated segment within the facility.

In this dissertation, a methodology is developed to assess the operational performance of an extended length of a two-lane highway facility, which might include an occasional signalized intersection and other control or roadway segments.

The methodology maintains some fidelity to the HCM 2000 by using the existing methodologies for the two-lane highway segments and signalized intersections. Meanwhile, the individual segment in the whole facility is not regarded as an isolated object, and the impacts between segments are taken into account. A common time/delay based measure (percent time-delayed) is used for each segment and point of the entire facility.

The primary contribution of this dissertation is the development of analytical relationships to determine the upstream and downstream boundary points that define the transition between basis tow-lane highway segments and the effective influence area of the signalized intersection and the TWSC intersection, as well as the corresponding method for calculation of an overall facility service measure based upon percent time-delayed and then translating it to a Level Of Service (LOS) grade.

The dissertation focuses the efforts on developing the methodology for operational analysis of a two-lane highway with an isolated signalized intersection at the facility level. Nonetheless, this research also provides a model for the basic structure of a facility-level analysis that will be amenable to the incorporation of a variety of segment types. Finally the methodology has been incorporated into the existing HIGHPLAN software program.

CHAPTER 1
INTRODUCTION

1.1 Background

Two-lane highways, which account for approximately 80 percent of all paved rural highways in the United States and carry about 30 percent of all traffic, are important facilities in our transportation network system [1]. With the increased development in rural areas, more signals are being installed on two-lane highways typically when these highways travel through small towns. Additionally, there are a number of other design and operational treatments developed on extended lengths of two-lane highways, such as passing lanes, two-way stop-controlled intersections, driveway turnouts, and two-way left-turn lanes. They can be effective in alleviating some operational problems on two-lane highways.

Because these design and operational treatments significantly affect traffic operations on two-lane highways, there is ongoing demand for analysis methodologies with which to analyze the operating effectiveness of the entire length of two-lane highway, that is, the facility as a whole. This is consistent with the fact that drivers typically evaluate the quality of their trip over its entire length, not just in separate segments.

1.2 Problem Statement

Personnel with the Florida Department of Transportation (FDOT) Systems Planning Office have indicated that a facility-based evaluation methodology for two-lane highways would be much more useful to them than just the individual segment and point

analysis methodologies. Segment is a stretch of roadway with homogenous conditions. Facility is composed of different segments. A facility level analysis will allow the various features (e.g., isolated intersections, continuous grades, passing lanes) that are typical to an extended length of two-lane highway to be addressed in a combined analysis with a single performance measure and level of service value resulting.

Frequently, a traveler is less concerned about the quality of service offered by a particular segment than the service over a facility that may be served by more than one segment type. For example, on a two-lane highway with several isolated intersections, most travelers are concerned about the operation of the whole facility and not just the operation of a particular intersection, or a particular two-lane highway segment.

From the viewpoint of travelers or transportation engineers, a facility level analysis on a two-lane highway facility, instead of the segment level, is more practical and meaningful. Currently, there is not any operational analysis methodology to address two-lane highways with different segment types at the facility level.

In the *Highway Capacity Manual* (HCM) 2000 [1], the basic two-lane highways with or without passing lane can be evaluated with the methodology in Chapter 20, "Two-Lane Highways". Isolated signalized intersections on two-lane highways can be evaluated with the methodology in Chapter 16, "Signalized Intersections". The scope of analysis provided in the HCM 2000 for two-lane highways is mainly limited to separate segments within the facility, while the methodology to evaluate the facility as a whole is of much more practical value to transportation engineers.

In the HCM 2000, Chapter 15, "Urban Streets", presents the methodology for evaluating arterials in urban and suburban areas with multiple signalized intersections at a

spacing of 2.0 miles or fewer. To some degree, the analysis procedure is performed at the facility level, which combines the segment running time and control delay at the signalized intersection when determining the performance measure (average travel speed) for the entire facility. However, this methodology has some obvious drawbacks. They are the following:

1. The potential impacts between roadway segments and signalized intersections are not taken into account in this methodology. Continuing research has shown that the installation of signalized intersections can significantly affect traffic operations on the two-lane highways, such as decreasing average travel speed, and increasing percent time-spent-following. The impact between different segment types is a big issue differentiating the facility-level analysis from the segment-level analysis.

2. In this methodology, the urban street is divided into multiple segments, which is the full distance from one signalized intersection to the next. The signalized intersection is regarded as a typical point location within a traffic network, and control delay is regarded as a typical point performance measure without covering any distance. In the HCM 2000 [1], by definition, control delay includes movements at slower speeds and stops on intersection approaches as vehicles move up in queue position or slow down upstream of an intersection, as well as delay due to re-acceleration downstream of a signal after stopping or slowing. It implies that control delay happens not at a point, but actually within a certain distance. Although the time lost due to slow movement before and after a stop is technically part of the running time, it is also included in control delay. Segment division in this methodology causes the problem of double-counting the

deceleration-acceleration delay. Segment division introduces error due to the segment between intersections being longer than they should.

In this dissertation, a methodology for the operational performance assessment of two-lane highways with isolated signalized intersections (spacing of signalized intersections is 3 miles or more) will be explored, in addition to a way to combine a number of different segments (two-way stop-controlled intersection, passing lanes, basic segment, etc.).

1.3 Objectives and Tasks

The objective of this research is to develop a methodology that can be used to assess the operational performance of an extended length of a two-lane highway facility, which might include occasional signalized intersections, unsignalized intersection, and other control or roadway treatments. This two-lane highway would then be comprised of multiple segments, with segment delineations occurring with a change in either roadway or control attribute. The most common types of two-lane highway segments are the following:

- Basic segment—this is a segment that consists of a simple two-lane cross section, either level or rolling terrain
- Basic segments with a continuous specific up or down grade
- Three-lane cross section segments, with the additional lane being a passing lane
- Three-lane cross section segments, with the additional lane being a center left-turn lane
- Three-lane cross section segments, with the additional lane being a right-turn only lane
- Segments terminating with an isolated signalized intersection
- Segments terminating with an unsignalized intersection

- Segments terminating into a multilane highway

This research focuses on developing the methodology for operational analysis of a two-lane highway with isolated signalized/un-signalized intersections at the facility level. Nonetheless, it is intended that this research will also provide a model for the basic structure of a facility level analysis that will be amenable to the incorporation of a variety of segment types. The tasks required to accomplish the research study objectives are as follows.

Task 1: Perform a literature review on current analytical and simulation methods for evaluating the performance of basic two-lane segments and signalized intersections, along with previous research on the effects of signalized intersections on a two-lane highway.

Task 2: Define the basic conceptual framework, from a segmentation and service measure perspective, for combining two-lane highway segments with intersections into a facility-wide operational analysis.

Task 3: Investigate traffic operations at the boundary of a two-lane segment and a signalized intersection (upstream of the signal) and develop a method that can be used to determine the effective length of the signal's influence area upstream of the signal.

The conceptual approach being taken is that the effective upstream length of the signal's influence area is a function of average queue length on the approach to the signal and some portion of perception/reaction time and braking distance before the queue. The combination of the stopping sight distance (SSD) equation, from the AASHTO "Green Book" [2], and average queue length formulas from Chapter 16 of the HCM 2000, are compared to simulation output from CORSIM. A comprehensive experimental design

for simulation is utilized to fully explore the relationship between influence area and the appropriate traffic and control variables. These results are reconciled against those from the SSD + Average Queue Length results to arrive at an appropriate relationship.

Task 4: Investigate traffic operations downstream of a signalized intersection and develop a method that can be used to determine the effective length of the signal's influence area downstream of the signal.

The conceptual approach being taken is that the effective downstream length of the signal's influence area is a function of vehicle acceleration and platoon dispersion. Thus, three types of areas are investigated:

- Simple vehicle dynamics equations related to vehicle acceleration.
- Platoon dispersion downstream of a signal, such as the model currently used in the TRANSYT-7F program.
- Changes in the vehicle headway distribution downstream of a signal. Previous work performed by Dixon et al. [3] related to this issue is investigated, with two main differences being:
    - A composite headway distribution model is used for headways instead of a simple negative exponential model, and
    - The EPF (Entering Percent Following) measure as used by Dixon et al. is related to downstream vehicle speeds as opposed to percent time-spent-following.

Task 5: Investigate traffic operations at the boundary of a two-lane segment and a two-way stop-controlled intersection (without sharing left-turn lanes), and develop a method that can be used to determine the effective length of the two-way stop-controlled intersection on through traffic on the major street and right-turn traffic from the major street behind it.

Task 6: Join the components (basic two-lane segment, upstream signal influence area, signal delay, and downstream signal influence area) into an integrated methodology

for the operational analysis of a two-lane highway facility. This methodology will be predicated upon the use of an aggregated percent-time-delayed measure as the facility-wide service measure. As part of this, two example problems are presented.

Task 7: Determine LOS thresholds that maintain a reasonable relationship with existing LOS thresholds in Chapter 20 (two-lane highways) of the HCM 2000. For example, with the use of a new service measure for the facility analysis, it should not be possible to get a better level of service for the two-lane highway when installing a signal compared to the previous LOS method.

Task 8: Provide a qualitative overview of how the method of analysis for multilane highways might be modified to fit into this framework such that combinations of two-lane highway segments, multilane highway segments, and occasional traffic signals can be analyzed as an overall facility.

Task 9: Modify HIGHPLAN program to include this new methodology for a facility-level analysis. This will require the addition of a new screen for segment and intersection data inputs, as well as a new result.

1.4 Organization of the Document

This document is organized into six chapters in addition to the Introduction. Each of the chapters covers a different aspect of the issue of assessing the operational performance of a two-lane highway facility consisting of different segment type. In Chapter 1, the background, objectives, and tasks of this research are introduced and the stage is set for the organization of the rest of the document. Chapter 2, the literature review gives a review of the current methodologies for evaluating the operational performance of basic two-lane segments, and signalized intersections. It also provides a brief overview of the simulation models and their potential ability to contribute to the

performance evaluation for the two-lane highway facility. Chapter 3 presents the conceptual framework of the operational analysis procedure for a two-lane highway facility, and puts forward the methodology of operational analysis for the two-lane highway facilities. The first two steps of the selection of facility-wide service measure and facility segmentation are also explored in this chapter. Chapter 4 describes the procedures for determining component segment lengths of the facility, mainly including the length of a signalized intersection influence area, and the length of the downstream segment affected by the upstream signal. Chapter 5 describes the methods for calculating the service measure value of each segment type. They include the free-flow speed, and average travel speed on three kinds of two-lane highway segments, and control delay at the signalized and unsignalized intersections. In Chapter 6, two examples are provided to illustrate the application of the developed methodology. Finally, Chapter 7 provides summary and recommendations for future research.

CHAPTER 2
OVERVIEW OF ANALYSIS METHODS

This chapter summarizes current methodologies for evaluating the operational performance of basic two-lane segments, and signalized intersections. It also provides a brief overview of the TWOPAS, TRARR, and CORSIM simulation models and their potential ability to contribute to the performance evaluation for the two-lane highway facility. Finally previous research on effects of signalized intersections on a two-lane highway segment is presented.

2.1 Analytical Methods

The following sections give an overview of analytical methodologies presented in the HCM 2000. They are for two-lane highways, signalized intersections, and urban streets. Finally the adjustment method used in the FDOT HIGHPLAN software for the facility level analysis is presented.

2.1.1 HCM Methodology for Two-Lane Highways

The *Highway Capacity Manual* (HCM) published by the Transportation Research Board (TRB) presents the widely accepted standards for analysis of two-lane highway capacity and quality of service.

In the 1950 HCM, the first version of HCM, the procedure for analysis of two-lane highway capacity developed by O. K. Norman was presented. The capacity of a two-lane road was determined by comparing the demand for passing with observed actual passing

rates at various flow rates. In the subsequent editions of 1965, 1985, and 2000 [1], the capacity and quality-of-service analysis procedure of a two-lane highway and their related service measures were revised. In the 1965 HCM, the capacity of a two-lane highway was estimated for both directions of travel combined, regardless of the direction split of traffic, and the two service measures for the operational analysis were the operating speed of traffic over a roadway section and the volume-to-capacity ratio. A great improvement in analysis of two-lane highway capacity and quality of service was achieved in the 1985 HCM. The capacity of a two-lane highway was determined to be a function of the directional split of traffic, ranging from a capacity of 2800 pc/h in both directions of travel combined for a 50/50 directional split to 2,000 pc/h for a 100/0 split. In this version, a new level of service measure named "percent time delay" was developed. Percent time delay is measured as the percentage of vehicles traveling at headway of 5 sec or less at one or more representative points within the analyzed roadway.

In Chapter 20 of the HCM 2000, an improved operational analysis procedure for two-lane highway was presented. Key features of the improved operational analysis procedure are revised factors for the effects of grades and heavy vehicles, separate computational procedures for two-lane and directional segments, provision of operational analysis procedures for passing lanes in level and rolling terrain, climbing lanes on steep upgrades, and steep downgrades on which some trucks must use crawl speeds [1]. The combination of average travel speed and percent time-spent-following was determined as the level of service measure (i.e., the performance measures used to base level of service upon).

The above discussion reviews the historical development of the HCM procedure for analysis of two-lane highway capacity and quality of service. The analysis procedure has been limited to the segment level. The operational analysis methodologies in this chapter do not address two-lane highways with signalized intersections or with other types of segments.

2.1.2 HCM Methodology for Signalized and Unsignalized Intersections

In the HCM 2000, Chapter 16 "*Signalized Intersections*" contains a methodology for analyzing the capacity and level of service of signalized intersections [1]. The methodology addresses the capacity, LOS, and other performance measures for lane groups and intersection approaches and the LOS for the intersection as a whole. The ratio of demand flow rate to capacity is used as a capacity utilization measurement. The capacity analysis methodology for signalized intersections is based on known or projected signalization plans, and traffic characteristics. The control delay per vehicle is used as the service measure. In this methodology, the signalized intersection is regarded as an isolated point location. It does not take into account the potential impact of downstream congestion on intersection operation.

In the HCM 2000, Chapter 17 "*Unsignalized Intersections*" contains a methodology for analyzing the capacity and level of service, lane requirements, and effects of traffic and design features of unsignalized intersections [1]. The analyzed unsignalized intersections include two-way stop-controlled (TWSC), all-way stop-controlled (AWSC) intersections, and roundabouts. For a TWSC intersection, LOS is determined by the computed or measured control delay and is defined for each minor movement. For an AWSC intersection, control delay is also used to determine LOS.

2.1.3 HCM Methodology for Urban Streets

In the HCM 2000, Chapter 15 "*Urban Streets*" contains a methodology used to access the mobility function of the urban street [1]. Four urban street classes are defined and reflect unique combinations of street function and street design. The degree of mobility provided is assessed in terms of average travel speed for the through-traffic stream. Computing the urban street or section speed requires the total time that a vehicle spends on the urban street. The total time consists of the segment running time and the intersection control delay of the lane group for through traffic.

The methodology may be used to analyze urban streets that have a traffic signal spacing of 2 miles or less. To some degree, the analysis procedure is performed at the facility level, which combines the segment running time and control delay at the signalized intersection when determining the performance measure (average travel speed) for the entire facility. However, the potential impacts between roadway segments and signalized intersections are not taken into account in this methodology.

2.1.4 Performance Measure Definition

Each facility type has a defined method for assessing capacity and level of service using performance measures. These measures reflect the operating conditions of a facility, given a set of roadway, traffic, and control conditions. Service measures are the performance measures used to base level of service upon. In this dissertation, a methodology of operational performance analysis is developed for the two-lane highway facility composed of different types of segments. A special service measure will be selected for this facility to determine its level of service.

The definitions of some performance measures often used in the HCM are summarized here.

- Volume to Capacity Ratio (v/c)

The v/c ratio is defined as the volume to capacity ratio, often used as a measure of the sufficiency of existing or proposed capacity.

- Average Travel Speed (ATS)

ATS is defined as the length of the roadway segment under consideration divided by the average total travel time for all vehicles to traverse that segment during some designed time interval.

- Percent Time-Spent-Following (PTSF)

PTSF is defined as the average percentage of travel time that vehicles on a given roadway segment must travel in platoons behind slower vehicles due to the inability to pass during some designed time interval.

- Travel Time

Travel time is defined as the time spent traversing a section of highway.

- Percent Free Flow Speed (PFFS)

Percent free flow speed is defined as the ratio of vehicle average travel speed to free flow speed.

- Density

Density is the number of vehicles occupying a given length of highway or lane and is generally expressed as vehicles per mile per lane.

- Control Delay

Control delay includes "Movements at slower speed and stops on intersection approaches as vehicles move up in queue position or slow down upstream of an intersection" [1]. It includes deceleration delay, stopped delay, and acceleration delay.

- Percent Time-Delayed

Percent time-delayed is defined as the percentage of the travel time that vehicles on a given roadway segment must travel at speeds less than their desired speed (typically the free-flow speed) due to inability to pass or to delays caused by traffic control.

2.1.5 FDOT HIGHPLAN Software

HIGHPLAN, designed for uninterrupted flow highway level of service analysis for planning applications, is FDOT's software for two-lane and multilane uninterrupted flow

highways [4]. HIGHPLAN maintains fidelity to the HCM 2000 two-lane and multilane procedures to the extent possible. However due to some unique characteristics in the State of Florida, HIGHPLAN incorporates a number of concepts and calculations that differ significantly from the basic procedures in the HCM 2000.

HIGHPLAN includes an adjustment to account for whether the analysis is at the segment level or the facility level. If a segment level analysis is performed, it is assumed that the highway section under consideration is short enough that it does not include any capacity reducing effects due to the presence of intersecting driveways or cross streets. If a facility level analysis is chosen, a 10% reduction is applied to the base capacity to account for driveway and cross street friction. This value is consistent with the capacity reducing effects of interchanges experienced on Florida freeways [5]. Nonetheless, this is a gross adjustment necessitated by the lack of a specific facility-level methodology.

2.2 Simulation Methods

The following section provides a brief overview of the TWOPAS, TRARR and CORSIM simulation models and their potential ability to contribute to the performance evaluation for the two-lane highway operations.

2.2.1 TWOPAS Software

TWOPAS (TWO-lane PASsing) rural highway simulation software is used for modeling traffic conditions on two-lane two-way roadways. This software was used extensively in developing the two-lane analysis methodology in the HCM 2000.

TWOPAS was first developed in the 1970s by Mid-West Research Institute for the US Federal Highway Administration (FHWA). TWOPAS was revised most recently in 1998, and was contained in a graphic interface, UCBRURAL, developed by the University of California-Berkeley. UCBRURAL provides a menu-driven interactive

graphical interface with comprehensive input checking, carefully selected default values, and user-selected output options including graphic depictions of traffic performance, which is more convenient for users to run TWOPAS model.

Figure 2-1 shows a view of the UCBRURAL road editor. Recently the TWOPAS traffic simulation model is built in the Traffic Analysis Module (TAM) of the Interactive Highway Safety Design Model (IHSDM) to estimate traffic quality-of-service measures for an existing or proposed design. The TAM facilitates use of TWOPAS by feeding it the roadway geometry data stored by IHSDM. Figure 2-2 shows a view of TAM input interface in IHSDM.

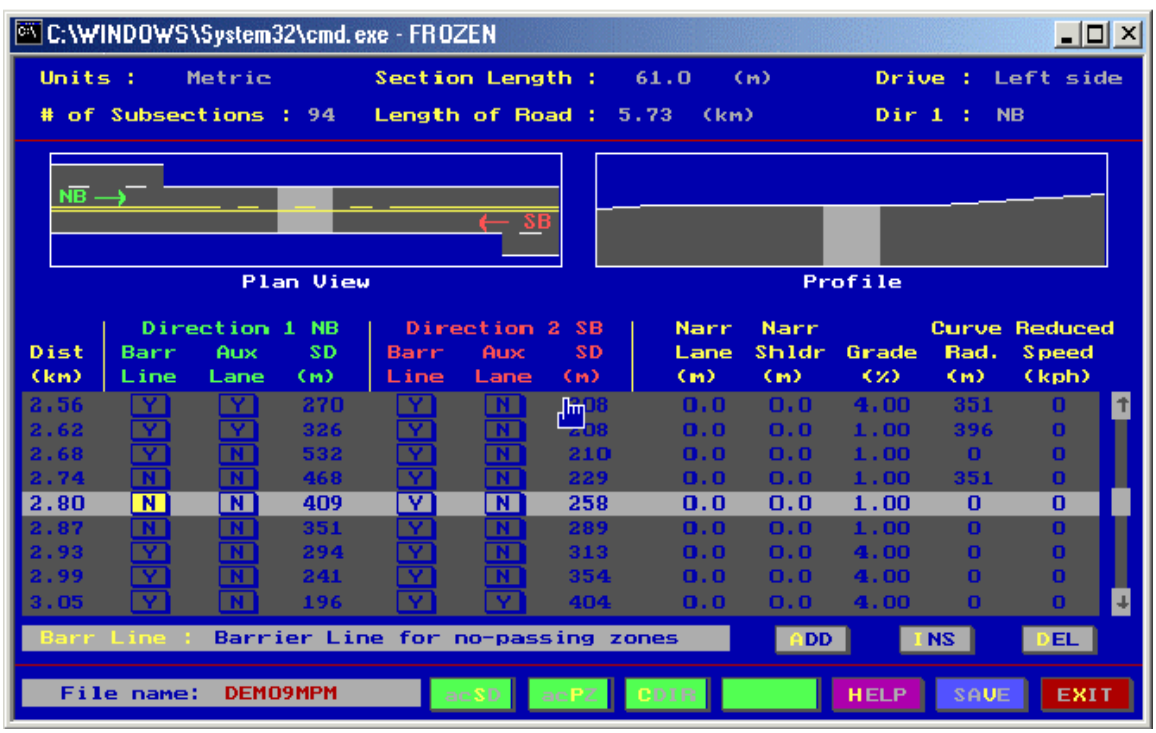

Figure 2-1. Screenshot from TWOPAS road editor

As a microscopic, stochastically based model, TWOPAS simulates traffic operations on a two-lane highway by reviewing the position, speed, and acceleration of each individual vehicle along the roadway at 1-second intervals. The operation of each

vehicle is also influenced by the characteristics of the vehicle and its driver, by the geometric of the roadway, and by the surrounding traffic simulation in a realistic manner as it advances along the road [6]. TWOPAS incorporates the major features:

- Highway Geometry specified in terms of grades, horizontal curves, lane and shoulder width, along with passing and climbing lanes.
- Traffic control specified by users, especially passing and no-passing zones, and reduced speed zones.
- Driver Characteristics and preferences including desired speeds, preferred acceleration levels, limitations on sustained use of maximum power, passing and pass-abort decisions, and realistic behavior in passing and climbing lanes.
- Entering Traffic streams generated in response to user-specified flow rate, vehicle mix, immediate upstream alignment, and the percent of traffic platooned.
- Driver speed choices in unimpeded traffic based on user-specified distribution of desired speeds; in the impeded traffic based on a car-following model that simulates driver preferences for following distances, relative leader/follower speeds, and desire to pass the leader [6].

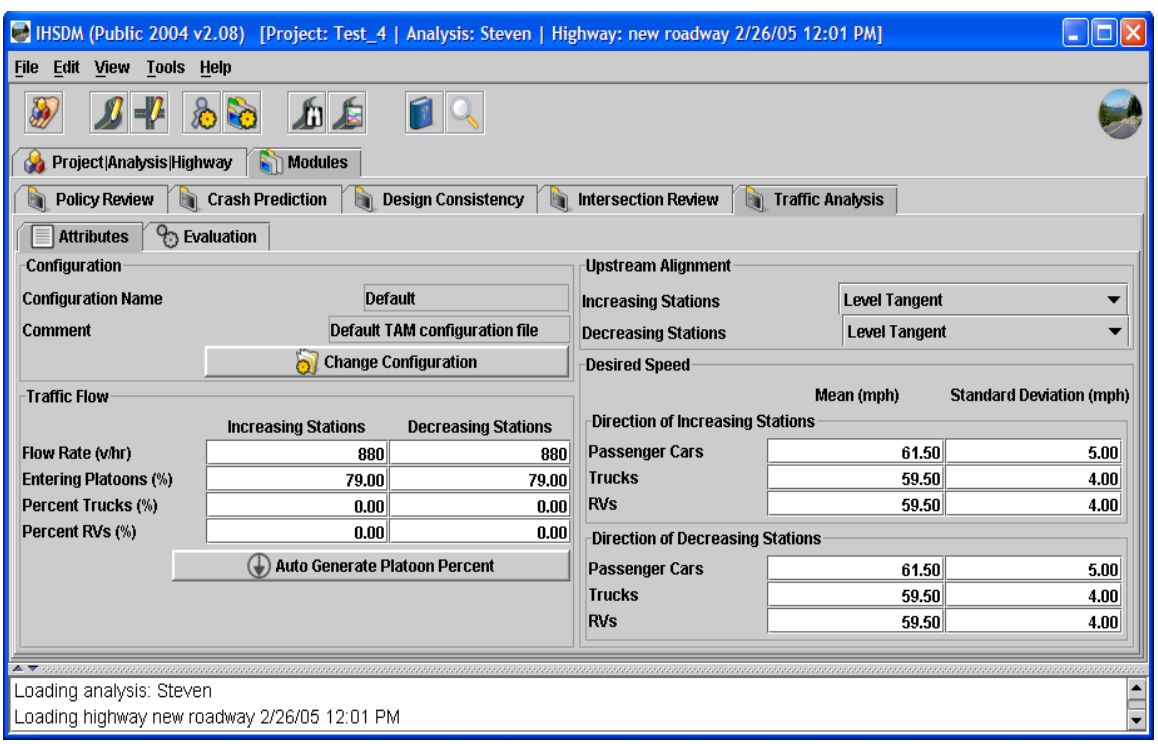

Figure 2-2. Screenshot from Traffic Analysis Module of IHSDM

TWOPAS has the capability to simulate both conventional two-lane highways and two-lane highways with added passing lanes. However, TWOPAS does not have the ability to simulate traffic turning on or off the highway at driveways, unsignalized intersections, or signalized intersections.

2.2.2 TRARR Software

TRARR (TRAffic on Rural Roads) was developed in the 1970s and 1980s by the Australian Road Research Board. TRARR is designed for two-lane rural highways, with occasional passing lane sections. It is a microscopic simulation model, that is, it models each vehicle individually. Each vehicle is randomly generated, placed at one end of the road and monitored as it travels to the other end. Various driver behaviors and vehicle performance factors determine how the vehicle reacts to changes in alignment and other traffic. TRARR uses traffic flow, vehicle performance, and highway alignment data to establish, in detail, the speeds of vehicles along rural roads. This determines the driver demand for passing and whether or not passing maneuvers may be executed [7]. Figure 2-3 shows an interface of the TRARR road editor.

TRARR is designed for two-lane rural highways, with occasional passing lane sections. TRARR can be used to obtain a more precise calculation of travel time, frustration (via time spent following), and benefits resulting from passing lanes or road realignments. For strategic assessment of road links, TRARR can also be used to evaluate the relative benefits of passing lanes at various spacing.

Similar to TWOPAS, TRARR has no ability to handle varying traffic flows down the highway, particularly due to major side roads or signalized intersections. However, TWOPAS was developed with U.S. data and, therefore, was better representative of U.S. conditions than TRARR.

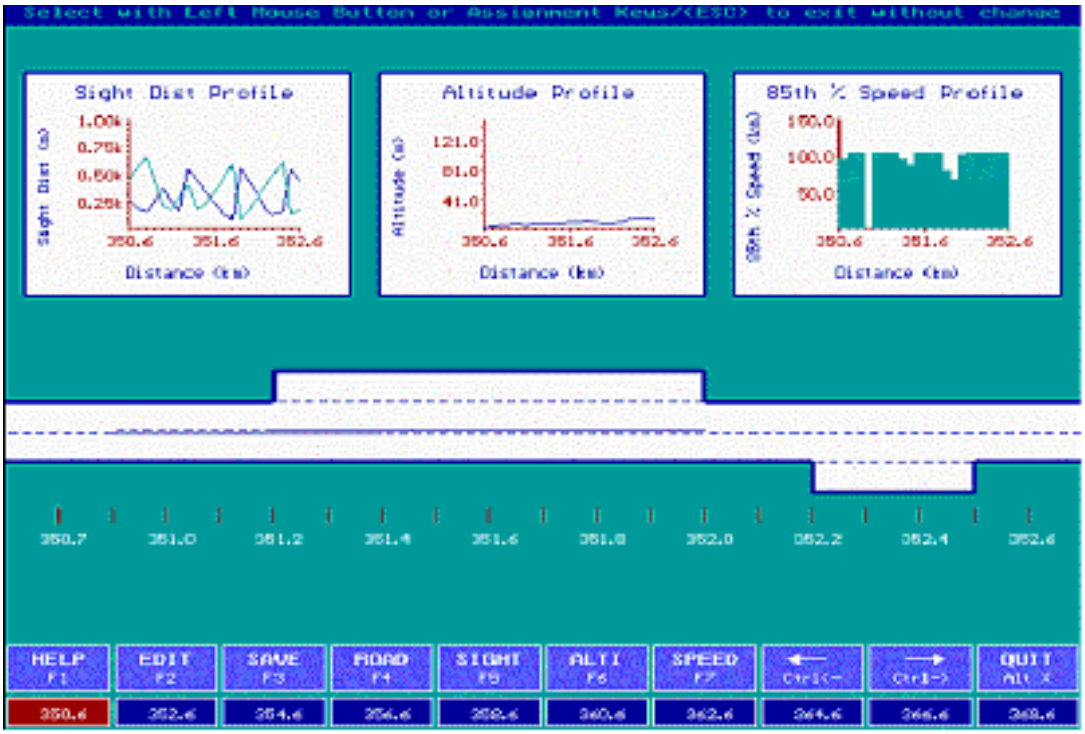

Figure 2-3. Screenshot from TRARR road editor

2.2.3 CORSIM Software

CORSIM (CORridor SIMulation), developed by the Federal Highway Administration, is the core simulation engine in the TSIS (Traffic Software Integrated System) suite [8]. CORSIM is a comprehensive traffic simulation program, applicable to surface streets, freeways, and integrated networks with a complete selection of control devices, such as stop/yield signs, traffic signals, and ramp metering. CORSIM is a microscopic, discrete time, stochastic, “state-of-the-practice” model used to simulate traffic operations. It integrates two microscopic traffic simulation models: the arterial network model, NETSIM, and the freeway model, FRESIM. CORSIM is able to simulate existing or proposed conditions on very large networks. CORSIM has been applied by thousands of practitioners and researchers worldwide over the past 30 years and

embodies a wealth of experience and maturity [8]. Figure 2-4 shows an interface simulating traffic operations at a signalized intersection.

CORSIM has expanded the capabilities of NETSIM and FRESIM with the following major enhancements:

- HOV lanes in FRESIM
- Freeway ramp metering
- Vehicle-type-specific turn percentages
- Support Larger Networks
- Path Following Capacity

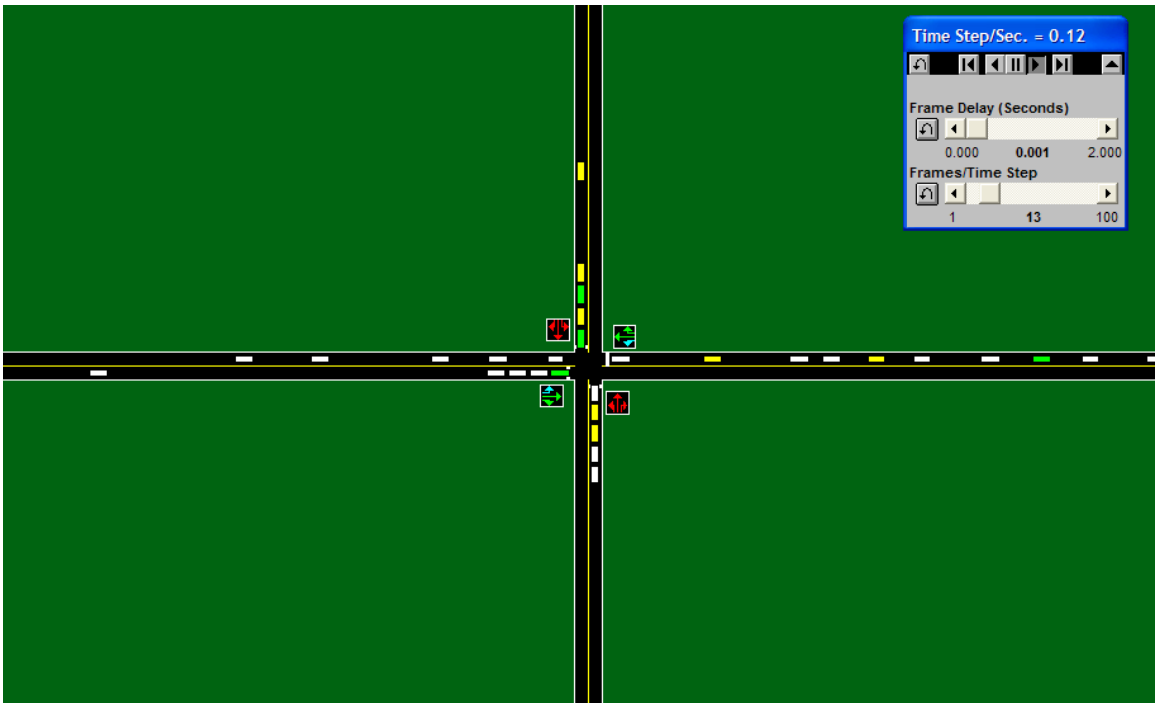

Figure 2-4. Screenshot from CORSIM simulation animation (TRAFVU)

CORSIM can simulate traffic and traffic control system using commonly accepted vehicles and driver behavior models. However, it does not have the ability to simulate vehicle passing operations on a two-lane highway using the opposing lane.

2.3 Effect of Upstream Signal on Two-Lane Highway PTSF

In the research of Dixon et al. [3], they developed a methodology to estimate the effects of a simple isolated signalized intersection on a downstream two-lane highway segment in terms of percent time-spent-following. In their research, the effect of an upstream signalized intersection on the two-lane highway segment was to modify the distribution of entering headways. The condition with no signalized intersection is represented by assuming the negative exponential distribution of headways for entering traffic, which is derived from the Poisson distribution for random arrivals. However, the upstream signalized intersection will modify the headway distribution of the traffic stream entering the downstream two-lane highway segment.

In TWOPAS, the distribution of headways is defined through the input variable, Entering Percent Following (EPF), which is the percent of the total number of vehicles in the direction of travel that are following in platoons, defined as headways less than 3.0 seconds, as they enter the road being analyzed. In Dixon et al.'s research, it was assumed that as long as the percentage of vehicles following, immediately downstream of the signalized intersection, could be determined, it was appropriate to represent the effects of the signalized intersection through the EPF parameter. Vehicle headways were assumed to follow a random distribution, and EPF was calculated using a cumulative exponential distribution of headways less than 3.0 seconds.

The analysis procedure of two-lane highway segment affected by the upstream signalized intersection operations was broken down into four steps.

Step 1: Determine the percentage of vehicle following (EPF) downstream of the signalized intersection.

Step 2: Determine the Percent Time Spent Following (PTSF) for the downstream highway section without the upstream signalized intersection.

Step 3: Estimate the *PTSF* for the downstream highway section with the upstream signalized intersection. In this step, two methods can be used. One method is using TWOPAS and another method is using the HCM 2000 two-lane highway directional analysis procedures and deterministic adjustment factors.

Step 4: Estimate the level of service based on the criteria suggested in the HCM 2000 two-lane highway analysis procedure.

CHAPTER 3
CONCEPTUAL OVERVIEW OF TWO-LANE HIGHWAY FACILITY ANALYSIS METHODOLOGY

This chapter describes the development of operational analysis procedures for two-lane highway facilities. The developed methodology would maintain some fidelity to the HCM by using the existing methodologies for two-lane highway segments and signalized intersections. This chapter begins with a discussion of the conceptual framework of the operational analysis procedure for a two-lane highway facility. It then puts forward a methodology of operational analysis for two-lane highway facilities and presents an overview of this methodology. Finally, the chapter discusses the selection of a facility-wide service measure and the first step of this methodology—facility segmentation.

3.1 Conceptual Framework for Facility Evaluation Methodology

To develop a methodology for the operational analysis of a two-lane highway facility, a two-lane highway with an isolated signalized intersection will be used as a model. This section discusses the conceptual framework of the operational analysis procedure for such a configuration. Aspects of the conceptual framework addressed are the definition of the two-lane highway facility, segment types, the features of operational analysis at the facility level, and the proposed methodologies.

3.1.1 The Definition of a Two-Lane Highway with Signalized/Unsignalized Intersections

In the HCM 2000, the primary highway system structure consists of points, segments, facilities, corridors and areas. A facility is a length of roadway composed of

points and segments. A point is a boundary between segments, in other words, points are where modal users enter, leave, or cross a facility, or where roadway characteristics change. A segment is a portion of a facility defined by two end points. Segments are the primary building blocks of facility analyses. In addition, a sub-segment is a further division of a segment. Whereas segments are delineated by points (e.g., intersections) or changes in geometric conditions, sub-segments for the purpose of discussion in this document are delineated only by changing operational conditions. For example, an isolated signalized intersection on a two-lane highway produces operational effects on the upstream segment. The upstream two-lane highway segment can be divided into the upstream sub-segment within the effective length of the signalized intersection, and the upstream sub-segment beyond the effective length of the signalized intersection. Although both sub-segments have homogenous geometric conditions, their operational features are different. One sub-segment is affected by the downstream signal; the other is not affected.

The potential segment types on a two-lane highway could include the following:

- Basic segment, this is a segment that consists only of a two-lane across section. Figure 3-1 shows a typical view of this type of segment.
- Basic segment with continuous specific upgrade or downgrade
- Three-lane cross section segment, with the additional lane being a passing lane. Figure 3-2 shows a typical view of this type of segment.
- Segment with an unsignalized intersection. Figure 3-3 shows a typical view of this type of segment.
- Three-lane cross section segment, with the additional lane being a center left-turn lane or a right-turn lane
- Segment terminating into a multilane highway
- Segment with an isolated signalized intersection

Figure 3-1. Typical basic two-lane, two-way highway segment

Figure 3-2. Typical two-lane, two-way highway with a passing lane

A two-lane highway with signalized intersections is a type of facility composed of isolated signalized intersections, and the basic two-lane highway. A two-lane roadway generally extends from one signalized intersection to the next signalized intersection. This type of facility is typically located in a rural area, but the signal may be present in a small town.

Figure 3-3. Typical two-lane, two-way highway with an unsignalized intersection

Figure 3-4 shows a typical view of a two-lane highway with a signalized intersection. The main features are as follows:

- Roadside development is not intense.
- Density of traffic access point is not high.
- Signalized intersections are more than 2 miles apart.
- These conditions result in a smaller number of traffic conflicts, smoother flow, and dissipation of the platoon structure.

Figure 3-4. Typical two-lane, two-way highway with a signalized intersection

3.1.2 Service Measure Consistency

LOS is a qualitative designation of the operational conditions within a traffic stream based on performance measures such as speed, travel time, freedom to maneuver, traffic interruptions, comfort, and convenience. Six levels of service are defined in the HCM, using the letters A through F for each type of facility, where A is good, and F is bad. The performance measure chosen to base LOS upon is referred to as a service measure. For application in the segment LOS analysis, every type of segment has its own service measure based on to determine its LOS.

When performing the facility-level operational analysis, occasional inconsistencies can arise because of different service measures being applied. For example, in the two-lane highway with an isolated intersection, the combination of average travel speed and percent time-spent-following is used as the service measure to evaluate the level of

service on the basic two-lane highway segment, however the service measure for a signalized intersection is based on control delay. In the HCM 2000, the measure of operational quality used for point locations is not related to highway segment. Thus, anomalies are possible when changing from one facility type to another.

Therefore selecting an appropriate service measure is a key issue in the development of operational analysis for the two-lane highway at the facility level. There are basically two methodological approaches that can be taken for an operational analysis of a facility composed of different types of segment. They are:

1. Each segment uses the service measure(s) already specified for it in the HCM 2000. The LOS of the entire facility is determined by combining the LOS of each segment in some manner.
2. A common service measure is used for each segment and point. LOS of the entire facility is determined by the aggregated service measure.

With the first methodology, no unified facility-wide service measure is applied for the segments of the entire facility. Each segment or point uses its own service measure(s) defined in the HCM 2000. Because of different service measures (e.g., ATS, PTSF, or control delay) being applied, inconsistencies can arise. For example, when determining the level of service of a two-lane highway with multiple signals, the combination of average travel speed and percent time-spent-following is used as the service measure to evaluate the level of service on an uninterrupted flow two-lane highway segment; however the service measure for a signalized intersection is based on control delay. Thus, anomalies are possible when changing from one segment type to another.

Another drawback of this methodology is the aggregation of the point and segment LOS grades into an estimate of the LOS grade for the entire facility. In the HCM 2000, the measure of operational quality used for point locations is not related to highway

segments. It is very difficult to combine the LOS of points with that of segments. Equation 3-1 gives an example method of aggregating the LOS grades of segments and points weighted by the segment length.

$$LOS = \frac{\sum_{i=1}^{n} LOS_i L_i}{\sum_{i=1}^{n} L_i} \tag{3-1}$$

Where:

LOS: the level of service of the entire facility,

LOS_i: the level of service of segment i,

L_i: the length of segment i, ft, and

n: the number of segments.

With this approach, segment LOS values are weighted by the segment length; however, LOS is not a quantitative value. It is simply a measure of user satisfaction for that service along the roadway. It is difficult to accurately convert the LOS grade into the corresponding numerical value for aggregation. Even though a certain conversion method is available, because each segment type has its own strategy to determine the LOS, every segment type needs a unique conversion method, which makes the LOS combination method somewhat complicated and possibly subjective.

In the HCM 2000, Chapter 15, the average travel speed is used as the service measure on the urban street with multiple signalized intersections at a spacing of 2.0 miles or less. The method using a common service measure for a facility consisting of multiple different segment types is a good reference. In the second methodology, a common service measure would be applied to every segment of the entire facility. The service measures at each segment are aggregated to obtain an estimate of service measure

for the entire facility. The LOS of the entire facility is determined by this aggregated service measure. The unified facility-wide service measure not only avoids many disadvantages of the application of multiple service measures, but also provides a Measure of Effectiveness (MOE) describing traffic operations in terms discernible by motorists from the scope of the entire facility.

The proposed second method would also maintain some fidelity to the *Highway Capacity Manual* by using the existing methodologies for two-lane highway segments and signalized intersections. For the two-lane methodology (HCM Chapter 20), the method for calculation of average travel speed (*ATS*) is utilized; however, percent time-spent-following (*PTSF*) is not utilized. For signalized intersections (HCM Chapter 16), the current method for the calculation of control delay is utilized.

By using a time/delay based service measure, this method will be similar to the current HCM methodology for urban streets. For transportation agencies looking to analyze the impacts of adding a lane (or lanes) to a two-lane highway, along with adding some signalized intersections, thus possibly changing the classification to an urban arterial in some sections, this will provide for consistency in the analyses (assuming the LOS thresholds are set accordingly). This methodology will be completely presented and explained in Section 3.2.

3.1.3 Impacts of Signalized Intersection on Adjacent Highway Segments

Another important issue in the development of an operational analysis at the facility level is the impacts with different segment types. Continuing research has shown that installing a signalized intersection on a two-lane highway can produce effects on traffic operations of the upstream and downstream two-lane highway segment.

To illustrate the potential effects of an isolated signalized intersection on the two-lane highway operation, CORSIM and TRANSYT-7F programs are used to simulate the operations of a two-lane highway with an isolated signalized intersection.

3.1.3.1 Effects of intersections on the upstream two-lane highway operation

When vehicles approach the signalized intersection facing a red signal indication, drivers will safely stop their vehicles with sufficient sight distance to avoid entering the intersection or colliding with queued vehicles. Here CORSIM is used to model the variation of average travel speed as vehicles approach the upstream signalized intersection.

Six CORSIM simulations are made with 30 replicate runs for each. The six conditions are as follows:

Table 3-1. Traffic simulation conditions

	Traffic Volume (veh/h)	With or Without Signal
1	600	With
2	600	Without
3	1000	With
4	1000	Without
5	1400	With
6	1400	Without

The operational effects of a signalized intersection on the two-lane highway based on average travel speed are shown in Figure 3-5. This figure is directly derived from preliminary simulation runs. Figure 3-5 shows a comparison of the modeled average speed as it varies along a two-lane highway with an isolated signalized intersection and with no isolated signalized intersection under different traffic flow levels. As seen in Figure 3-5, on the two-lane highway segment upstream of the signalized intersection, when vehicles enter the basic two-lane highway segment, the difference in the values of

average travel speed is very small. The average travel speeds along the two-lane highway with an isolated signalized intersection are very much in agreement with those with no isolated signalized intersection. When near to the signalized intersection (about 1000 ft before the stop line of signalized intersection), the difference in average travel speed becomes very large. The average travel speed under the condition with an isolated signalized intersection drops dramatically because of queuing in front to the signal. After the signalized intersection, average traffic speed quickly increases and returns to its former level.

So installing a signalized intersection on a two-lane highway significantly affects traffic operations on the upstream two-lane highway segment based on average travel speed, and the effective length of a signalized intersection is greater than its actual length. The effective length of the upstream influence area of a signalized intersection is defined from the dividing point, at which vehicles begin decelerating to the stop line of this signalized intersection

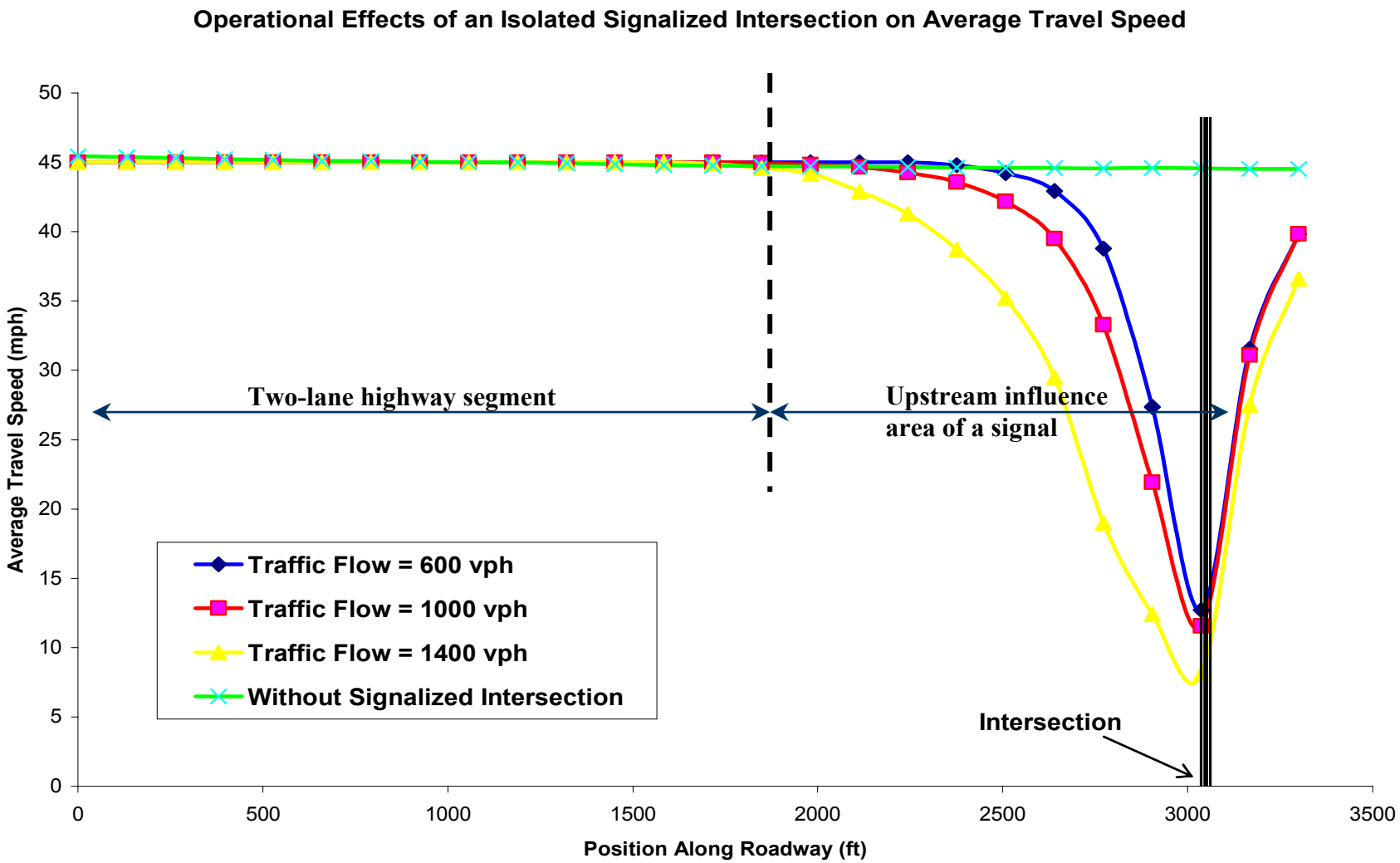

Figure 3-5. Effect of a signalized intersection on average travel speed

3.1.3.2 Effects of intersections on the downstream two-lane highway operation

After passing through the signalized intersection, the vehicle platoon will go into the downstream two-lane highway. The platoon dispersion pattern is affected not only by the upstream signalized intersection, but also by the right-turn vehicles and left-turn vehicles from minor streets. There are three movements that contribute to the flow profile, as follows:

- through movement from the major street
- right-turn movement from the minor street
- left-turn movement from another minor street

The start-and-stop operation of signals on the two-lane highways tends to create platoons of vehicles that travel along a two-lane highway link. Here TRANSYT-7F is used to model the dispersion of these platoons as they progress along the downstream two-lane highway segment. In TRANSYT-7F, for each time interval (step), *t*, the arrival flow downstream is found by the following recurrence equation [9]:

$$v'_{(t+\beta T)} = F \cdot v_t + \left\lfloor (1-F) \cdot v'_{(t+\beta T-1)} \right\rfloor \tag{3-2}$$

Where:

$v'_{(t+\beta T)}$: predicted flow rate (in time interval of the predicted platoon);

v_t: flow rate of the initial platoon during step *t*;

β : an empirical factor, generally 0.8;

T: the cruise travel time on the link in steps; and

F: a smoothing factor

$$F = (1 + \alpha \cdot \beta \cdot T)^{-1} \tag{3-3}$$

Where:

α : platoon dispersion factor (PDF)

Equation 3-3 is based on field studies by Hillier and Rothery [10]. The factor α has been found by researchers to best represent measured dispersion on typical urban streets in the U.S. when it was set at 0.35. This PDF will vary to consider site-specific factors such as grades, opposing flow interference and other sources of impedance.

The diagrams below illustrate the nature of platoon dispersion on the downstream two-lane highway of a signalized intersection. As traffic moves downstream, the initially tight platoon formed from the departing queue tends to disperse the farther downstream it travels. Because drivers tend to maintain safe headways, or spacing, between vehicles and often travel at different speeds, the platoon tends to spread out - a few moving ahead and some dropping back. The flow rate decreases with time as the platoon reaches each point of observation. They are the "snapshots" of the traffic flow at the different observation stations of the downstream link (the average traffic flow is 1200 veh/h).

The first diagram (Figure 3-6) illustrates a platoon after it has traveled 300 feet after being stopped at the upstream signalized intersection. The most intense portion of the platoon is at a rate higher than 1870 veh/h, and the lowest portion is at a rate near 0 veh/h. The platoon has spread out extremely unevenly. At this point, the timing plan of the upstream signal and traffic streams from the minor streets produce significant effects on the pattern of platoon dispersion.

As traffic moves downstream, the initially tight platoon formed from the departing queue tends to disperse the farther downstream it travels. The second diagram (Figure 3-7) illustrates the same platoon after traveling one-full mile, or 5280 feet. Notice that after a full-mile, the most intense portion of the platoon is a rate slightly higher than 1500

veh/h, whereas after 300 feet the most intense portion of a platoon is approximately 1900 veh/h. At this point, the platoon has spread out to cover the whole portion of the cycle. The effect produced by the upstream signal on the platoon dispersion becomes smaller.

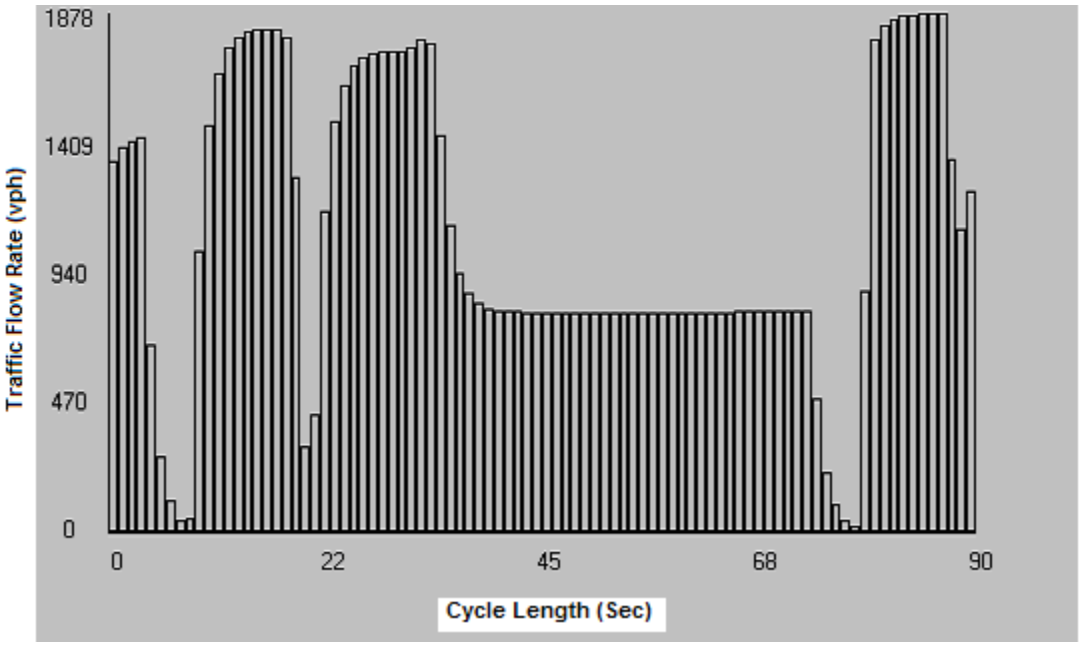

Figure 3-6. Platoon dispersion, 300 ft from the upstream signalized intersection

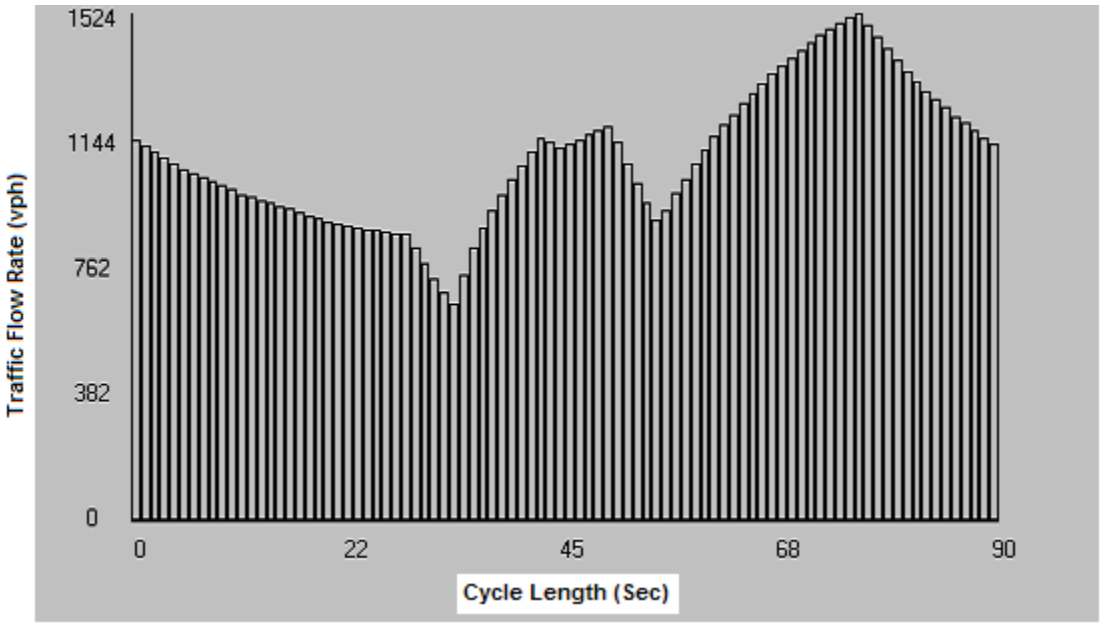

Figure 3-7. Platoon dispersion, 5280 ft from the upstream signalized intersection

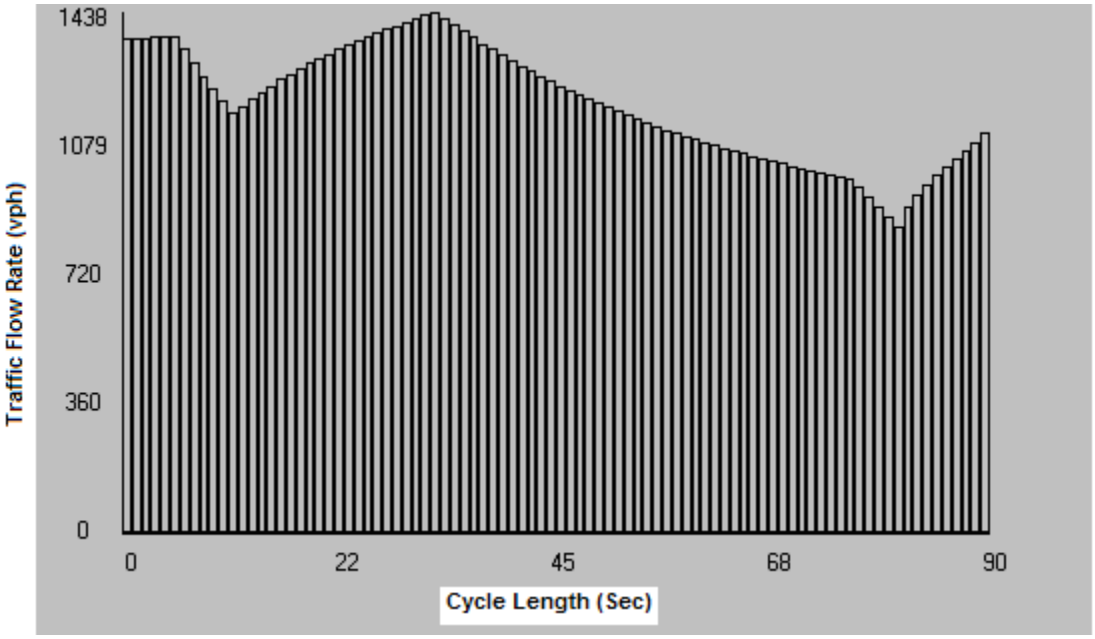

Figure 3-8. Platoon dispersion, 10560 ft from the upstream signalized intersection

The same phenomenon can be observed from the third diagram (Figure 3-8), which illustrates the same platoon after traveling 2 miles, or 10560 feet. The platoon has spread out more evenly and covered the whole portion of the cycle. The most intense portion of the platoon is about 1400 veh/h, which is near to the average flow rate of 1200 veh/h. At this point, the effect produced by the upstream signal on the platoon dispersion is negligible.

Based on the above analysis, it can be determined that the upstream signalized intersection and traffic streams from the cross street alter the pattern of platoon dispersion. The degree of platoon dispersion in turn directly affects vehicle delay, speed, queuing, and other measures of effectiveness.

Given the potential impact of a signalized intersection on downstream two-lane highway operations, it is necessary to investigate the effects further when performing an operational performance assessment of two-lane highway facilities. To quantify the

effect of a signalized intersection on the downstream two-lane highway segment, a key issue is to determine this effective length of influence area, downstream segment of the signalized intersection.

3.2 Methodological Approach

Based on the discussion in the former section, using a common service measure for the entire facility is preferred. A two-lane highway with an isolated signalized intersection will be used as a model. This section begins with a discussion of service measure selection, facility segmentation for a two-lane highway with isolated intersections. Next the section presents an overview of the operational analysis procedures for a two-lane highway with an isolated signalized intersection.

3.2.1 Service Measure Selection

In this methodology, a common service measure is applied to every segment of the entire facility. The LOS of the entire facility is determined by this aggregated service measure. One of the key steps in the methodology is the selection of a service measure(s) used to define the overall level of service for the facility. Based on the features of the two-lane highway with occasional signalized and unsignalized intersection, some candidate service measures are estimated. They are described as follows:

- **Volume to Capacity Ratio (v/c)**

  The *v/c* ratio is often used as a measure of the sufficiency of existing or proposed capacity. According to the 2000 HCM, this *v/c* ratio measure of capacity sufficiency of the overall intersection is a good indication of whether the physical geometry design features and the signal design provide sufficient capacity for the intersection. But the ratio is not sensitive to speeds and travel time. With an acceptable LOS grade, a *v/c* ratio may indicate that the same facility is operating at or near all capacity.

Conversely, road segments operating at deficient levels of service may have an acceptable *v/c* ratio in cases where the adjoining intersections are not operating efficiently.

Generally, the v/c ratio is used as a measure of the sufficiency of existing or proposed capacity. The ratio however, is not sensitive to speeds and travel time. The v/c ratio is better as a measure of the capacity sufficiency, but not good as a measure of the quality of service. The combination of v/c ratio and other performance measures may be better.

- **Average Travel Speed (ATS)**

ATS reflects the mobility function of traffic facilities. Speed, as represented by *ATS*, is a very important part of the LOS definition and is also easy for the public to understand. And it is easily calculated using the data that is already being collected. As a space-average measure, *ATS* can be estimated in the field by travel time studies or by measure of spot speeds. One potential drawback to the use of average travel speed as the single service measure for two-lane highways is that it is not as sensitive as *PTSF* to the relative balance between passing demand and passing supply.

- **Percent Time-Spent-Following (PTSF)**

Given the platooned nature of traffic on the two-lane highway, *PTSF* represents freedom to maneuver and the comfort and convenience of travel on a two-lane highway. However, some researchers [5] think this measure is not appropriate for application to developed, tourist-oriented sections, such as US Route 1 in the Florida Keys, on which motorists are more concerned about the ability to maintain a reasonable speed. *PTSF* is also a space-averaged measure, which is difficult to measure directly in the field. While the HCM suggests that it be estimated as the

percentage of vehicles traveling at a headway of 3 seconds or less at a representative point, the LOS estimation is very sensitive to the headway threshold [11].

Both *ATS* and *PTSF* are measured over a section of roadway. In the highway structure system of the HCM, the signalized intersection is regarded as a point, or a segment with a short length, so *ATS*, *PTSF*, or their combination is a conceptually adequate service measure for two-lane highway segments, but a poor one for the signalized intersection by itself. So *ATS*, *PTSF*, or their combination is not a good facility-wide service measure for the facility consisting of two-lane highway segments and signalized intersections.

- **Travel Time**

Travel time, particularly in the context of reliability, is gaining momentum as a performance and service measure for some types of facilities. Travel time is essentially the same performance measure as speed (it is simply the reciprocal). However, just like speed, travel time alone is not sufficient as a service measure over a length of two-lane highway facility. It would also need to be qualified per some unit distance (e.g., min/mi). In this situation, it is generally a simple task to convert to other time-based measures if so desired. Currently, for the purposes of the HCM 2000 methodologies, speed is still generally used in preference to travel time.

- **Percent Free Flow Speed (PFFS)**

PFFS is defined as the ratio of vehicle average travel speed to free flow speed. Washburn, et al. [5] proposed percent free flow speed as the primary performance measure for two-lane highways in developed areas. This measure makes some sense for these areas due to the fact that drivers probably do not have much expectation for

passing in these areas and they are willing to tolerate following other vehicles as long as their speed is close to the desired free-flow speed.

- **Density**

  Density is used as the primary service measure for the types of uninterrupted flow facilities, such as freeway and multilane highway. Given the platooned nature of traffic on a two-lane highway, density is much less evenly distributed on a two-lane highway than on a freeway or multilane highway [11]. Density is not a good service measure for the two-lane highway facility. Percent time-spent-following does a much better job of representing density; percent time-spent-following is the percentage of the total travel time that drivers spend traveling in local high-density conditions. An additional difficulty with density is that direct measurement of it in the field is difficult, requiring a vantage point for photographing, videotaping, or observing significant lengths of highway. Furthermore, conceptually it does not work for signalized intersections.

- **Control Delay**

  Control delay includes "Movements at slower speed and stops on intersection approaches as vehicles move up in queue position or slow down upstream of an intersection" [1]. It is the principal service measure for evaluating LOS at intersections, which are point locations within a traffic network. However, the measure of operational quality of effectiveness used for point locations can not be used for highway segments, such as two-lane highway segments.

- **Percent Time-Delayed**

  Percent time-delayed is defined as the percentage of the travel time that vehicles on a given roadway segment must travel at speeds less than their desired speed due to

inability to pass or traffic control during some designated time interval. Percent time-delayed is a good performance measure for interrupted-flow facilities, such as two-lane highways with occasional signalized intersections. It reflects the effects of speed reductions by motorists due to restrictive roadway geometry, traffic control, and other traffic, and represents the degree to which drivers are forced to travel at speeds less than their desired speed.

Just like other delay-related performance measures, percent time-delayed also has a direct economic interpretation and can be used in economic studies if the monetary value of a travler's delay can be established. The primary drawback of percent time-delayed as a performance measure is the difficulty of measuring it in the field.

Table 3-2. Service measure evaluation

	Undeveloped uninterrupted two-lane segments	Developed uninterrupted two-lane segments	Intersection influence area	Facility incorporating two-lane highway and signalized intersection
Volume/Capacity Ratio	F	F	F	F
Average Travel Speed	G	G	P	P
Percent Time-Spent-Following	E	F	P	P
PTSF and ATS	E	E	P	P
Travel Time	G	G	P	P
Percent Free-Flow-Speed	G	E	P	P
Density	F	F	F	F
Control Delay	P	P	E	P
Percent Time-Delayed	G	G	G	G

Note: E = excellent, G = good, F = Fair, P = poor

Table 3-2 summarizes the evaluation of potential service measures for a two-lane highway with signalized intersections. Based on a review of the advantages and disadvantages of candidate service measures discussed above, it is concluded that percent time-delayed is an appropriate selection as the single service measure for the interrupted-flow facility of a two-lane highway with signalized intersections. Percent time-delayed is a measure that directly relates to the driver's experience. It not only represents freedom to

maneuver and the comfort and convenience of travel on a two-lane highway, but also reflects the effects of speed reductions due to traffic control (e.g., signalized intersection, stop sign), and due to restrictive geometric features (e.g., vertical grade, horizontal curve, no-passing zone), and other traffics (e.g., opposite traffic flow, heavy vehicles).

3.2.2 Facility Segmentation

To perform an operational analysis for a facility consisting of different segment types, and obtain the LOS of the facility, the entire facility is divided into several segments. Thus, the analysis methodology must prescribe how to segment the facility.

In the HCM 2000, Chapter 15, *Urban Streets,* presents the methodology for evaluating arterials in urban and suburban areas with multiple signalized intersections at a spacing of 2.0 miles or less. In this methodology, the urban street is divided into segments, which is the full distance from one signalized intersection to the next. Figure 3-9 illustrates the segmentation method for this methodology. Running time is computed for each segment, along with control delay at each signalized intersection. In this methodology, the signalized intersection is regarded as a point location within a traffic network, and control delay is regarded as a typical point performance measure without covering any distance. In the HCM 2000 [1], the definition of control delay implies that control delay happens not at a point, but actually within a certain distance. Although the time lost due to slow movement before and after a stop is technically part of the running time, it is also included in control delay. Thus, this segment division method introduces error due to the segment between intersections being longer than they should.

In the HCM 2000, Chapter 16, *Signalized Intersections,* presents the methodology for evaluating isolated signalized intersections. In this methodology, the signalized intersection is regarded as a single isolated traffic control installation. So the length of the

signal influence area is not a key factor in determining control delay, or in the decision of LOS based on control delay.

When evaluating highways with multiple signalized intersections, the signalized intersection should not be regarded as an isolated point. The impacts between signalized intersections and highway segments should be taken into account. In Sections 3.1.3.1 and 3.1.3.2, it has been shown that the installation of a signalized intersection actually affects the operations on the highway segments, and the signal influence area does extend a certain length. So when evaluating a two-lane highway with signalized intersections, the signalized intersection is not regarded as a single point, but as a segment with a certain length.

Figure 3-10 shows the division of a two-lane highway with multiple isolated signalized intersections. The whole facility is divided into three types of segments, described as follows:

- Type 1: the basic two-lane highway. This type of segment may be located in the upstream or downstream of the signalized intersection, but beyond the signal effective length. These segments are not affected by signalized intersections.
- Type 2: the signal influence area. In the highway structure system of HCM 2000, the signalized intersection is defined as a point; the boundary between segments. In this operational analysis methodology, it will be regarded as a segment with a certain length, which is composed of not only its own actual length, but also the deceleration and acceleration lengths. The length of the signal influence area corresponds to the three components of control delay—deceleration delay, stop delay, and acceleration delay.
- Type 3: the affected downstream segment. This type of segment is still the two-lane highway, but affected by the upstream signalized intersection. Potential operational effects on this segment are produced by the upstream signalized intersection and traffic flows coming from the cross street. Note the length of this type of segment does not include the acceleration length of the signal influence area.

The lengths of these different segment types should add up to the total length of the analyzed facility.

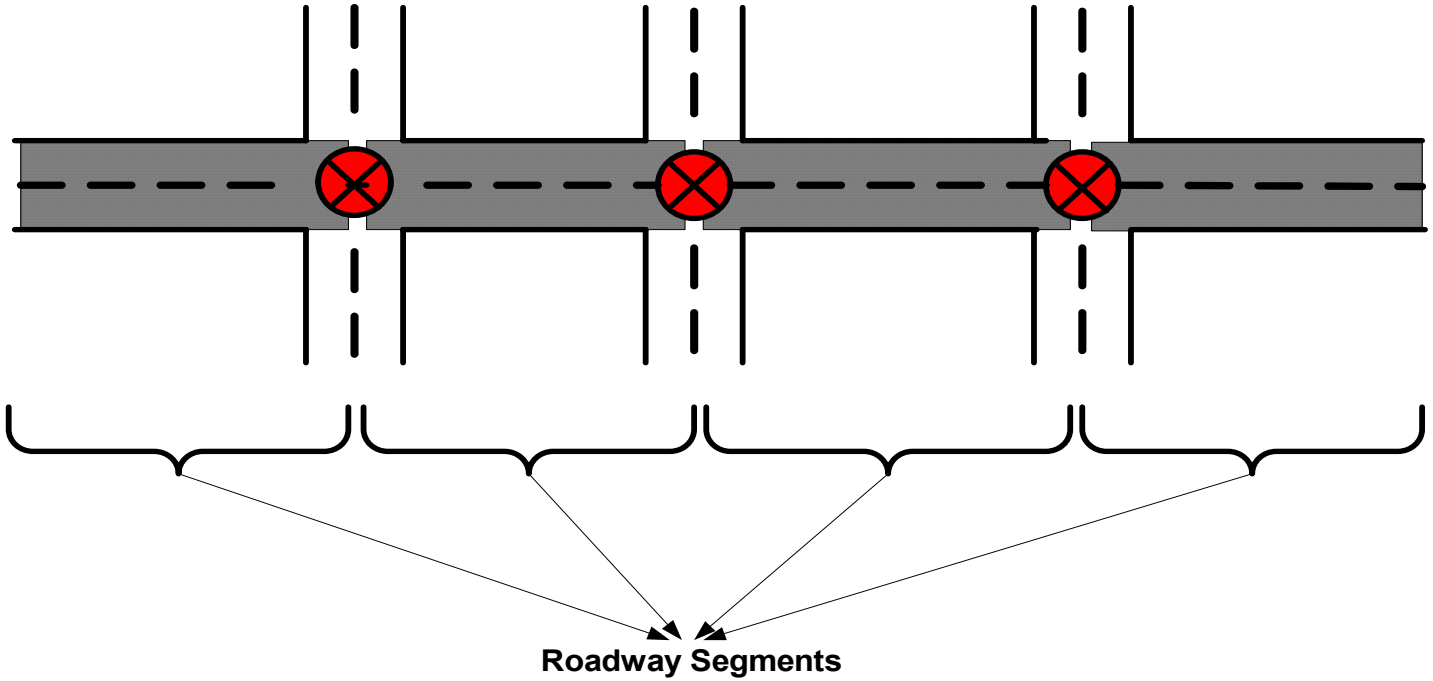

Figure 3-9. Segment division of a two-lane highway with multiple isolated signalized intersections

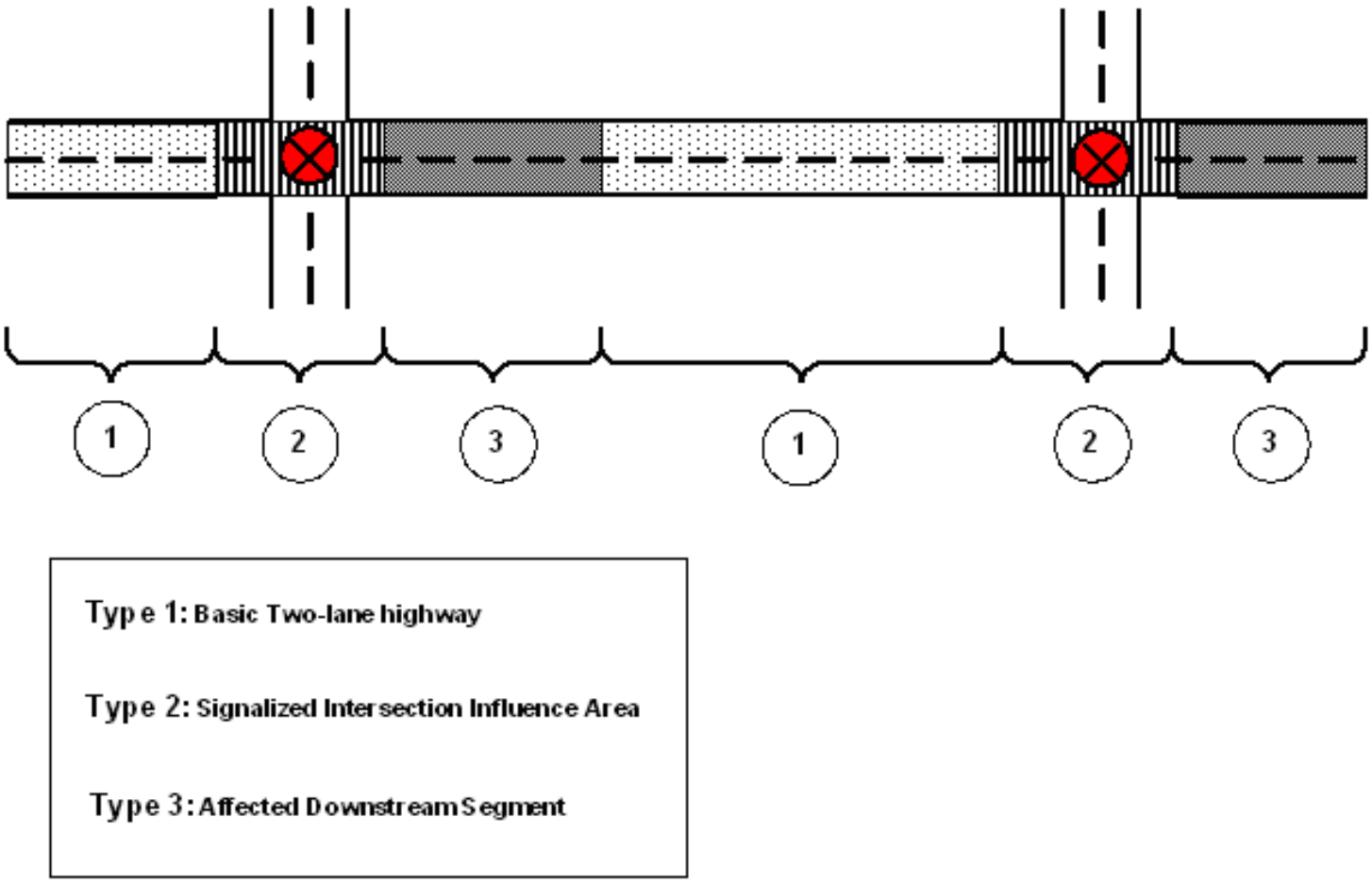

Figure 3-10. Facility segmentation of a two-lane highway with multiple isolated signalized intersections

3.2.3 Overview of Computational Methodology

In this methodology, a common service measure would be applied to every segment of the entire facility. The service measures at each segment are aggregated to obtain an estimate of the overall service measure for the entire facility. The LOS of the entire facility is determined by this aggregated service measure.

An example of analyzing a two-lane highway with signalized intersections using this methodology is provided here. Percent time-delayed is applied as the common service measure for the whole facility composed of two-lane highway segments and signalized intersections.

The first step in this analysis is to segment the facility based on the features of segment type. The second step is to determine the free-flow speed. The free-flow speed is used to determine the average travel speed and delay time at each segment. The basic free-flow speed for the two-lane highway is observed at basic conditions and ranges from 45 to 65 mile/h, depending on the highway's characteristics. The speed study should be conducted at a representative site within the study section. The best location to measure free-flow speed on the two-lane highway is mid-block and as far as possible from the nearest signalized or stop-controlled intersection. If field observation of free-flow speed is not practical, free-flow speed on the two-lane highway may be estimated using the method presented in the HCM 2000.

The next step in the analysis is to perform operational analysis at the point and segment levels. At the first type of segment, which is the basic two-lane highway sections, and not affected by the signalized intersection, the average travel speed can be calculated using the two-lane highway procedure presented in Chapter 20 of HCM 2000.

The length of the conventional two-lane highway segment is determined by the actual placement of the signalized intersection within the analysis section.

At the second type of segment, which is the signalized intersection influence area, the control delay is the portion of the total delay for a vehicle approaching and entering a signalized intersection. Control delay concludes the delays of initial deceleration, move-up time in the queue, stops, and acceleration. It can be calculated using the signalized intersection procedure presented in Chapter 16 for the through-traffic lane group. The length of the signal influence area includes the deceleration length, stopping length, and acceleration length.

The third type of segment is the downstream segment, affected by the upstream signalized intersection, and the traffic flow coming from the cross streets. The potential impacts of the signalized intersection on this segment will be assessed further in the term of average travel speed. The effective length of influence area downstream of the signalized intersection is also decided. For the analysis of this type of segment, statistical methods and TWOPAS simulation model will be used to quantify the impacts. The methodology will be presented in Chapter 4.

Once average travel speed on the two-lane highway segments and control delay within the signalized intersection are determined, the delay time on the two-lane highway segments and the signalized intersection can be calculated using the following equations.

Delay time on the two-lane highway segment:

$$D_H = \frac{L_H}{S_H} - \frac{L_H}{FFS} \tag{3-4}$$

Where:

D_H : delay on the two-lane highway segment, s/veh

L_H : length of two-lane highway segment, ft

FFS : free flow speed for the two-lane highway segment, ft/s

S_H : average travel speed for two-lane highway segment, ft/s

Control delay at the signalized intersection:

$$D_S = d_1(PF) + d_2 + d_3 \tag{3-5}$$

Where:

D_S : control delay per vehicle at the signalized intersection, s/veh

d_1 : uniform control delay, s/veh

d_2 : incremental delay, s/veh

d_3 : initial queue delay, s/veh

PF: uniform delay progression adjustment factor

After estimates of delay time at the segment and point levels are obtained, segment and point delays are then added together to obtain the entire facility estimate. Percent time-delayed is then computed through dividing total delay time on the entire facility by the total travel time at the free-flow speed on the entire facility. Equation 3-6 shows the aggregation of point and segment results to obtain an estimate of percent time-delayed for the entire facility. After the facility-wide performance measure, percent time-delayed is obtained, the facility's LOS grade can be determined based on the LOS table. An initial set of thresholds will be established as part of this research, but further research on this issue will likely be warranted. Figure 3-11 illustrates the analysis procedure for determining LOS on the two-lane highway with signalized intersections.

$$PTD = \frac{\sum_{H,S}(D_H + D_S)}{\sum_{H,S}(\frac{L_H}{FFS_H} + \frac{L_S}{FFS_S})}$$

$$L = \sum_{H,S} L_H + L_S \tag{3-6}$$

Where:

PTD: percent time-delayed per vehicle for the entire facility, %

D_H: delay time per vehicle for the two-lane highway segment, s/veh

D_S: delay time per vehicle for the signalized intersection influence area, s/veh

FFS_H: free flow speed for the two-lane highway segment, ft/s

FFS_S: free flow speed for the signalized intersection influence area, ft/s

L: length of the entire facility, ft

L_H: length of the two-lane highway segment, ft and

L_S: length of the signalized intersection influence area, ft

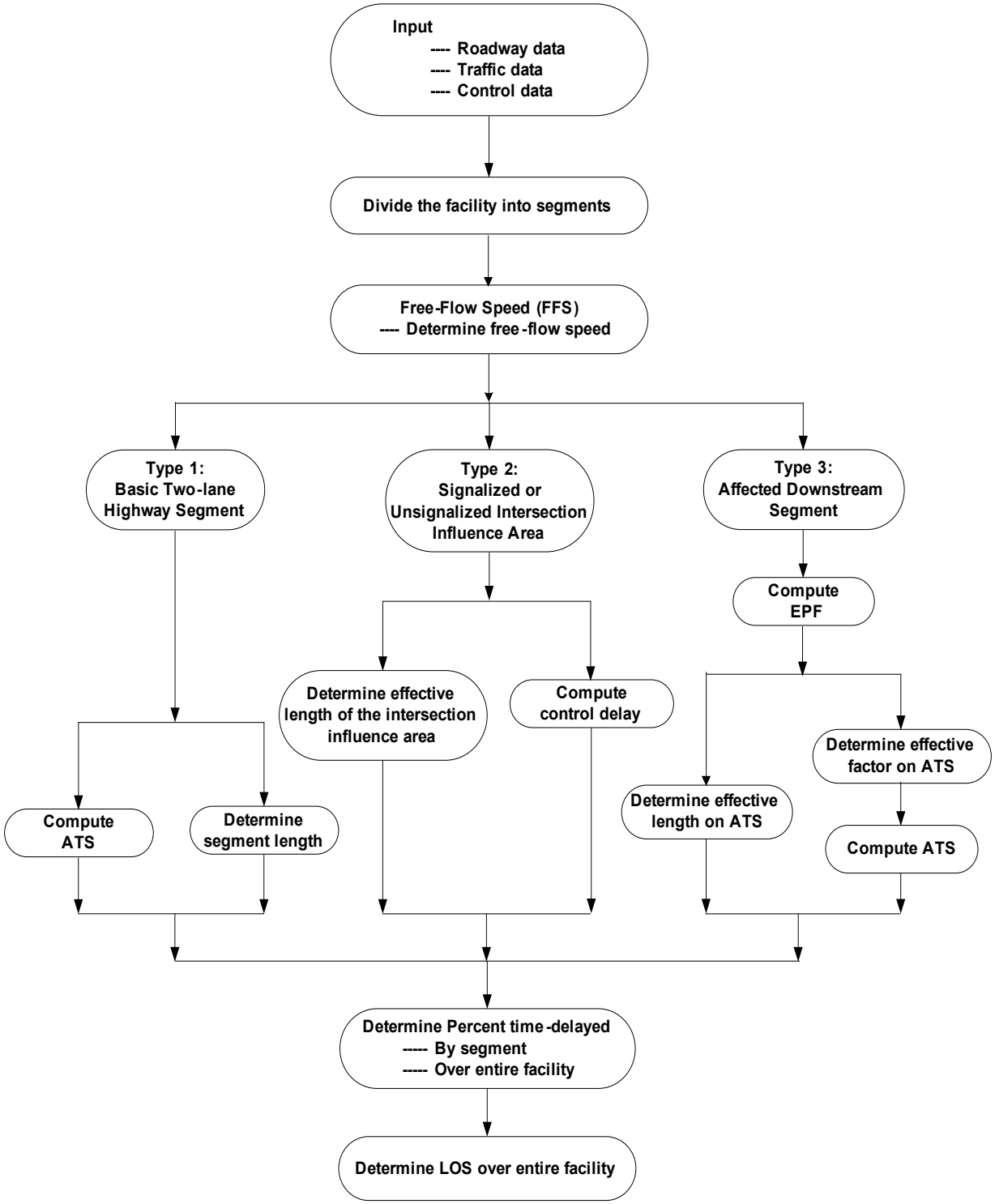

Figure 3-11.Flow diagram of two-lane highway facility operational analysis methodology

In developing this methodology, some simulation and analysis methods were used at each methodological step. Table 3.3 summarizes those programs and methods used.

Table 3-3. Summary of programs used at each step

Step 1: Divide the facility into segments.
Facility is divided into segments based on its geometric conditions. Segments in the facility are not regarded as isolated. The interactions between different segments are also taken into account.
Step 2: Determine the free-flow speed.
The methods presented in the HCM 2000 are used – field measurement, or estimating free-flow speed. (Chapter 20, Page 20-4, 20-5)
Step 3: Determine segment length.
3.1 Determine the length of the signalized intersection influence area The length of signal influence area is the sum of three components, deceleration distance, queue length, and acceleration distance. Acceleration distance can be determined using the linearly-decreasing acceleration model. The deceleration distance and queue length are determined using the CORSIM simulation program and regression analysis. 3.2 Determine the length of the affected downstream segment The length of the affected downstream segment is determined using the TWOPAS simulation program and regression analysis. The method is built upon work done previously by Dixon, et al. 3.3 Determine the length of the TWSC intersection influence area The length of TWSC influence area is the sum of three components, deceleration distance, queue length, and acceleration distance. The Synchro/SimTraffic program was used to determine the queue length.
Step 4: Calculate the unaffected average travel speed on basic two-lane highway segments.
The analysis methods presented in Chapter 20 of the HCM 2000 are used to calculate the average travel speed on basic two-lane highway segments.
Step 5: Calculate control delay at the signalized or unsignalized intersection influence area.
The analysis methods presented in Chapter 16 of the HCM 2000 are used to calculate control delay at the signalized intersection. The analysis methods presented in Chapter 17 of the HCM 2000 are used to calculate control delay at the Two-Way Stop-Controlled intersection.
Step 6: Determine average travel speed on the affected downstream segment.
An adjustment factor is used to account for the effects of the upstream signalized intersection on the downstream highway segment. The values of adjustment factor are determined using the TWOPAS simulation program and regression analysis.
Step 7: Determine the delay time at every segment.
The delay time at every segment is calculated using Equations 3-4 and 3-5.
Step 8: Determine the percent time-delayed and LOS of the entire facility.
The percent time-delayed is calculated using Equation 3-6. LOS is determined from Table 5-8 (Chapter 5).

In this methodology, three simulation programs are selected to simulate traffic operations on the two-lane highway in the vicinity of an intersection. The reasons why they were selected are presented in Table 3-4.

Table 3-4. Simulation models evaluation

Simulation Model	Evaluation
CORSIM	CORSIM, developed by the Federal Highway Administration, is the most widely used and accepted traffic simulation model in the U.S. It has the ability to simulate traffic operations on a two-lane roadway and includes detailed modeling of traffic signal operations. In addition, the CORSIM simulation model simulates the traffic system on a vehicle-by-vehicle basis by updating roadway position, speed, acceleration, and other state variables in discrete time steps. The ability to calibrate, modify, and manipulate these parameters is a key characteristic of the CORSIM simulation model amenable for use to determine the effective length of the signal influence area. CORSIM cannot simulate passing maneuvers using the on-coming lane of traffic. However, experience shows that drivers usually do not undertake passing maneuvers in the vicinity of a traffic signal. Under this assumption, it is feasible to use a program such as CORSIM to model vehicular operations on a two-lane roadway in the vicinity of a traffic signal, and determine the effective length of the signal influence area on the upstream two-lane highway segment.
TWOPAS	TWOPAS is the only simulation program that is able to simulate the passing maneuver operation on the two-lane highway using the opposing lane, and was developed with U.S. data. Meanwhile the input variable, Entering Percent Following reflects the potential effects of a signalized intersection on the downstream two-lane highway operations. In this study, TWOPAS was used to simulate the effects of the signalized intersection on the downstream segment.
Synchro and SimTraffic	Synchro is a complete software package for modeling and optimizing traffic signal timing. SimTraffic is a companion product to Synchro for performing microscopic simulation and animation. SimTraffic uses a Synchro file for input. It is very convenient to obtain the queue information from its simulation output. In Synchro, there are two options for reporting the intersection statistical information, the Synchro method or the HCM method. The HCM method is used to determine the queue length to maintain fidelity to the existing HCM.

CHAPTER 4
DEVELOPMENT OF FACILITY SEGMENTATION COMPUTATIONS

The methodology developed in Chapter 3 divides the entire facility into three types of segments. They are the basic two-lane highway, the signalized intersection influence area, and the affected downstream two-lane highway segment. In this methodology, the overall LOS for the facility is calculated by aggregating the service measure values of the segments, as weighted by segment length. The focus of this chapter is the determination of the length of each of the component segments of a two-lane highway facility.

Here the relation of the signalized intersection influence area and the affected segment downstream of the signalized intersection is clarified again. The components of the signal influence area include deceleration distance, stopping distance, and acceleration distance, which are consistent with those of control delay defined in the HCM 2000–deceleration delay, stop delay, and acceleration delay. The segment delay time for the signalized intersection influence area is determined by the intersection control delay. The affected downstream segment is still affected by the upstream signalized intersection. As the traffic stream discharges from the upstream intersection into the downstream highway segment, it will take some distance for traffic to return to the same flow condition as before the influence of the signal. The delay time for the affected downstream segment is determined by the difference in free-flow travel time and actual travel time.

This chapter includes two sections. Section 1 presents three methods to determine the length of a signalized intersection influence area and their advantages and

disadvantages are evaluated. Section 2 explores the methodology to determine the length of the downstream segment affected by the upstream signal. How to accurately define the headway distribution and calculate the parameter of EPF is discussed in this section. Finally, this methodology is verified by comprehensive comparisons with other simulation programs.

4.1 Effective Length of the Influence Area Upstream of the Signalized Intersection

This section discusses the methodology of determining the effective length of the signalized intersection influence area. Three methods are presented. The first method is to apply the recommended length in FDOT's 2002 Level/Quality of Service Handbook; the second one determines the length of a signalized intersection influence area using the HCM equations; the third one is to apply simulation method.

4.1.1 Recommended Length in FDOT's 2002 Level/Quality of Service Handbook

In FDOT's 2002 Level/Quality of Service Handbook, for a preliminary engineering analysis FDOT recommends breaking the facility into uninterrupted and interrupted flow segments [4]. *The interrupted flow intersection segments, "intersection influence areas," extend 0.5 miles in length centered on the midpoint of the crossing facility. The LOS for this influence area is determined by the intersection LOS.* Figure 4-1 shows an example of how to determine the intersection length in the two-lane highway facility with signalized intersections.

In this example, a two-lane highway with a signalized intersection extends 10 miles, and the isolated intersection is located at the 6-mile point. The first 5.75 miles would be regarded as a two-lane highway segment, the next 0.5 miles would be regarded as the intersection area, and the last 3.75 miles would be regarded as a two-lane highway segment.

The recommended length in FDOT's 2002 Level/Quality of Service Handbook is only a simplified value. It does not take into account any actual factors such as traffic conditions and signal timing plans in the field. A new method to determine the effective length of the signalized intersection influence area under specific conditions is presented here from a component-based perspective.

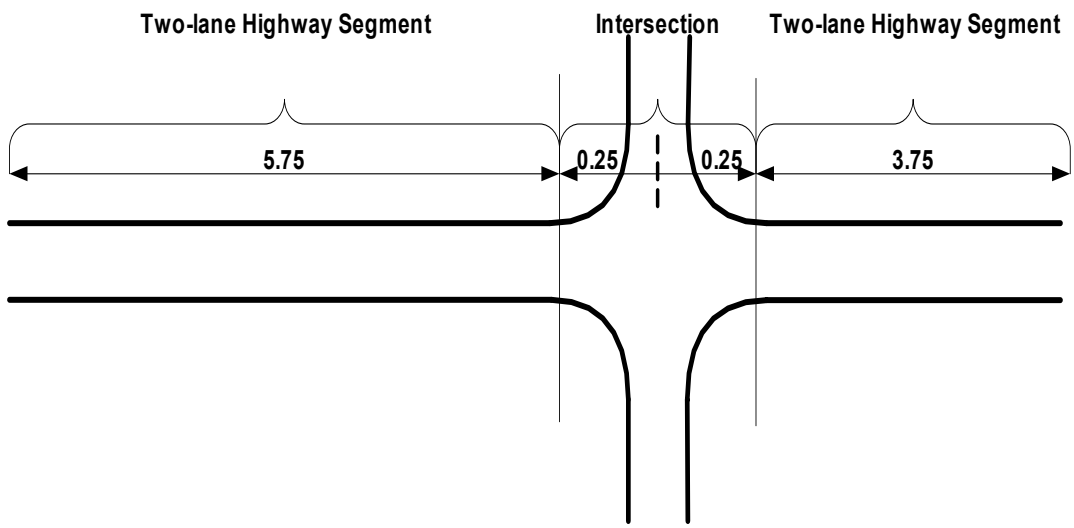

Figure 4-1. Length of intersection area

4.1.2 Components of the Signal Influence Area

The signalized intersection influence area is the place where control delay happens. Control delay is defined as the total delay due to the signalized intersection and includes deceleration delay, stop delay, and acceleration delay. The length of the signalized intersection influence area should be consistent with control delay, and its components correspond to those of the control delay. That is, the components of the signalized influence are deceleration length, stopping length, and acceleration length. The detailed distance-time diagram shown in Figure 4-2 is useful for defining the general shape of the

relationship of control delay (Time), and the length of the signalized intersection area (Distance) associated with a specific vehicle.

Figure 4-2 shows the main delay terms at a signalized intersection, and components of the signal influence area. Before Point 1 on the time-distance diagram, the vehicle is moving at a relatively uniform speed. From Point 1 to Point 2, the vehicle decelerates until it stops at Point 2 to join the standing queue before the signalized intersection. The vehicle remains stopped between Points 2 and 3. Between Points 3 and Point 4, the vehicle accelerates until it reaches a uniform speed again at Point 4. Notice that Point 3 is the stop bar.

In Figure 4-2, the deceleration distance L_D is given by

$$L_D = L_2 - L_1 \tag{4-1}$$

Similarly, the stopping distance L_S is given by

$$L_S = L_3 - L_2 \tag{4-2}$$

Similarly, the acceleration distance L_A is given by

$$L_A = L_4 - L_3 \tag{4-3}$$

To determine the overall length of the signal influence area, the lengths of each of the three components must be determined. The method to determine the lengths of these three components will be discussed in the following section.

Figure 4-3 illustrates several kinds of conditions for which vehicles pass through a signalized intersection. Figure 4-3(a) shows the condition for which vehicles are near the intersection facing a red signal indication and a queue exists in front of the intersection, so drivers will safely stop their vehicles within sufficient sight distance to avoid entering the intersection or colliding with queued vehicles. For this condition, the effective length

is equal to the sum of stopping sight distance (*SSD*) and queue length. At the end of the red period, the queue length increases to the maximum value.

Figure 4-3(b) shows the condition for which vehicles are near the intersection facing a green signal indication, a queue exists in front of the intersection, and drivers do not need to stop their vehicle completely, but still need to decelerate. For this condition, the effective length is still equal to the sum of *SSD* and queue length. At the end of the green period, the queue length decreases to the minimum value. Vehicle approaches the signal under different conditions. Here the average queue length is used.

Figure 4-3(c) shows the condition for which vehicles are near the intersection facing a green signal indication, and no queue exists in front of the intersection. In this case, the effective length is equal to *SSD* only. When a vehicle randomly arrives at the intersection, it may encounter any condition, where queue length is at a maximum, median, or not present. Based on the above discussion, the upstream effective length of the signalized intersection influence area can be calculated as the summation of stopping sight distance and average queue length. That is,

$$L_U = SSD + \overline{Q} \tag{4-4}$$

Where:

L_U: effective length of influence area upstream of signalized intersection, ft

SSD: Stopping Sight Distance, ft

$\overline{Q}$: average queue length, ft

In the above equation, *SSD* corresponds to the distance traveled during perception/reaction time plus the braking/deceleration distance, and $\overline{Q}$ corresponds to the stopping distance (i.e., distance over which queued vehicles are stopped).

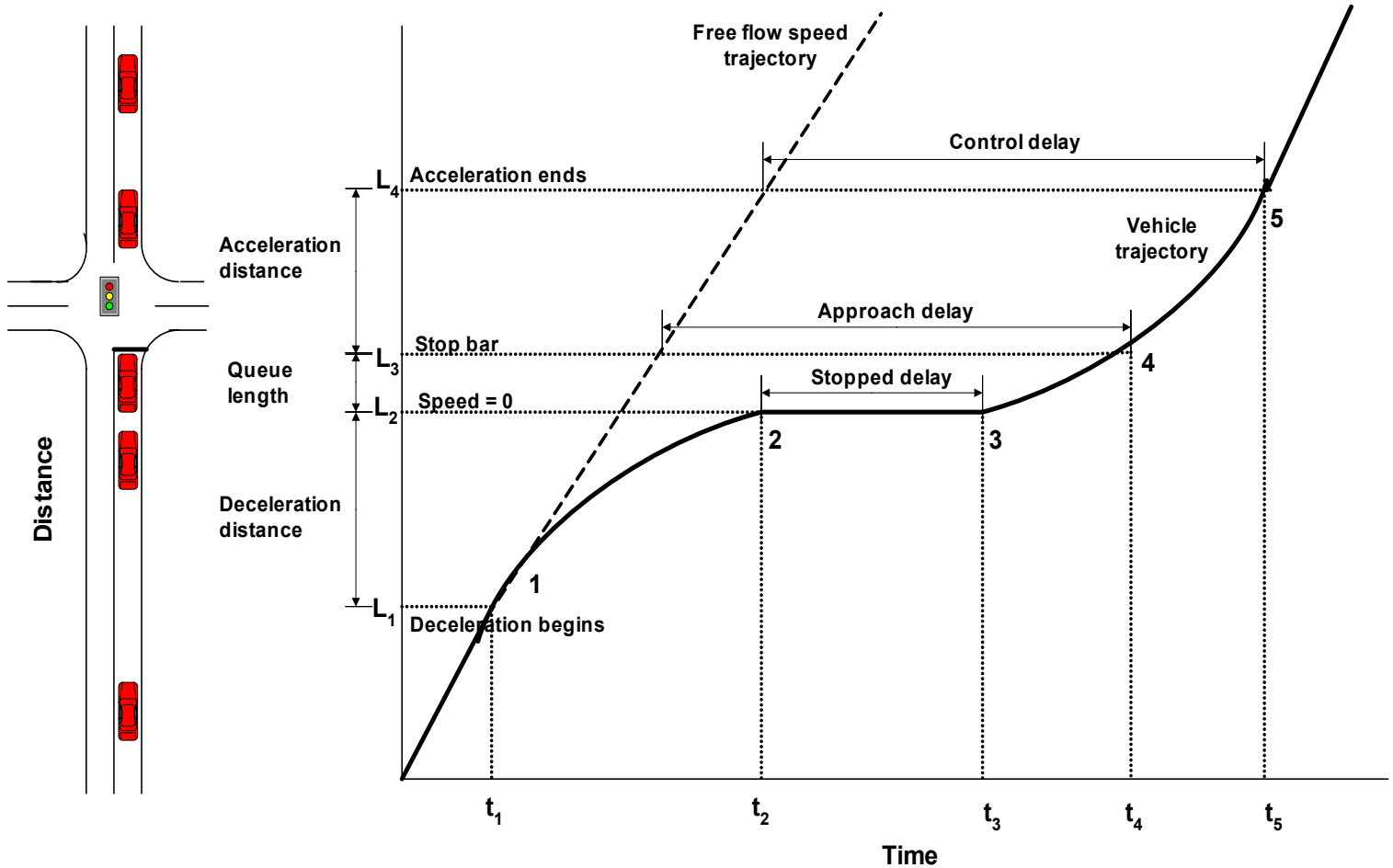

Figure 4-2. Schematic distance-time diagram at a signalized intersection

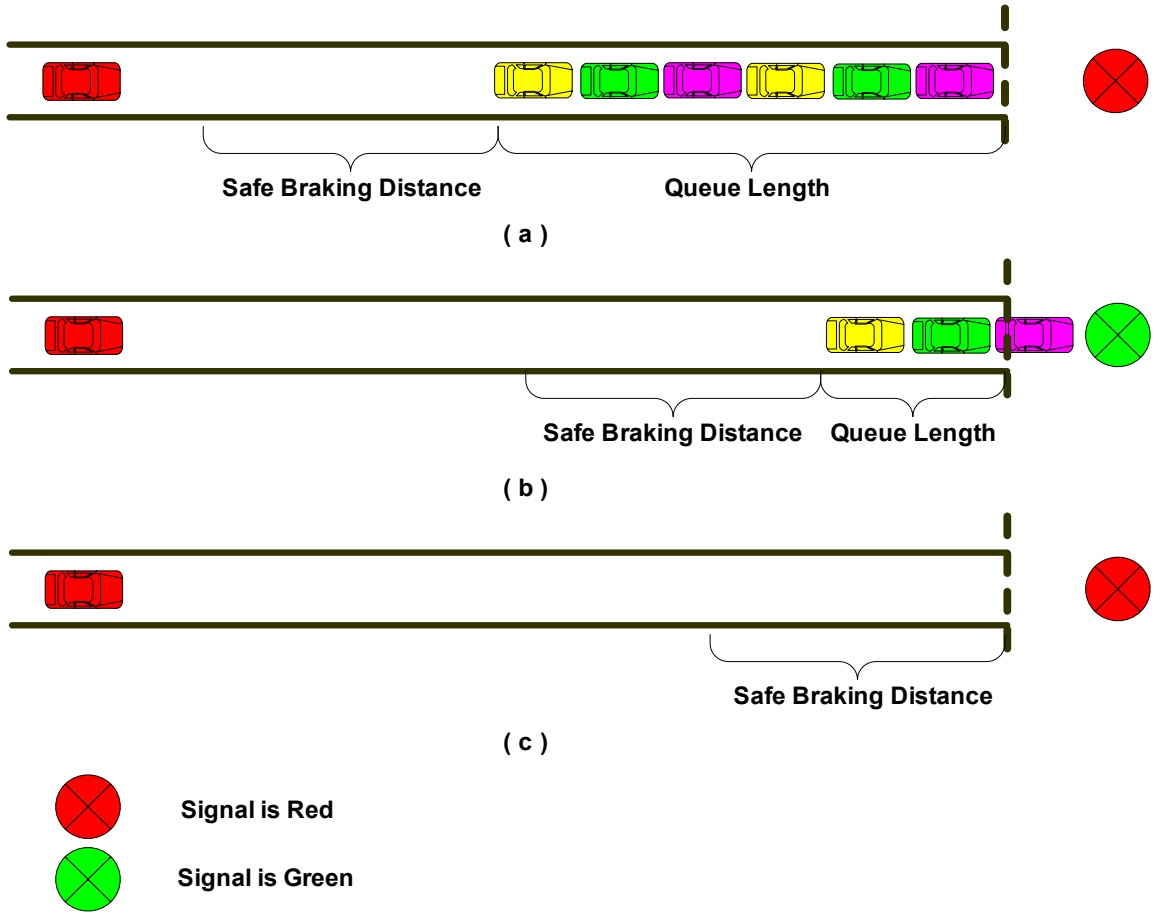

Figure 4-3. Queue length estimation

4.1.2.1 Determining stopping sight distance

The stopping sight distance can be calculated using Equation 4-5, as follows:

$$SSD = V_1 t_r + \frac{V_1^2}{2g\left(\left(\frac{a}{g}\right) \pm G\right)} \tag{4-5}$$

Where:

SSD: Stopping Sight Distance, ft

V_1: initial vehicle speed, ft/s

t_r: perception-reaction time, sec

a: deceleration rate, ft/s

g: gravitational constant, ft/s

G: roadway grade (+for uphill and – for downhill), percent/100

This equation is from AASHTO's "Green Book" [2]. In this equation, the perception-reaction time is taken as 2.5 seconds and a deceleration rate of 11.2 ft/s^2 (3.4 m/s^2) is assumed.

Perception-reaction time and initial vehicle speed are two important elements in calculating the stopping sight distance. The perception-reaction time is the time it takes to initiate the physical response, which includes the detection, identification, and decision elements involved in responding to a stimulus. The perception-reaction time used to calculate the stopping sight distance, when vehicles are near to the signalized intersection should be analyzed from the features of actions taken by drivers when near the signalized intersection.

- Vehicle deceleration when approaching an intersection is an expected event. Perception-reaction time varies depending on whether the event is expected or unexpected, with expected events logically requiring less time.
- Vehicle deceleration when approaching an intersection is a relatively simple task. Perception-reaction time varies with the complexity of the task. The simpler the task, the shorter the time required for a response.

Decelerating vehicles approaching the signalized intersection is an expected event, and it is also a fairly simple task. At the first part of perception-reaction time, vehicles still keep the initial speed; at the ending part of perception-reaction times, drivers begin taking actions to decelerate the vehicles. The AASHTO Green Book [2] suggests a perception-reaction time of 2.5 seconds, which is a design recommendation, accounting for unexpected events or obstacles in the roadway. Based on this recommended value and the characteristics of actions taken by drivers nearing a signalized intersection, the perception-reaction time is assumed to be in the range of 1 second (1 second is typically

used for yellow interval timing calculations). It is also assumed that the latter part of this perception-reaction time will consist of some vehicle deceleration as a driver will lift their foot off the accelerator in preparation for applying the vehicle's brakes.

Initial vehicle speed is another important element of stopping sight distance. The travel speed is generally inversely proportional to the traffic volumes. When the traffic volume is lower, vehicles approach the intersection at a higher speed; when the traffic volume is higher, vehicles approach the intersection at a relatively lower speed.

4.1.2.2 Determining average queue length

Appendix G of Chapter 16 in the HCM 2000 puts forward the concept of the average back-of-queue measure [1] at signalized intersections. In this model the back of queue is the number of vehicles that are queued depending on arrival patterns of vehicles and vehicles that do not clear the intersection during a given green phase. The average back of queue is used as the average queue length, and can be calculated using Equation 4-6:

$$Q = Q_1 + Q_2 \tag{4-6}$$

Where:

Q: maximum distance in vehicles over which queue extends from stop line on average signal cycle, veh

Q_1: first-term queued vehicles, veh, and

Q_2: second-term queued vehicles, veh

The first term, Q_1, represents the number of vehicles that arrive during the red phases and during the green phase until the queue has dissipated. The first term is calculated using equation 4-7.

$$Q_1 = PF_2 \frac{\frac{V_L C}{3600}\left(1 - \frac{g}{C}\right)}{1 - \left[\min(1.0, X_L)\frac{g}{C}\right]} \tag{4-7}$$

Where:

PF_2: adjustment factor for effects of progression

V_L: lane group flow rate per lane, veh/h

C: cycle length, sec

g: effective green time, sec, and

X_L: ratio of flow rate to capacity

Q_1 represents the number of vehicles that arrive during the red phases and during the green phase until the queue has dissipated. The adjustment factor for effects of progression is calculated by Equation 4-8.

$$PF_2 = \frac{\left(1 - R_P \frac{g}{C}\right)\left(1 - \frac{v_L}{s_L}\right)}{\left(1 - \frac{g}{C}\right)\left(1 - R_P \frac{v_L}{s_L}\right)} \tag{4-8}$$

Where:

PF_2: adjustment factor for effects of progression, veh

v_L: lane group flow rate per lane , veh/h

s_L: lane group saturation flow rate per lane, veh/h

C: cycle length, sec

g: effective green time, sec, and

R_p: platoon ratio

The second term, Q_2, is an incremental term associated with randomness of flow and overflow queues that may result because of temporary failures. This value can be an

approximate cycle overflow queue when there is no initial queue at the start of the analysis period. The second term of the average back of queue can be computed using Equation 4-9.

$$Q_2 = 0.25 c_L T \left[(X_L - 1) + \sqrt{(X_L - 1)^2 + \frac{8 k_B X_L}{c_L T} + \frac{16 k_B Q_{bL}}{(c_L T)^2}} \right] \tag{4-9}$$

Where:

c_L: lane group capacity per lane, veh/lane

T: length of analysis period, h

k_B: second-term adjustment factor related to early arrivals, and

Q_{bL} : initial queue at start of analysis period, veh

The second term adjustment factor related to early arrivals is calculated using Equation 4-10:

$$\begin{aligned} k_B &= 0.12 I \left(\frac{s_L g}{3600} \right)^{0.7} \qquad (\textit{pretimed signals}) \\ k_B &= 0.10 I \left(\frac{s_L g}{3600} \right)^{0.6} \qquad (\textit{actuated signals}) \end{aligned} \tag{4-10}$$

Where:

k_B: second-term adjustment factor related to early arrivals

s_L: lane group saturation flow rate per lane, veh/h

g: effective green time, sec

I: upstream filtering factor for platoon arrivals

4.1.2.3 Determining acceleration distance

Another component of the signal influence area, acceleration distance after the signalized intersection stop bar, can be determined using a linearly-decreasing

acceleration model. Continuing research [12] has shown that the linearly-decreasing acceleration model better represents both maximum vehicle acceleration capacities as well as actual motorist behavior. The linearly-decreasing acceleration model can be rewritten as a differential equation and integrated to derive the following relationships (treating a grade as being constant), as Equation 4-11 through 4-14. It should be noted that this is only part of the full derivation. The full derivation can be found in most traffic flow theory textbooks, for example [13].

$$\frac{\partial v}{\partial t} = \alpha - \beta v \pm Gg \tag{4-11}$$

$$v = \frac{(a \pm Gg)}{\beta} - \left(\left(\frac{a \pm Gg}{\beta} \right) - v_o \right) e^{-\beta t} \tag{4-12}$$

$$t = \frac{\beta d + v - v_0}{a \pm Gg} \tag{4-13}$$

$$d = \frac{(a \pm Gg)}{\beta} t - \left(\left(\frac{a \pm Gg}{\beta} \right) - v_o \right) \frac{(1 - e^{-\beta t})}{\beta} \tag{4-14}$$

Where:

v: speed at the end of the acceleration cycle, ft/s

v_0: speed at the beginning of the acceleration cycle, ft/s

α, β: acceleration model parameters, based on the design vehicle type

g: gravitational constant, ft/s

G: roadway grade (+for uphill and – for downhill), percent/100

t: time for vehicle to accelerate from beginning speed, v_0, to ending speed, v, sec

d: distance for vehicle to accelerate from beginning speed, v_0, to ending speed, v, ft

The equations presented above arising from the linearly-decreasing acceleration model are not quite as simple or as easy to apply as their counterparts based on constant acceleration rates, but they are processed readily by a computer.

Transportation and Traffic Engineering Handbook [14] also contained one of the most comprehensive summaries of previous research and field studies of maximum and normal acceleration and deceleration rates. Table 4-1 summarizes acceleration rates, distances traveled, and elapsed time for passenger vehicles on level terrain and under normal operating conditions.

Table 4-1. Normal acceleration rates, distance, and elapsed time

Initial Speed			Final Speed (mph)				
			15	30	40	50	60
0	Acceleration Rate (mph/s)		3.3	3.3	3.3	3.1	2.9
	Elapsed Time	(sec)	4.5	9.1	12.1	15.9	20.9
	Distance Traveled	(ft)	49	200	354	574	929
30	Acceleration Rate (mph/s)				3.3	2.9	2.5
	Elapsed Time	(sec)	---	---	3.0	6.8	11.8
	Distance Traveled	(ft)			154	374	729
40	Acceleration Rate (mph/s)					2.6	2.3
	Elapsed Time	(sec)	---	---	---	3.8	8.8
	Distance Traveled	(ft)				220	575
50	Acceleration Rate (mph/s)						2.0
	Elapsed Time	(sec)	---	---	---	---	5.0
	Distance Traveled	(ft)					355

Source: Reference 14.

After the SSD, back of queue, and acceleration length are determined, the length of the signalized intersection influence area can be calculated as the summation of the three components. That is,

$$L_S = SSD + \overline{Q} + L_A \tag{4-15}$$

Where:

L_S: length of a signalized intersection influence area, ft

L_A: acceleration length, ft

The components of the signal influence area, *SSD*, back of queue, and acceleration length, are consistent with those of control delay defined in the HCM 2000, which are deceleration delay, stop delay, and acceleration delay. A regression model was developed for the relationship of control delay calculated using the methodology presented in the HCM 2000, and the length of the signal influence area as the summation of *SSD*, average back of queue, and acceleration length. The results indicate that the assumption of a linear relationship is reasonable with an adjusted *R*-squared value of 0.895. The regression model summary is presented in Table 4-2.

Table 4-2. Regression model summary

1.SUMMARY OUTPUT:

Regression Statistics	
Multiple R	0.9469
R Square	0.8966
Adjusted R Square	0.8946
Standard Error	23.9621
Observations	54.0000

2. ANOVA:

	df	*SS*	*MS*	*F*
Regression	1.0000	258814.8753	258814.9	450.7529
Residual	52.0000	29857.5439	574.1835	
Total	53.0000	288672.4193		

	Coefficients	*Standard Error*	*t Stat*	*P-value*
Intercept	-114.408	8.80784	-12.9893	5.94E-18
X Variable 1	0.105078	0.00495	21.23094	2.79E-27
	Lower 95%	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
	-132.082	-96.7335	-132.082	-96.7335
	0.095146	0.115009	0.095146	0.115009

In this methodology, the length of a signal influence area is calculated as the summation of its components. In determining the length of each component, especially the *SSD* and back of queue, several significant factors are not reflected in the calculation formulas, such as the availability of a left-turn bay, the directional distribution of traffic flow, and the percentage of left-turn vehicles in the traffic flow. A new methodology is explored in the next section to fully take into account all major contributing factors, which can affect the length of a signalized intersection influence area.

4.1.3 Simulation and Regression Analysis

To fully account for all significant contributing factors affecting the length of a signalized intersection influence area, the method of regression analysis method is applied. Ideally, field data would largely be used to develop the regression model. However, in many cases, available study sites are either too limited and/or data cannot be collected without great complication. Additionally, it is often difficult to collect enough field data to provide a statistically valid sample size. In this study, the simulation method is applied to simulate the operations of a two-lane highway with a signalized intersection. The overall procedure consists of the following four major steps:

1. Select the potential contributing factors that are expected to have an impact on the effective length.
2. Select the appropriate simulation model.
3. Develop the simulation model to simulate the effects of contributing factors on effective length.
4. Develop the regression model.

These steps are discussed in detail in the following sections.

4.1.3.1 Contributing factor selection

The contributing factors considered include those that are expected to affect the effective length. Many factors can produce effects on the effective length of a signalized intersection. In the following section, traffic data, geometric data, and signal data are discussed, respectively.

- **Traffic Data**

  Traffic data include the hourly traffic volume, a Peak-Hour Factor (PHF), the proportion of trucks and recreational vehicles in the traffic stream, and the directional split (D-factor). Traffic flow rate can be used to represent the traffic conditions by making adjustments to the hourly traffic demand. These adjustments are the PHF, the heavy-vehicle adjustment factor, and the grade adjustment factor. The conversion can be made using Equation 4-16 [1]:

$$v_P = \frac{V}{PHF \times f_G \times f_{HV}} \tag{4-16}$$

Where:

v_p: passenger-car equivalent flow rate for peak 15-min period, pc/h

V: demand volume for the full peak hour, veh/h

PHF: peak-hour factor

f_G: grade adjustment factor

f_{HV}: heavy-vehicle adjustment factor

Traffic data also include the proportions of through vehicles, left-turn vehicles and right-turn vehicle in the traffic stream. The left-turning vehicles may have a negative effect on the flow of the through movements, particularly when higher percentage of left-turning vehicles may result in lane overflow or obstruction of the through

movements. The directional distribution of traffic flow is another important characteristic of traffic stream. On two-lane highways, lane changing and passing are possible only in the face of oncoming traffic in the opposite lane. There is a strong interaction between the directions of travel on a two-lane highway because passing opportunities are reduced and eventually eliminated as the opposing traffic volume increases. At an intersection, left-turn vehicles execute their turning maneuvers through the gaps of the opposing through traffic stream. When the opposing through traffic volume is high, left-turn vehicles have less opportunity to execute their turning movements.

- **Geometric Data**

  Geometric data include the two-lane highway geometry and intersection geometry. The basic geometric conditions of the two-lane highway and intersection are used to determine the effective length. The existence of exclusive left-or right-turn lanes, along with the storage lengths of such lanes should be noted, as these are important factors in determining the effective length.

- **Signal Data**

  The signalization conditions include control mode (i.e., pre-timed, semi-actuated, and fully-actuated), the phase plan, cycle length, green time, and clearance intervals. In this study, the simplest and most widely used form of signalization, the two-phase pre-timed signal, is used. All left-turn and right-turn movements are made on a permitted basis from shared or exclusive lanes. The cycle length and effective green time are selected as contributing factors to determine the effective length.

Based on the above discussion on traffic, geometric, and signalization conditions, contributing factors are selected for calibration of the upstream length of roadway affected by the signalized intersection. They are:

- peak volume
- D-factor
- percentage of left-turn and right-turn movements
- cycle length
- ratio of effective green time to cycle length
- availability of a left-turn bay

4.1.3.2 Simulation model selection

The next step is to select a simulation model to simulate traffic operations on a two-lane highway with a signalized intersection. As reviewed in Chapter 2, TWOPAS rural highway simulation model has the ability to simulate traffic operations on a conventional two-lane roadway. However, the model has no ability to simulate traffic turning on or off the highway at driveways and does not handle signalized intersections. Therefore, the TWOPAS simulation model is not appropriate to determine the effective length of the influence area upstream of the signalized intersection.

CORSIM, developed by the Federal Highway Administration, is the most widely used and accepted traffic simulation model in the U.S. It has the ability to simulate traffic operations on a two-lane roadway and includes detailed modeling of traffic signal operations. However, CORSIM cannot simulate passing maneuvers using the on-coming lane of traffic.

Before making a decision, TWOPAS was used to simulate the traffic operations on the basic two-lane roadway to study the relation of passing demand, passing capacity, the

percentage of passing zones, the advancing traffic volume, and the opposing traffic volume. CORSIM was used to determine the features that affected performance measure variation on the two-lane highway segment upstream the signalized intersection. These results are presented in Appendix A. After large quantities of simulations, the following conclusions can be drawn from the study:

- Although on the two-lane highway, passing operations can be performed using the opposite lane in the face of oncoming traffic, the percentage of vehicles undertaking passing maneuvers is rarely more than 6% of the traffic volume under different conditions of advancing traffic flow rate and opposing traffic flow rate.
- At the same advancing traffic volume level, the difference in average travel speed at the different opposing traffic volume levels is very small, less than 2%; the difference in the average travel speed between 100% no-passing zones and 0% no-passing zones is also small. As the advancing traffic flow rate increases, the difference decreases and gradually becomes negligible.
- The variance in travel speed due to a downstream signalized intersection is much larger than due to following a slower leading vehicle.

As vehicles approach a signal (i.e., within the influence area of the signalized intersection), the spacing between vehicles decreases, and following vehicles are unlikely to pass leading vehicles. Experience has shown that as drivers approach a signal, they generally will be more cautious; thus usually not undertaking passing maneuvers and possibly slowing down even if the signal indication is green. The roadway is also often marked with solid yellow dividing lines (i.e., no passing) in the vicinity of traffic signals. Under this assumption, it is feasible to use a program such as CORSIM to model vehicular operations on a two-lane roadway in the vicinity of a traffic signal, and determine the effective length of the signal influence area on the upstream two-lane highway segment. In addition, the CORSIM simulation model typically simulates the traffic system on a vehicle-by-vehicle basis by updating roadway position, speed, acceleration, and other state variables in discrete time steps. The ability to calibrate,

modify, and manipulate these parameters is a key characteristic of the CORSIM simulation model amenable for use to determine the effective length of the signal influence area.

4.1.3.3 Simulation model experimental design

A two-way, two-lane roadway network with an isolated fixed-time signalized intersection was simulated using CORSIM. It extended 3 miles, and the isolated intersection was located at the 1-mile point. The attributes of the simulated network were set to fulfill the basic conditions for a two-lane highway and signalized intersection according to the HCM 2000. These were defined as:

- Design speed greater than or equal to 60 mi/h
- Lane widths greater than or equal to 12 ft
- Clear shoulder wider than or equal to 6 ft
- Level terrain
- All passenger cars in traffic stream
- Two phase pre-timed signal

Two sets of CORSIM base road network were developed. One is the signalized intersection with a 250-foot left-turn bay; the other is the signalized intersection without a left-turn bay. Once the base road networks were developed, the values for the contributing variables were systematically changed to model different scenarios. The values for each contribution variable are displayed in Table 4-3 and Table 4-4. The different inputs resulted in a combination of 243 ($3\times3\times3\times3\times3=243$) simulation scenarios for the base network without a left-turn bay. Ten simulation runs were made for each scenario to account for the variability in stochastic micro-simulation program output. A

total of 2430 simulated runs were performed. The length of simulation time for each run was 15 minutes.

Table 4-3. Variable input values (with a left-turn bay)

Peak Volume (pc/h)	Cycle length (sec)	g/C	Percentage of left-turn and right-turn vehicles
400	60	0.55	5%
800	75	0.65	10%
1200	90	0.75	15%

Table 4-4. Variable input values (without a left-turn bay)

Peak Volume (pc/h)	D-Factor	Cycle length (sec)	g/C	Percentage of left-turn and right-turn vehicles
400	0.50	60	0.55	5%
700	0.55	75	0.65	8%
1100	0.60	90	0.75	11%

At an intersection, left-turn vehicles execute their turning maneuvers through the gaps of the opposing through traffic stream. When the opposing through traffic volume is lower, the left-turn vehicles can execute their turning movements. When the opposing through traffic volume is very high and left-turn vehicles cannot execute their turning movements, the left-turn vehicles can stay temporarily at the left-turn bay, and will not block the advancing through traffic behind them. So when developing the effective length model for the signalized intersection with a left-turn bay, the D-factor is not included as a contributing factor.

4.1.3.4 Regression model development

After simulation, average travel speeds at the interval of 0.025 miles along the two-lane roadway were obtained from the CORSIM output file. Figure 4-4 illustrates the variations of average travel speed along the two-lane highway with an isolated signalized intersection.

Based on the variation of average travel speed, the effective length of the signalized intersection on the upstream two-lane highway segment can be measured from the bifurcation point, at which point vehicles begin decelerating to the stop line of the signalized intersection, such as the section AO in Figure 4-4. After extracting the needed data, regression analysis was performed to establish the model of the upstream effective length with contributing factors.

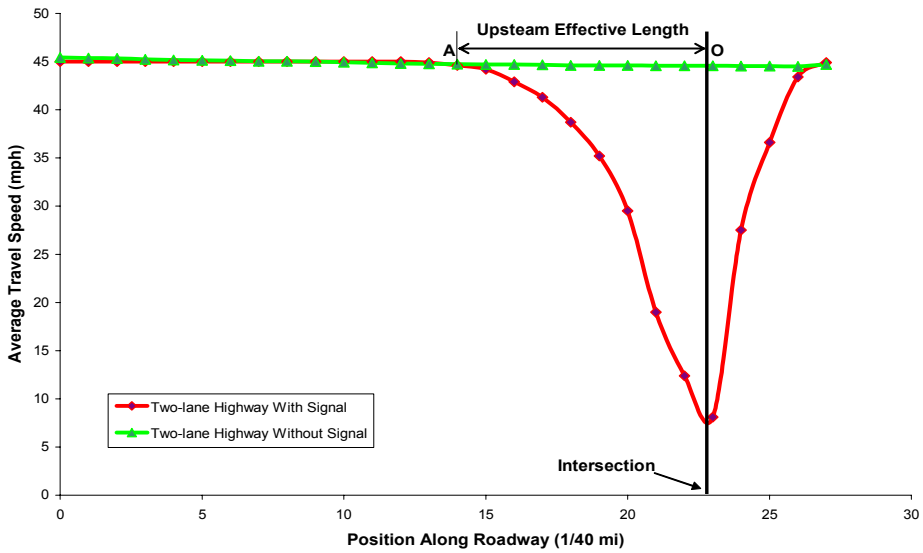

Figure 4-4. Average travel speed along the two-lane highway with signal

The regression model for the upstream effective length of a signalized intersection with a left-turn bay is developed as follows:

$$\begin{aligned} Len_{eff\_up} &= 43.2463 + 4.2688 \times (V/100)^2 + 5.2178 \times Cycle \\ &\quad - 57.3041 \times (V/100) \times \%LT - 5.2444 \times Cycle \times g\_C \end{aligned} \tag{4-17}$$

Where:

Len_{eff_up}: upstream effective length of a signalized intersection, ft

V: traffic flow rate, veh/h

$Cycle$: cycle length, sec

g_C: ratio of effective green time to the cycle length, and

%LT: percentage of left-turn vehicles in the directional traffic flow

The statistical results for the effective length model for the signalized intersection with the left-turn bay (250 ft) are presented in Table 4-5. For a significance level of 0.05 (or 95% confidence), α =0.05, and the degrees of freedom, n - (k + 1) = 30 – 5 = 25 df. The critical t value obtained from *t*-Distribution Table is $t_{0.05}$ = 1.708. All the t-stat absolute values are greater than 1.711, so all the variables used in the chosen model are useful.

Table 4-5. Regression model (with a left-turn bay)

R^2 = 0.95743; Adj R^2 = 0.95519		
	Coefficient	t-stat
Intercept	43.2463	4.84765
V/100(Q)	4.2668	29.38653
Cycle (L)	5.2178	11.41796
V/100(L) × %LT(L)	-57.3041	-3.05196
Cycle (L) × gCRatio (L)	-5.2444	-9.45781

(L) – Linear; (Q) – Quadratic

The regression model for the upstream effective length of a signalized intersection without a left-turn bay is developed as follows:

$$\begin{aligned} Len_{eff\_up} &= 3074.49 + 5.89 \times (V/100)^2 - 440.00 \times DFactor \\ &+ 1.69 \times Cycle - 7336.59 \times g\_C + 4758.52 \times (g\_C)^2 \\ &+ 1171.01 \times (V/100) \times (\%LT)^2 \end{aligned} \quad (4\text{-}18)$$

Where:

Len_{eff_up}: upstream effective length of a signalized intersection, ft

V: traffic flow rate, veh/h

DFactor: percentage of traffic traveling in the peak direction

Cycle: cycle length, sec

g_C: the ratio of effective green time to the cycle length, and

%LT: the percentage of left-turn vehicles in the directional traffic flow

The statistical results for the effective length model for the signalized intersection without a left-turn bay are presented in Table 4-6. For a significance level of 0.05 (or 95% confidence), α =0.05, and the degrees of freedom, n - (k + 1) = 30 – 5 = 24 df. The critical t value obtained from *t*-Distribution Table is $t_{0.05}$ = 1.711. All the t-stat absolute values are greater than the value of 1.711, so all the variables used in the chosen model are useful.

Table 4-6. Regression model (without a left-turn bay)

R^2 = 0.77764; Adj R^2 = 0.77199		
	Coefficient	t-stat
Intercept	3074.49	3.97573
V/100(Q)	5.89	20.44190
Dfactor (L)	-440.00	-2.08639
Cycle (L)	1.69	2.40322
gCRatio (L)	-7336.59	-3.08700
gCRatio (Q)	4758.52	2.60546
V/100(L) × %LT(Q)	1171.01	3.98667

(L) – Linear; (Q) – Quadratic

When calculating the upstream effective length, Table 4-7 is used for the following conditions:

(1): The maximum g/C value for this regression model is 0.8.

(2): When the traffic flow rate is less than or equal to 300 veh/h

Table 4-7. Upstream effective length with low traffic volume (ft)

V (veh/h)	g/C			
	0.5	0.6	0.7	0.8
100	160	130	110	90
200	180	150	130	110
300	210	180	160	140

4.2 Effective Length of the Influence Area Downstream of the Signalized Intersection

After passing through the signalized intersection, the vehicle platoon will travel into the downstream two-lane highway. The platoon dispersion pattern is affected not only by the upstream signalized intersection, but also by the right-turn vehicles and left-turn vehicles from minor streets.

This section begins with the discussion of Entering Percent Following (EPF) in the TWOPAS model and headway distribution. Then the effect of the signalized intersection on the downstream two-lane highway segment is quantified through the parameter of EPF. Next, the methodology using TWOPAS simulation to determine the effective length of a signalized intersection on the downstream segment is presented. Finally CORSIM simulation is used to validate this methodology.

4.2.1 Entering Percent Following of TWOPAS

A study by Dixon et al. [3] concluded that the potential effect of a signalized intersection on the downstream two-lane highway operations was to modify the distribution of headways. The condition with no signalized intersection is represented by assuming randomly distributed headways for entering traffic. However, the signalized intersection in the upstream will modify the headway distribution of the traffic stream entering the two-lane highway.

The TWOPAS model is used to simulate the effects of a signalized intersection on the downstream two-lane highway. In TWOPAS, the distribution of headway is defined through the input variable, Entering Percent Following (EPF), which is the percentage of the total vehicles in the direction of travel that are following in platoons when they enter the road being analyzed. Figure 4-5 illustrates an interface of TWOPAS for inputting

traffic data. In this interface, EPF is identified in the text, '% Traf in Platoons' in Dixon et al.'s study, it was assumed that it was appropriate to represent the effects of a signalized intersection through the EPF parameter, the percentage of vehicles following immediately downstream of a signalized intersection. To analyze the potential effect of a signalized intersection on the downstream two-lane highway operation, the key point is how to accurately decide the EPF at the point immediately downstream of a signalized intersection.

Figure 4-5. TWOPAS traffic data input interface

4.2.2 Headway Distribution

The time headway distribution between vehicles is an important flow characteristic that affects the safety, level of service, driver behavior, and capacity of a transportation system. Previous research [15] has established that the shape of the time headway distribution varied considerably as the traffic flow rate increased. In Dixon et al.'s study, the negative exponential distribution is used to define the headway distribution for the different traffic flow levels. For example, for the basic two-lane highway without a

signalized intersection, the EPF parameter is calculated using a cumulative exponential distribution for headways less than or equal to 3.0 seconds, using Equation 4-19:

$$\% \, Platooned = 100(1 - e^{-q\frac{t}{3600}}) \tag{4-19}$$

Where:

q: hourly flow rate of traffic entering the two-lane highway, veh/h

t: headway criteria used to define when vehicles are following, (3.0 sec)

The simple negative exponential distribution could not completely capture the features of headway distribution. To accurately quantify the effect of an isolated signalized intersection on the downstream two-lane highway segment, the shifted negative exponential distribution and composite distribution are introduced into the Dixon et al. methodology to calculate the EPF parameter.

4.2.2.1 Shifted negative exponential distribution

Under very low volume conditions, all the vehicles may be thought of as traveling independent of one another. Any point in time is as likely to have a vehicle arriving as any other point in time. This situation will be classified as the random headway state. The negative exponential distribution can be used to define the time headway distribution for this condition. However, drivers typically maintain a minimum time headway for safety considerations, although their perception of the minimum safe headway is often too low. Thus, the shifted negative exponential distribution can better define the time headway distribution under very low volume conditions. The probability density function of the shifted negative exponential distribution is given by equation 4-20:

$$f(t) = \lambda e^{-\lambda(t-\alpha)} \tag{4-20}$$

Where:

f(t): probability density function,

α : user-selected parameter greater than or equal to zero that affects the shift of the

distribution, sec, and

λ : parameter that is a function of the mean time headway and α .

λ can be calculated as:

$$\lambda = \frac{1}{\bar{t} - \alpha} \tag{4-21}$$

The percentage of vehicles in platoon with the shifted negative exponential distribution can be calculated using Equation 4-22:

$$\begin{aligned} \%platoon &= 1 - P(h \geq t) \\ &= 1 - \int_t^{\infty} \left[\lambda e^{-\lambda(t-\alpha)} \right] dt \\ &= 1\text{-}e^{-(t-\alpha)/(\bar{t}-\alpha)} \end{aligned} \tag{4-22}$$

4.2.2.2 Composite distribution

As the traffic flow rate increases, there is increasing interaction between vehicles. Gerlough et al. [16] proposed that the traffic flow consisted of two classes of vehicles: constrained vehicles and free-moving vehicles. According to May [15], the random headway state (Negative exponential distribution) is best suited for very low flow conditions, while the nearly-constant headway state (Normal distribution) is best suited for very high flow conditions. The intermediate headway state lies between the two boundary conditions of the random- and constant-headway states. The composite model is a better alternative to represent the headway distribution as the traffic flow level increases. The composite model approach utilizes the combination of a normal headway distribution for these constrained cars that are in the car-following or platoon mode and a shifted negative exponential distribution for those free-moving vehicles. The composite

distribution represents the time headway distribution well when the traffic flow rate is higher. The percentage of vehicles in platoon with the composite distribution can be calculated using Equation 4-23.

$$\begin{aligned} \%Platooned &= 1 - P(h \geq t) \\ &= 1 - \left[P_{NP} e^{-(t-\alpha)/(\bar{t}_{NP}-\alpha)} + P_P \int_t^{\infty} \frac{1}{s\sqrt{2\pi}} e^{-\frac{1}{2}(\frac{t-\bar{t}_P}{s})^2} dt \right] \end{aligned} \tag{4-23}$$

Where:

P_p: proportion of vehicles in platoon, %

P_{NP}: proportion of vehicles not in platoon, %

$\bar{t}_p$: mean headway of the vehicles in platoon, sec

$\bar{t}_{Np}$: mean headway of the vehicles not in platoon, sec

α : the minimum time headway for vehicles not in platoon, sec

s: standard deviation of normal distribution

t: time headway being investigated, sec

$\bar{t}$: mean headway, sec/veh

In the composite distribution, there are four independent parameters that need to be specified: mean and standard deviation of the normal distribution, the proportion of vehicles in platoon, and the minimum time headway for vehicles not in platoon. Numerous calculations and sensitivity analyses of a matrix of the four independent parameters need to be conducted to find the "best" composite model distribution for each traffic flow level. An example is given here to show how to find an appropriate composite distribution for the traffic flow of 1636 veh/h. Detailed calculations of the theoretical time headway for this traffic flow level are shown in Table 4-8. The

theoretical shifted negative exponential headway distribution, normal headway distribution, composite headway distribution, and the measured time headway distribution are presented graphically in Figure 4-6. The Chi-Squared test is used to access statistically how closely the measured distribution is similar to the theoretical composite distribution. An example is given to compare the measured time headway distribution for the traffic flow level of 1636 veh/h with a composite distribution. The Chi-Squared test calculations are shown in Table 4-9. The individual Chi-Squared contributions are summed, and the calculated Chi-Squared value is found to be 13.94. The number of degrees of freedom is determined to be 10 based on 15 time intervals and 4 parameters required for the composite distribution. Assuming a 0.05 significance level the reference Chi-Squared value is determined to be 18.30. Since the calculated Chi-Squared value is less than the reference Chi-Squared value, the hypothesis is not rejected and the conclusion is that there is no evidence of a statistical difference between the two distributions.

Although the composite distribution is the combination of a normal headway distribution and a shifted negative exponential distribution, when the traffic flow rate is lower, a larger difference occurs between the composite distribution and the measured distribution. So in this study, the shifted negative exponential distribution and composite distribution are used together to mathematically describe time headway distribution, including boundary conditions of random headway state and nearly-constant headway state, and the intermediate headway state. Detailed calculations of the theoretical time headway for all four traffic flow levels are presented in tabular form and figure in Appendix B.

Table 4-8. Composite headway distribution calculation (volume = 1500-1740 veh/h)

	Vehicles Not in Platoons						Platoon Vehicles				Composite Distribution	
t	$t-\alpha$	$\bar{t}-\alpha$	$\bar{t}-\alpha$	$e^{-\frac{(t-\alpha)}{(\bar{t}-\alpha)}}$	100%	P	z	$P(t<z)$	100%	1-P	(Prob.)	(Freq.)
0.0	0.0	1.5	0.0000	----	----	0.0000	-3.000	0.0013	0.0215	0.0147	0.0147	24
0.5	0.0	1.5	0.0000	----	----	0.0000	-2.000	0.0228	0.1359	0.0927	0.0927	152
1.0	0.0	1.5	0.0000	----	----	0.0000	-1.000	0.1587	0.3413	0.2327	0.2327	381
1.5	0.0	1.5	0.0000	1.0000	0	0.0000	0.0000	0.5	0.3413	0.2327	0.2327	381
2.0	0.0	1.5	0.0000	1.0000	0.1813	0.0577	1.0000	0.8413	0.1359	0.0927	0.1503	246
2.5	0.3	1.5	0.2000	0.8187	0.2321	0.0738	2.0000	0.9772	0.0215	0.0147	0.0885	145
3.0	0.8	1.5	0.5333	0.5866	0.1663	0.0529	3.0000	0.9987	0.0013	0.0009	0.0538	88
3.5	1.3	1.5	0.8667	0.4204	0.1192	0.0379	4.0000	1			0.0379	62
4.0	1.8	1.5	1.2000	0.3012	0.0854	0.0272					0.0272	45
4.5	2.3	1.5	1.5333	0.2158	0.0612	0.0195					0.0195	32
5.0	2.8	1.5	1.8667	0.1546	0.0438	0.0139					0.0139	23
5.5	3.3	1.5	2.2000	0.1108	0.0314	0.0100					0.0100	16
6.0	3.8	1.5	2.5333	0.0794	0.0225	0.0072					0.0072	12
6.5	4.3	1.5	2.8667	0.0569	0.0161	0.0051					0.0051	8
7.0	4.8	1.5	3.2000	0.0408	0.0116	0.0037					0.0037	6
7.5	5.3	1.5	3.5333	0.0292	0.0083	0.0026					0.0026	4
8.0	5.8	1.5	3.8667	0.0209	0.0059	0.0019					0.0019	3
8.5	6.3	1.5	4.2000	0.0150	0.0043	0.0014					0.0014	2
9.0	6.8	1.5	4.5333	0.0107	0.003	0.0010					0.0010	2
9.5	7.3	1.5	4.8667	0.0077	0.0077	0.0024					0.0024	4
											1.0000	1636

$\bar{t}_{NP} = 3.7\,\text{sec}, \alpha = 2.2\,\text{sec}, s_{NP} = 1.5\,\text{sec}, P_{NP} = 0.3182, E_P = 1.5\,\text{sec}, s_p = 0.5\,\text{sec}, P_p = 0.6818$

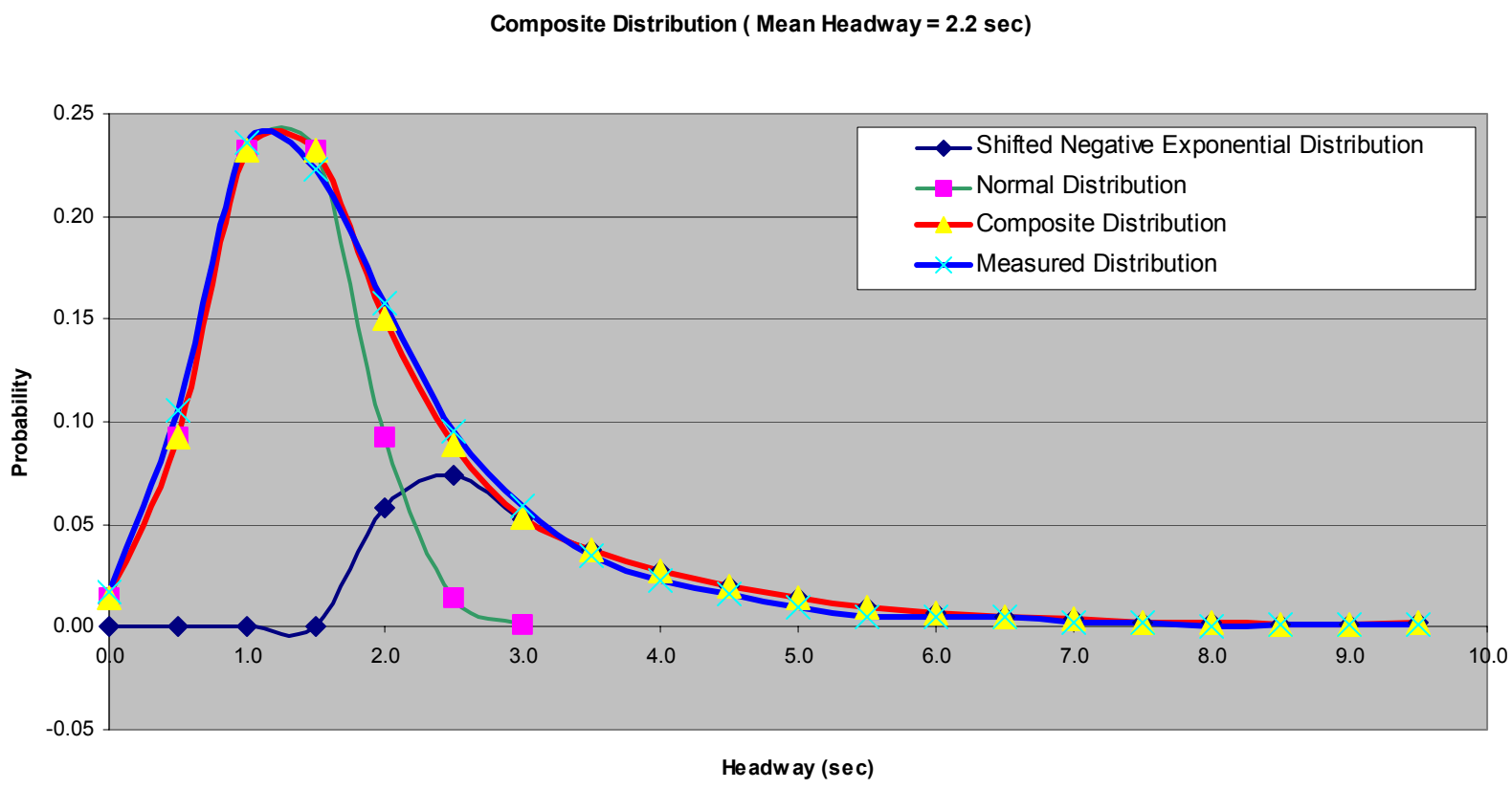

Figure 4-6. Composite time headway distribution

Table 4-9. Chi-Squared test calculation

Time Headway Group	f_0	f_t	$f_0 - f_t$	$(f_0 - f_t)^2$	$\frac{(f_0 - f_t)^2}{f_t}$
0.0 - 0.5	22	19	3	10	0.4934
0.5 – 1.0	140	122	18	310	2.5355
1.0 - 1.5	312	307	4	19	0.0616
1.5 – 2.0	294	307	-13	164	0.5342
2.0 – 2.5	209	198	10	102	0.5158
2.5 – 3.0	125	117	9	74	0.6293
3.0 - 3.5	78	71	7	47	0.6638
3.5 – 4.0	46	50	-4	15	0.2955
4.0 - 4.5	30	36	-5	30	0.8433
4.5 – 5.0	21	26	-5	21	0.8143
5.0 - 5.5	13	18	-5	27	1.4748
5.5 – 6.0	7	13	-7	43	3.2939
6.0 - 6.5	7	9	-3	8	0.8607
6.5 – 7.0	7	7	0	0	0.0044
7.0 – 7.5	3	5	-2	5	
7.5 – 8.0	3	3	-1	1	1.9678
8.0 – 8.5	0	2	-2	6	
8.5 – 9.0	2	2	0	0	
9.0 - 9.5	1	1	0	0	0.4183
> 9.5	3	1	-2	4	
	1320	1320	0		$\chi^2_{CALC} = 15.41$

$n = (I - 1 - p) = (16 - 1) - 4 = 11$. Significance Level = 0.05, $\chi^2_{ref} = 19.70$.

$\chi^2_{CALC} < \chi^2_{ref}$, $15.41 < 19.70$; Therefore, do not reject null hypothesis

4.2.3 Determining Entering Percent Following

Dixon et al. [3] concluded that it was appropriate to represent the effects of a signalized intersection on the downstream two-lane highway operations through the EPF parameters, as long as the percent following immediately downstream of a signalized intersection can be determined. In this study, the methodology for determining Entering Percent Following is based on Dixon et al.'s methodology. The main difference from their methodology is the application of distributions for time headway. As discussed in

Section 4.2.2, the shifted negative exponential distribution and composite distribution are introduced into this methodology. Estimation of the percentage of entering traffic following is based on a flow profile immediately downstream of the signalized intersection. A flow profile immediately downstream of the signalized intersection at location "A" is shown in the Figure 4-7. The "A" denotes a location immediately downstream of the signalized intersection. As shown in Figure 4-7, there are three movements that contribute to the flow profile:

- Movement 1: Primary contributing movement. They are through movements from the upstream major street;
- Movement 2: Secondary contributing movement. They are right-turn movements from the minor street; and
- Movement 3: Secondary contributing movement. They are left-turn movements from the minor street.

As shown in the Figure 4-7, the total cycle-length is divided into three states. The above three movements are charged through the three states. They are:

- First state: Discharged from the through movement queue during the first phase
- Second state: Discharge from the through movement without a queue plus any right-turn on red executed during the first phase
- Third state: Discharge from the right and left turn movements during the second phase

Entering percent following at location A can be estimated using equation 4-24:

$$EPF_a = \frac{VF_a}{V_a}$$
$$VF_a = \sum_i VF_i \tag{4-24}$$

Where:

EPF_a: percent of vehicles following at Point A, immediately downstream of a

signalized intersection,

VF_a: total number of vehicles following per cycle at location A, veh

VF_i: total number of vehicles following per cycle from movement i, veh, and

V_a: total number of vehicles per cycle at location A, veh

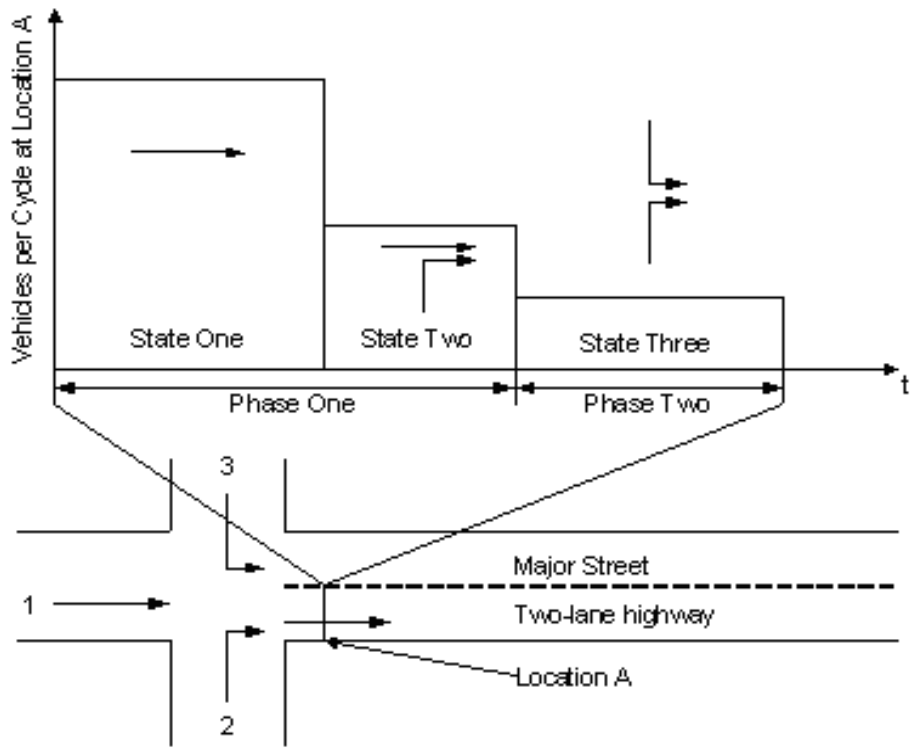

Figure 4-7. Two-lane highway traffic flow downstream of a signalized intersection

Source: Dixon, Michael P., Michael Kyte, and Satya Sai Kumar Sarepali. Effects of Upstream Signalized Intersections on Two-Lane Highway Operations, Transportation Research Board, Washington D.C., 2004

To determine the EPF_a, the key point is to decide on the value of the denominator, V_a, and numerator, VF_a. Because V_a is the summation of the cycle-by-cycle volumes from movements 1, 2, and 3, it can be determined if volumes for movements 1, 2 and 3 and the cycle length are known. This leaves the estimation of VF_a, the number of vehicles

following at location A, which can be estimated by each movement. The detailed procedure for calculating EPF is presented in Appendix C.

Table 4-10 summarizes the values of EPF under different traffic conditions and signal timing plans. Due to the complexity of the calculations, a Visual Basic program was developed to calculate the EPF values. The Visual Basic codes programmed for this research are presented in Appendix D.

Table 4-10. Entering Percent Following

g/C=0.6					
		Cycle Length (sec)			Without signal
		60	90	120	
Traffic Volume	220	0.2069	0.2978	0.3433	0.1458
	440	0.4515	0.4969	0.5197	0.2778
	660	0.5988	0.6291	0.6442	0.3962
	880	0.7213	0.7441	0.7554	0.5015
	1100	0.8435	0.8617	0.8708	0.5941

g/C=0.7					
		Cycle Length (sec)			Without signal
		60	90	120	
Traffic Volume	220	0.1192	0.2101	0.2556	0.1458
	440	0.3662	0.4117	0.4344	0.2778
	660	0.5149	0.5452	0.5803	0.3962
	880	0.6372	0.6599	0.6713	0.5015
	1100	0.7789	0.7980	0.8071	0.5941
	1320	0.8657	0.8808	0.8884	0.6745

g/C=0.8					
		Cycle Length (sec)			Without signal
		60	90	120	
Traffic Volume	220	0.0324	0.1233	0.1687	0.1458
	440	0.2826	0.3281	0.3508	0.2778
	660	0.4334	0.4637	0.4789	0.3962
	880	0.5582	0.5789	0.5903	0.5015
	1100	0.7184	0.7366	0.7475	0.5941
	1320	0.8030	0.8182	0.8258	0.6745
	1540	0.8915	0.9045	0.9110	0.7434

After estimating EPF_a, the percent following immediately downstream of a signalized intersection is determined and then input into the TWOPAS model. A series of runs of TWOPAS model are performed to determine the effective length of a signalized intersection on the downstream two-lane segment, and performance measures (average travel speed and percent time-spent-following) of the downstream two-lane highway segment. The detailed procedure is presented in the next section.

4.2.4 Effective Length of a Signalized Intersection on the Downstream Segment

In this section, the input variable, EPF will be entered into the TWOPAS simulation model to illustrate the potential downstream effects of a signalized intersection on the two-lane highway operations. Two types of TWOPAS run were made with 10 replicate runs for each. The two conditions are as follows:

- A traffic stream of 600 veh/h travels along a two-lane highway with no signalized intersection.
- A traffic stream composed of through vehicles from the main street, left-turn vehicles and right-turn vehicles from the minor streets disperses from the signalized intersection into the downstream two-lane highway. The volumes of these three movements are 400 veh/h, 100 veh/h and 100 veh/h, respectively.

A value of EPF = 36.83% was used for the condition with no signalized intersection, assuming a volume of 600 veh/h. A higher value of EPF = 48.04% is used to represent the situation where a signalized intersection is present and modifying the headway distribution of a traffic stream consisting of 400 through vehicles from the main street, 100 left-turn vehicles from one minor street, and 100 right-turn vehicles from the cross-street entering the downstream two-lane highway segment.

A 5-mile section of two-lane highway was simulated using TWOPAS with the following conditions:

- Through movement saturation flow rate, s_1 = 1800 veh/h

- *PHF* = 1.0
- 100% passenger cars
- 0% no-passing zones
- 0% reduced speed zone
- Level terrain
- Lane width = 12 ft
- 50/50 directional split
- Two-phase timing plan
- Inter-green time is equal to the lost time per phase
- Desired speed and speed standard deviation using the recommended default values presented in Table 4-11.

Table 4-11. Recommended default values for desired speed by vehicle type

	Passenger Car	Recreational Vehicle	Truck
Mean Desired Speed (mi/h)	61.5	59.5	59.5
Standard Deviation (mi/h)	5.0	4.0	3.5

When establishing the TWOPAS model, a series of data collection stations were set along the two-lane highway at 100 ft intervals. After simulation, the average travel speed at each data collection station was obtained from the TWOPAS output file. $ATS_{sig,i}$ denotes the average travel speed at the ith data collection station on the two-lane highway with a signalized intersection. $ATS_{wo\_sig,i}$ denotes the average travel speed at the ith data collection station on the two-lane highway without a signalized intersection. Figure 4-8 shows the difference between $ATS_{sig,i}$ and $ATS_{wo\_sig,i}$ along the two-lane highway downstream of the signalized intersection.

Point 'O' in Figure 4-8 is the location of the stop line of the signalized intersection. Point 'C' is the end of the two-lane highway downstream segment. The black thick line is the trendline of the difference between $ATS_{sig,i}$ and $ATS_{wo\_sig,i}$ along the roadway. As observed from Figure 4-8, when the traffic stream travels into the downstream segment

from Point 'O', the difference between $ATS_{sig,i}$ and $ATS_{wo\_sig,i}$ becomes larger until it reaches the peak point (Point 'A'). In the section ('OA'), vehicles in the tight platoon have no opportunity to pass the slow leading vehicles and travel at the desired speed, mainly due to the initially tight platoon formed from the upstream signalized intersection. As traffic moves downstream, the initially tight platoon formed from the departing queue tends to disperse the farther downstream it travels. The platoon tends to spread out – a few moving ahead and some dropping back. After the peak point, it can be observed that the difference between $ATS_{sig,i}$ and $ATS_{wo\_sig,i}$ drops dramatically. It is mainly because the initially tight platoon formed from the signalized intersection has spread out and its effect is becoming smaller and smaller. Point 'B' can be considered as the transition point, at which point the decreasing slope changes from steep to fairly level. After Point 'B', the difference between $ATS_{sig,i}$ and $ATS_{wo\_sig,i}$ becomes negligible, near to 0.2 miles per hour. It still keeps dropping, but the decreasing rate becomes extremely small. After a certain distance, the platoon has spread out, and the impacts produced by the upstream signal on the platoon dispersion gradually disappear.

Based on the above discussion, the effective length of a signalized intersection on the downstream two-lane highway segment can be determined from the stop line of this signalized intersection to the point, at which the difference between $ATS_{sig,i}$ and $ATS_{wo\_sig,i}$ becomes negligible, and its decreasing rate becomes smaller.

To estimate the downstream effective length of a signalized intersection, a key issue is to measure the variation of the difference between ATS_{SIG} and ATS_{W_SIG}. Ideally, the downstream effective length of a signalized intersection can be measured from the on-site observation. However, available study sites are often too limited.

Furthermore, it is very difficult to get the information about the difference in average travel speed between the condition with signalized intersection and without signalized intersection at the same location. In this study, TWOPAS is used to simulate traffic operations on the two-lane highway under different traffic conditions, and it is assumed that it is appropriate to represent the effects of a signalized intersection through the entering percent following parameter.

The average travel speed at the interval of 100 ft along the downstream two-lane highway can be obtained from the TWOPAS simulation output. Figure 4-9 illustrates the difference in the average travel speed between the condition of with a signalized intersection and with no signalized intersection. The traffic volume is 1100 veh/h. When the signalized intersection is present, the cycle length is 60 seconds, 90 seconds, and 120 seconds, respectively. The ratio of effective green time to the cycle length is 0.6, 0.7, and 0.8 respectively. Similar figures for the other levels of traffic volume are plotted and presented in Appendix E.

On observing these figures, the following conclusions can be drawn:

- The traffic volume is the decisive factor for the downstream effective length of a signalized intersection. The bigger the traffic volume into the downstream segment, the shorter the downstream effective length.
- Cycle length and the ratio of effective green time to cycle length also have some effect on the downstream effective length of the signalized intersection influence area, but it is much smaller.

Based on the above analysis, it was decided to develop a simplified model for the downstream effective length of a signalized intersection, that is, one that is just a function of directional traffic volume being served during the analysis period, as shown in Equation 4-25:

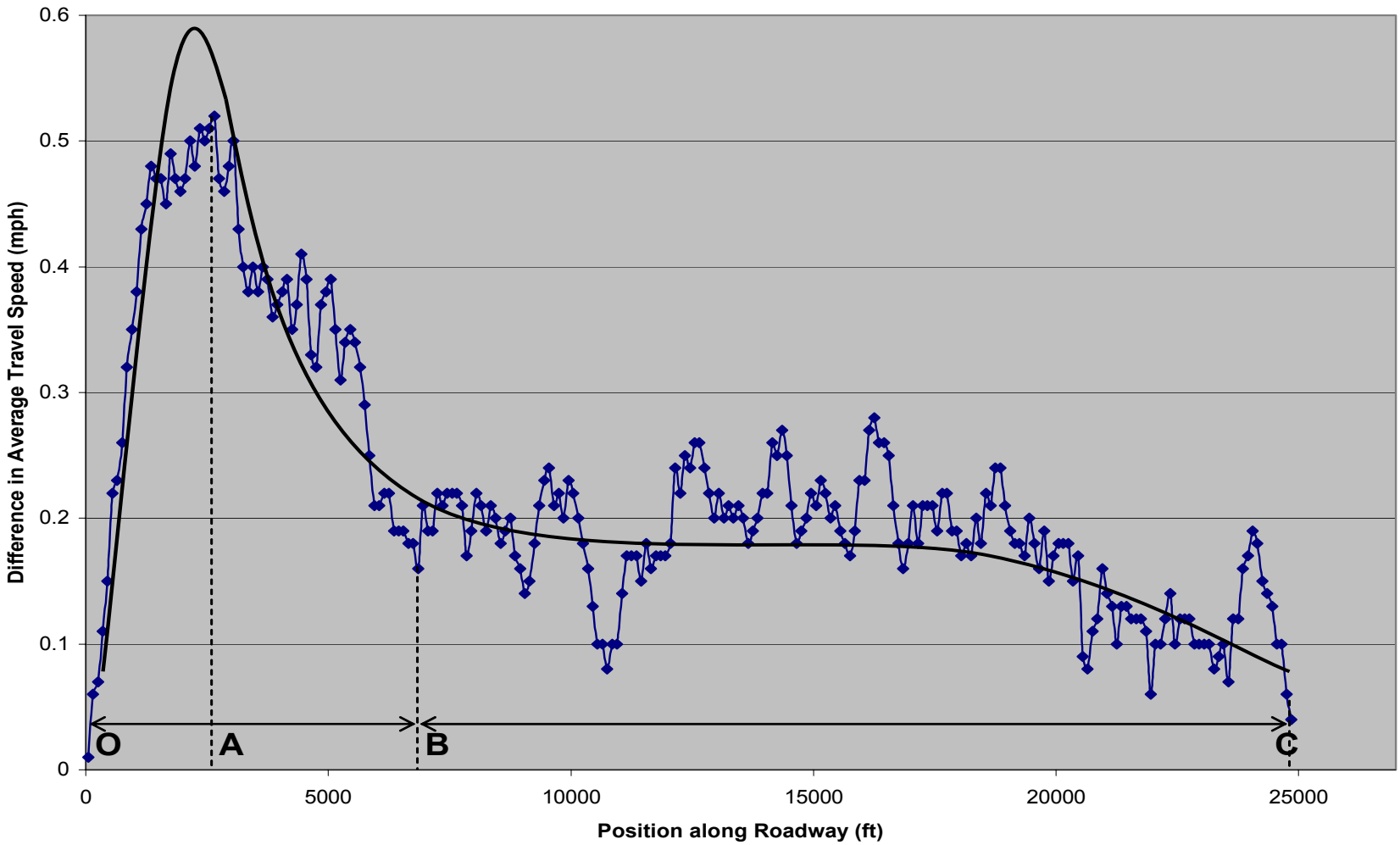

Figure 4-8. Difference in the average travel speed between with and without signalized intersection along the downstream two-lane highway

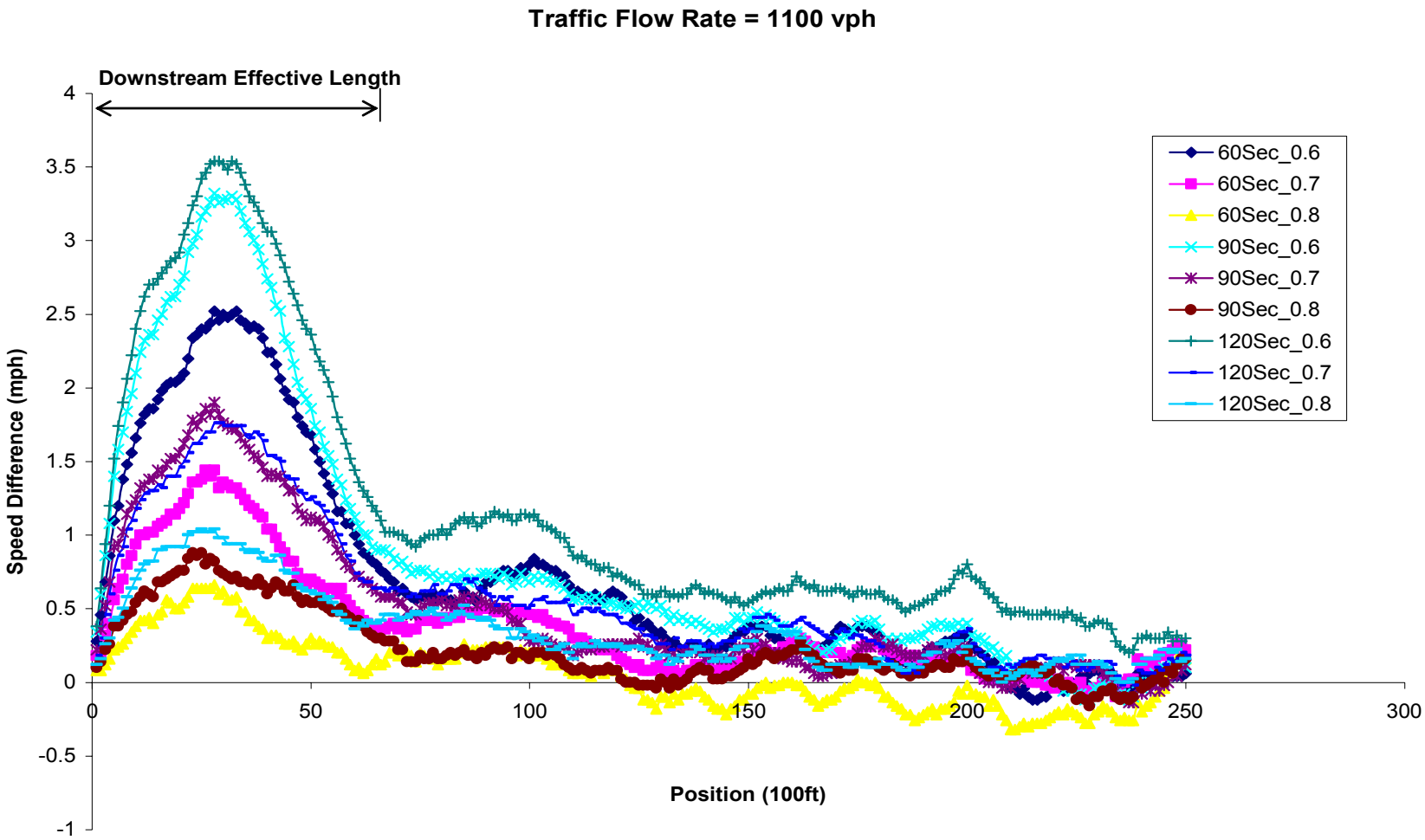

Figure 4-9. Difference in the average travel speed with traffic volume = 1100 veh/h

$$Len_{eff\_down} = 2.218584 - 0.122942 \times (V/100) \quad (4\text{-}25)$$

Where:

Len_{eff_down}: Downstream effective length of a signalized intersection, mi, and

V: Directional traffic flow rate, veh/h

Table 4-12. Regression model for the downstream effective length

$R^2 = 0.98645$; Adj. $R^2 = 0.98306$		
Variable	Coeff.	t-stat
Intercept	2.218584	38.8479
Volume/100	-0.122942	-17.0642

From the above model, it can be seen that the downstream effective length is inversely proportional to the traffic volume. As traffic volume increases, the downstream effective length decreases. When the traffic volume is low, the headway between vehicles is relatively large; thus, vehicles are generally not traveling in platoons and the average travel speed is at or very near to the free-flow speed. The presence of a controlled intersection introduces platooning in the traffic stream in the vicinity of the intersection. As the traffic stream discharges from upstream of the intersection into the downstream section beyond the intersection, it will take a relatively longer distance to return to the free-flow state from the platooned state. However, when the traffic flow rate is high, the headway between vehicles is much smaller; thus, a high percentage of vehicles are already in the platooned state and the average travel speed is lower than the free-flow speed. As the traffic stream discharges from upstream of the intersection into the downstream section beyond the intersection, it will take a relatively shorter distance to return to a similar platoon state from the signal queued state.

This method of determining the effective length of an isolated signalized intersection influence area on the downstream two-lane highway segment takes full advantage of TWOPAS's ability to simulate traffic operations on the two-lane highway. It also applies the parameter of EPF at the immediate downstream point of a signalized intersection to reflect the potential effect of a signalized intersection on the downstream two-lane highway operation. The evaluation for this methodology will be done in the next section.

4.2.5 Evaluation Based on CORSIM Simulation

TWOPAS is the only simulation software that is able to simulate the passing maneuver operation on the two-lane highway using the opposing lane, and was developed with U.S. data. In this study, TWOPAS was used to study the effects of the signalized intersection on the downstream segment with the reasonable Entering Percent Following input variables. In this section, a simulation approach is used to evaluate the methodology for determining the downstream effective length of a signalized intersection on the two-lane highway.

The CORSIM simulation model was selected as the traffic simulator. It can analyze a wide range of traffic, geometric, and control conditions and produces a relatively rich set of performance measures. CORSIM is not able to simulate the passing operation on the two-lane highway using the opposing lane, but this drawback will be considered in the results analysis.

A two-way, two-lane roadway network with an isolated fixed-time signalized intersection was simulated using CORSIM. It was a total of 5 miles in length, and the isolated intersection was located at the 0.5-mile point. Once the CORSIM base road network was developed, the values for the independent variables were systematically

changed to model different scenarios. After simulation, average travel speeds at the interval of 0.025 miles along the two-lane roadway are obtained from the CORSIM output file. The downstream effective length of a signalized intersection based on average travel speed can be determined from its variation trend. Figure 4-10 shows the average travel speed variation along the downstream two-lane highway segment of a signalized intersection and the average travel speed variation under the condition with no signalized intersection, assuming the traffic flow rate is 220 veh/h. The red line represents the average travel speed variation with no signalized intersection. The other lines represent the average travel speed variation with a signalized intersection, which have different cycle lengths and g/C (the ratio of green time to the cycle length). Similar figures for the other levels of traffic volume are plotted and presented in Appendix F.

As observed from this figure, after the signalized intersection, vehicles travel some distance to return to the speed at which they would have traveled under the condition of no signalized intersection. According to the average travel speed variation under the two conditions, the distance of OB in this figure is defined as the downstream effective length due to the influence of the signalized intersection. In addition, it can be observed that with the same traffic flow rate, there is not a large difference in the variation of average travel speed when the signalized intersections have different cycle lengths and g/C ratios. These phenomena are the same with the conclusion drawn from the methodology for determining the downstream effective length of a signalized influence area presented in Section 4.2.4. Similar figures are plotted for other levels of traffic volume. As observed from these figures, the downstream effective lengths from CORSIM simulation are presented in Table 4-13.

Table 4-13. Comparison of downstream effective length

Volume (veh/h)	Effective Downstream Length from CORSIM, (mi)	Effective Downstream Length Equation 4-26, (mi)	Difference (mi)
200	2.05	1.972	0.078
400	1.60	1.728	-0.128
600	1.35	1.480	-0.130
800	1.05	1.234	-0.184
1000	0.65	0.988	-0.338

As observed from Table 4-13, the downstream effective length of a signalized intersection from CORSIM simulation is also inversely proportional to the traffic flow rate. Through comparing the results obtained from the TWOPAS methodology with those from CORSIM simulation, it can be seen that when the traffic flow rate is low (such as 200 veh/h, 400 veh/h), the results obtained from the CORSIM simulation are very much in agreement with those obtained from the verified TWOPAS methodology. Although CORSIM has no ability to simulate the passing maneuver on the two-lane highway using the opposing lane, the low passing percentage under the low traffic demand on the two-lane highway weakens the effect of this drawback. When the traffic flow rate increase, the effective lengths obtained from the CORSIM simulation models are shorter than those obtained from the TWOPAS methodology. As the traffic flow rate increases, the traffic passing percentage also increases. The passing demand on the condition without any signalized intersection is larger than with a signalized intersection. It is CORSIM's inability of simulating passing maneuvers on the two-lane highway using the opposing lane that makes vehicle speeds drop quickly for the condition without any signalized intersection than for the condition with a signalized intersection. Therefore, the downstream effective lengths obtained from the CORSIM simulation model are shorter than these from TWOPAS when the traffic volume is higher. Based on the above comparison, it can be concluded that the downstream effective lengths obtained from TWOPAS essentially match those obtained from the CORSIM simulation.

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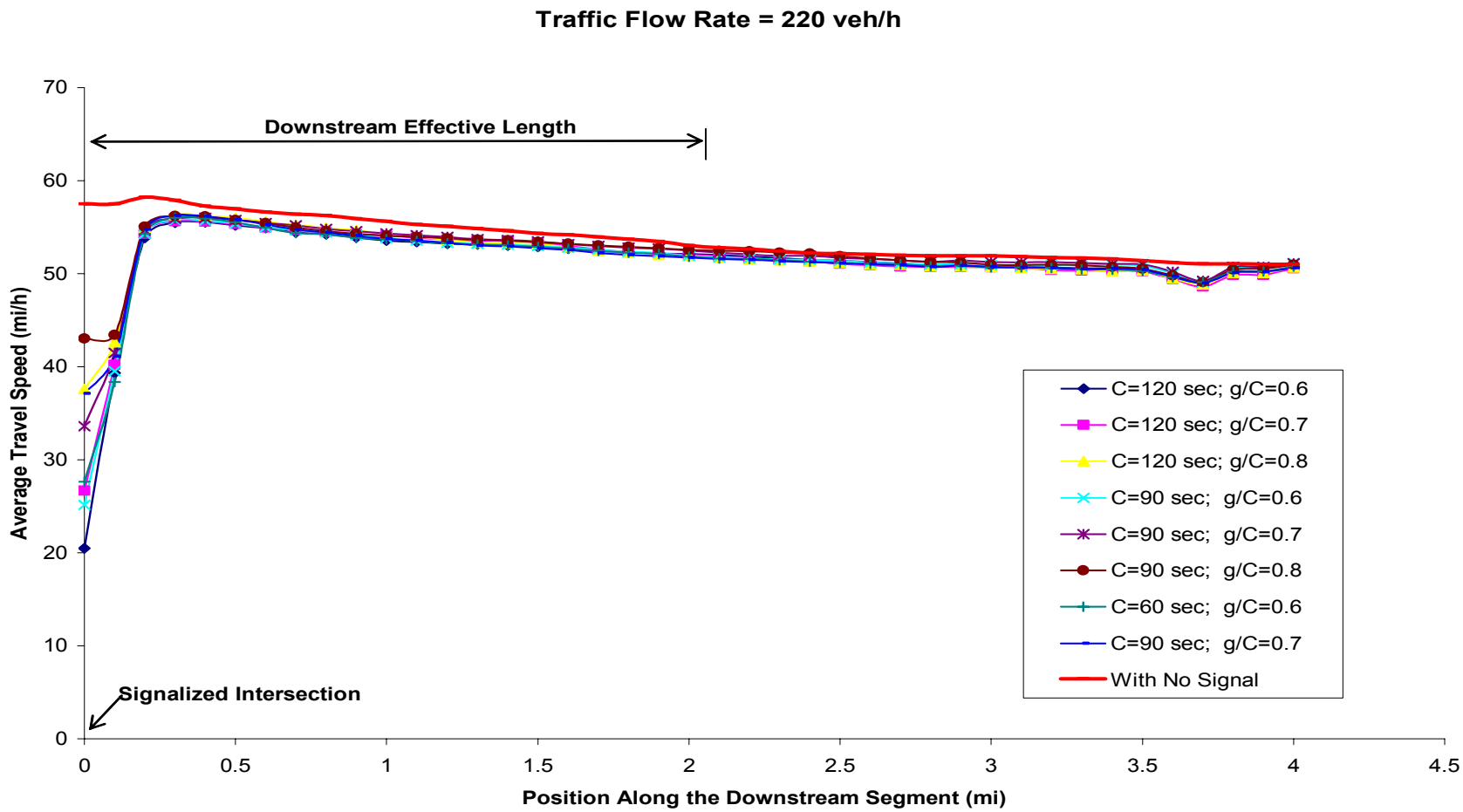


Figure 4-10. Average travel speed variation with traffic volume = 220 veh

CHAPTER 5
ESTIMATION OF SERVICE MEASURE VALUES

This chapter describes the methods for calculating the service measure values of each of these segment types. It begins with procedures for determining the free-flow speed on the two-lane highway segment. Next, it presents how to determine average travel speed on three kinds of two-lane highway segments: the basic two-lane highway segment that is unaffected by a signalized intersection, the downstream two-lane highway segment within the effects of the installed signalized intersection, and a two-lane highway with a passing lane. Next, the formulas for calculating control delay at the signalized intersection and the TWSC intersection are presented. Finally the recommend LOS table for the two-lane highway facility is provided.

5.1 Free-Flow Speed Estimation

Free-Flow Speed (*FFS*) is the average speed of the traffic stream when the traffic volume is sufficiently low so that drivers are not influenced by the presence of other vehicles and when intersection traffic control is not present or is sufficiently distant as to have no effect on speed choice [1].

In estimating expected operating conditions of a two-lane highway facility, the free-flow speed is a significant variable. The chosen service measure for this facility analysis methodology, percent time-delayed, is defined as the percentage of the travel time that a vehicle on a given roadway segment must travel at speeds less than their desired speed due to the inability to traffic control during some designed time interval.

There is actually no practical method by which to measure drivers' desired speed in the field. In practice, the desired speed, on aggregate, is usually considered to be the free-flow speed. In addition, free-flow speed is a necessary variable when calculating the average travel speed on the two-lane highway segment.

Two general methods are recommended in the HCM 2000 [1] to determine the *FFS* for a two-lane highway: field measurement, and estimation of free-flow speed.

5.1.1 Field Measurement of Free-Flow Speed

The Free-flow-speed of a two-lane highway can be determined directly from a speed study conducted in the field. The speed study should be conducted at a representative site within the study section. The best location to measure free-flow speed on the two-lane highway is mid-block and as far as possible from the nearest signalized or stop-controlled intersection. The measurement should be made under low flow conditions (less than 200 veh/h). The most appropriate section for performing a field study for the free-flow speed is the segment, which is not affected by the installed signalized intersection.

Free-flow speeds may be directly measured as follows:

- A representative speed sample of 100 or more vehicle should be obtained.
- Total two-way traffic flow should be 200 pc/h or less.
- All vehicle speeds should be observed during the study period, or a systematic sampling should be applied.
- When a two-direction analysis is considered, the speed sample should be selected from both directions of flow; when a one-direction analysis is considered, the speed sample should be selected only from the direction under study.

If field measurements must be made at a total flow level higher than 200 pc/h, the free-flow speed may be estimated as:

$$FFS = S_m + 0.00776(\frac{v_f}{f_{HV}}) \quad (5\text{-}1)$$

Where:

FFS: free-flow speed for the facility, mi/h

S_m: mean speed of the measure sample (where the total flow is greater than 200 veh/h), mi/h

V_f : observed flow rate for the period of the speed sample, veh/h

f_{HV} : heavy vehicle adjustment factor

5.1.2 Estimating Free-Flow Speeds

If field observation of free-flow speed is available, the free-flow speed on a two-lane rural highway may be estimated indirectly. Because the free-flow speed of a two-lane highway can range from 45 to 65 mi/h, this is a greater challenge on two-lane highways than on other types of uninterrupted-flow facilities. The free-flow speed can be estimated by applying the adjustments to the base free-flow speed (*BFFS*) using Equation 5-2 [1]:

$$FFS = BFFS - f_{LS} - f_A \quad (5\text{-}2)$$

Where:

FFS: estimated free-flow speed for the facility, mi/h

BFFS: base free-flow speed for the facility, mi/h

f_{LS}: adjustment for lane and shoulder width, mi/h

f_A: adjustment for access point density, mi/h

There are three important variables in the above estimating formula; the base free-flow speed (*BFFS*), adjustment for lane and shoulder widths, and adjustment for access

point density. Note that because of the broad range of speed conditions on two-lane highways and the importance of local and regional factors that influence drive-desired speeds, the HCM 2000 does not provide any detailed criteria for estimating the *BFFS*. It is limited to a range of 45-65 mi/h. The adjustment factors for lane and shoulder width are shown in Table 5-1. The adjustment factors for access point density are shown in Table 5-2.

Table 5-1. Adjustment (f_{LS}) for lane width and shoulder width

	Reduction in FFS (mi/h)			
	Shoulder Width (ft)			
Lane Width (ft)	>= 0 < 2	>= 2 < 4	>= 4 < 6	>=6
9 < 10	6.4	4.8	3.5	2.2
>= 10 < 11	5.3	3.7	2.4	1.1
>= 11 < 12	4.7	3.0	1.7	0.4
>= 12	4.2	2.6	1.3	0.0

Source: HCM 2000, Chapter 20

Table 5-2. Adjustment (f_A) for access-point density

Access Points per mi	Reduction in FFS (mi/h)
0	0.0
10	2.5
20	5.0
30	7.5
40	10.0

Source: HCM 2000, Chapter 20

Generally field measurement is best option for determination of the free-flow speed, but if not, HCM estimation method can be used. However, if an estimate of the BFFS is not available, FFS could be calculated as posted speed + 5 with the FDOT guidance.

5.2 Performance Measure on the Unaffected Two-Lane Highway Segment

The HCM 2000 Chapter 20 [1] presents operational analysis methodologies for two-way and directional segments of two-lane highways. On a two-lane highway with different types of segments, such as a signalized intersection, an unsignalized intersection, and/or a passing lane, two-way segments typically do not have

homogeneous cross sections or relatively constant demand volumes and vehicle mix proportions in the two directions. Thus, a separate analysis by direction of travel is particularly appropriate. The segment directional methodology for determining the average travel speed on the basic two-lane highway segment is summarized as follows.

Determining FFS

The first step in the analysis of a directional segment is to determine *FFS*. *FFS* can be determined by field measurement or estimation, which has been reviewed in Section 5.1. Note that these methods should be applied on a directional basis rather than to both directions combined.

Determining demand flow rate

This demand flow rate should be based on the *PHF*, the traffic composition, and the terrain or actual grade in the specific direction of travel. The demand flow rate for the peak 15-min period in the direction analyzed is determined with Equation 5-3 [1]:

$$v_d = \frac{V}{PHF \times f_G \times f_{HV}} \tag{5-3}$$

Where:

v_d: passenger-car equivalent flow rate for the peak 15-min period in the direction analyzed (pc/h)

V: demand volume for the full peak hour in the direction analyzed (veh/h)

f_G: grade adjustment factor, and

f_{HV}: heavy-vehicle adjustment factor

A directional analysis also requires consideration of the demand flow rate in the opposing direction. The opposing demand flow rate is computed using Equation 5-4 [1].

$$v_o = \frac{V_o}{PHF \times f_G \times f_{HV}} \tag{5-4}$$

Where:

v_o: passenger-car equivalent flow rate for the peak 15-min period in the opposing direction of travel, and

V_o: demand volume for the full peak hour in the opposing direction of travel.

Determining average travel speed

The average travel speed is estimated from the FFS, the demand flow rate, and an adjustment factor for the percentage of no-passing zones. Average travel speed is then estimated using Equation 5-5 [1].

$$ATS_d = FFS_d - 0.00776(v_d + v_o) - f_{np} \tag{5-5}$$

Where:

ATS_d : average travel speed in the analysis direction (mi/h)

FFS_d : free-flow speed in the analysis direction (mi/h)

v_d: passenger-car equivalent flow rate for the peak 15-min period in the analysis direction (pc/h)

v_o: passenger-car equivalent flow rate for the peak 15-min period in the opposing direction (pc/h)

f_{np}: adjustment for percentage of no-passing zones in the analysis direction

The detailed procedure is described in Chapter 20 (*Two-lane Highways*) of the HCM 2000.

5.3 Performance Measure on an Affected Two-Lane Highway Segment

The presence of a signalized intersection on a two-lane highway can significantly affect traffic operations on the downstream two-lane highway segment, such as decreasing travel speed, and increasing percent time-spent-following. In this section, the

methodology is described, by which the effects of an isolated signalized intersection on a downstream two-lane highway segment can be estimated in terms of average travel speed. That is, to quantify the decrease of average travel speed within the downstream effective length caused by the upstream signalized intersection. The procedure can be broken down into three steps.

Step 1: Determine the average travel speed for the basic two-lane highway section without the signalized intersection, ATS_1.

Step 2: Determine the adjustment factor for the effect of a signalized intersection on average travel speed within the downstream effective length, f_{ATS} .

Step 3: Determine the average travel speed within the downstream effective length, ATS_2.

Step 1: Determine *ATS* without signalized intersection, ATS_1

Two methods can be used to estimate the directional *ATS* for the two-lane highway. One is by microscopic simulation, and the other is using the HCM 2000 directional analysis procedure. The HCM 2000 directional analysis procedure is recommended.

Step 2: Determine the adjustment factor for the effect of a signalized intersection on average travel speed, f_{ATS}

This step can be broken down into three sub-steps. First, divide the downstream effective length of a signalized intersection influence area into multiple short equal-distance intervals, then determine the average travel speed within each interval. Second, divide the two-lane highway segment without an intersection into multiple short equal-distance intervals. Then determine the average travel speed within each interval. Note that the length of each interval is equal to the one above. Third, determine the adjustment

factor for the effect of a signalized intersection on the average travel speed of its downstream two-lane highway segment. The first two sub-steps can be performed with TWOPAS simulation. The simulation procedure is similar to the one for deciding the downstream effective length of a signalized intersection described in the last chapter. The characteristics of the highway section used in the simulation are also the same as those used to determine the downstream effective length of a signal influence area. Here the third sub-step is discussed in detail.

Figure 5-1 shows the difference between $ATS_{1,i}$ and $ATS_{2,i}$ along the facility. This figure is from the output of TWOPAS runs. $ATS_{1,i}$ denotes the average travel speed of the *i*th interval within a two-lane highway with no signalized intersection. $ATS_{2,i}$ denotes the average travel speed of the *i*th interval within a two-lane highway segment downstream of a signalized intersection. The line AB in this figure represents the downstream effective length of a signal influence area, which is divided into multiple equal-distance small intervals. The adjustment factor for ATS is calculated as the average difference between $ATS_{1,i}$ and $ATS_{2,i}$ along the facility, that is

$$f_{ATS} = \frac{1}{n}\sum_{i=1}^{n}(ATS_{1,i} - ATS_{2,i}) \tag{5-6}$$

Where"

f_{ATS} : adjustment factor for the effect of a signalized intersection on average travel speed within the downstream effective length of the signal influence area,

$ATS_{1,i}$: average travel speed of the *i*th section of a two-lane highway without a signalized intersection, mph,

$ATS_{2,i}$: average travel speed of the ith section of a two-lane highway downstream of a signalized intersection, mph, and

n: number of sections within the downstream effective length of a signal influence area.

The difference in average travel speed (Δv) between a two-lane highway with a signalized intersection and without a signalized intersection of ith interval, lying between the ith observing station i and the (i+1)th observing station, can be simplified to the following formula:

$$\Delta v(i+\frac{1}{2}) = \frac{\Delta v(i) + \Delta v(i+1)}{2}$$

And can be donated in Figure 5-1 as the line, $\Delta v(i+\frac{1}{2})$.

Note that the assumption is that the average travel speed variation curve is a straight line over the interval between the ith observing station and the (i+1)th observing station . This approximate solution is acceptable when the distance interval of Δx is small. The adjustment factor for average travel speed can be calculated using Equation 5-7 or Equation 5-8,

$$f_{ATS} = \frac{1}{n}\left(\frac{(\Delta v_1 + \Delta v_2)}{2} + \frac{(\Delta v_2 + \Delta v_3)}{2} + \cdots\cdots + \frac{(\Delta v_{n-1} + \Delta v_n)}{2} + \frac{(\Delta v_n + \Delta v_{n+1})}{2}\right) \quad (5\text{-}7)$$

or

$$f_{ATS} = \frac{1}{2n}\left(\sum_{i=1}^{n+1} \Delta v_i - (\Delta v_1 + \Delta v_{n+1})\right) \quad (5\text{-}8)$$

Step 3: Determine the average travel speed for the downstream highway section within the downstream effective length, ATS_2.

Average travel speed within the downstream effective length of a signalized intersection is generally lower than the average travel speed without a signalized intersection. The effect varies as a function of the directional flow rate and the timing plan of the signalized intersection. The adjustment factors are presented in Table 5-3.

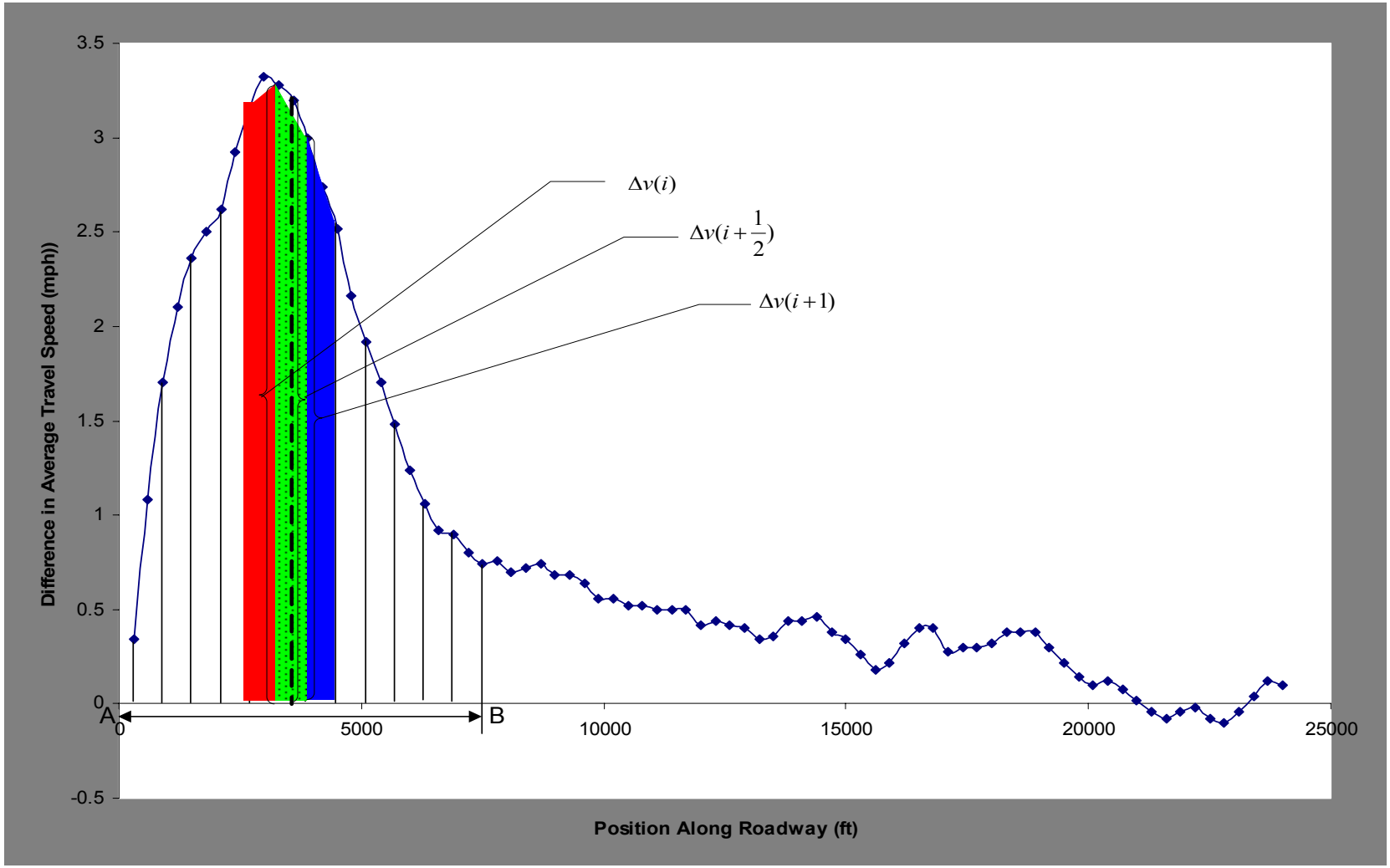


Figure 5-1. Downstream operational effects of a signalized intersection on AT

Within the downstream effective length, average travel speed is assumed to decrease evenly to the value without the effect of the upstream signalized intersection. Thus, the average travel speed within the downstream effective length of a signalized intersection can be computed using Equation 5-9:

$$ATS_2 = ATS_1 - f_{ATS} \tag{5-9}$$

Where:

ATS_2: average travel speed within the downstream effective length of the upstream signalized intersection, mi/h,

ATS_1: average travel speed without the effect of a signalized intersection, mi/h, and

f_{ATS}: adjustment factor for the effect of a signalized intersection on the average travel speed within the downstream effective length, mi/h.

Table 5-3. Adjustment factor, f_{ATS}, to ATS for a segment downstream of a signal

Directional Demand Flow Rate, (pc/h)	Free Flow Speed (mi/h)			
	60	55	50	45
Cycle Length=60, g/C=0.6				
≤220	0.908	0.835	0.762	0.689
440	1.051	0.978	0.905	0.832
660	1.437	1.284	1.130	0.976
880	1.824	1.589	1.354	1.119
≥1100	2.210	1.894	1.579	1.263
Cycle Length=60, g/C=0.7				
≤220	0.676	0.599	0.522	0.445
440	0.741	0.664	0.587	0.510
660	0.921	0.806	0.691	0.575
880	1.102	0.948	0.794	0.641
≥1100	1.282	1.090	0.898	0.706
Cycle Length=60, g/C=0.8				
≤220	0.382	0.321	0.260	0.198
440	0.484	0.423	0.361	0.300
660	0.595	0.530	0.466	0.402
880	0.705	0.638	0.571	0.503
≥1100	0.816	0.746	0.675	0.605
Cycle Length=90, g/C=0.6				
≤220	1.135	1.006	0.878	0.749

440	1.320	1.191	1.062	0.933
660	1.800	1.573	1.345	1.117
880	2.281	1.954	1.628	1.302
≥1100	2.761	2.336	1.911	1.486
Cycle Length=90, g/C=0.7				
≤220	0.814	0.703	0.592	0.482
440	0.912	0.801	0.691	0.580
660	1.107	0.964	0.821	0.678
880	1.303	1.127	0.952	0.777
≥1100	1.498	1.290	1.083	0.875
Cycle Length=90, g/C=0.8				
≤220	0.432	0.371	0.310	0.248
440	0.534	0.473	0.411	0.350
660	0.645	0.580	0.516	0.452
880	0.755	0.688	0.621	0.553
≥1100	0.866	0.796	0.725	0.655
Cycle Length=120, g/C=0.6				
≤220	1.229	1.060	0.892	0.724
440	1.485	1.317	1.148	0.980
660	1.945	1.709	1.472	1.236
880	2.404	2.100	1.797	1.493
≥1100	2.864	2.492	2.121	1.749
Cycle Length=120, g/C=0.7				
≤220	0.855	0.812	0.769	0.726
440	0.929	0.886	0.843	0.800
660	1.128	1.043	0.959	0.874
880	1.326	1.200	1.074	0.948
≥1100	1.525	1.357	1.190	1.022
Cycle Length=120, g/C=0.8				
≤220	0.505	0.454	0.403	0.352
440	0.583	0.532	0.481	0.430
660	0.718	0.648	0.578	0.508
880	0.852	0.763	0.675	0.587
≥1100	0.986	0.879	0.772	0.665

5.4 Service Measure on a Two-Lane Highway with a Passing Lane

When traffic operational problems occur on a two-lane highway, one method for alleviating these problems is to provide passing lanes at regular intervals. Passing lanes cannot increase the capacity of a two-lane highway but can improve its level of service [1]. A passing lane is a lane added in one direction of travel on a conventional two-lane

highway to improve opportunities for passing. The addition of a passing lane to a two-lane highway provides a three-lane cross section with two lanes in one direction of travel and one lane in the other.

Chapter 20 in the HCM 2000 provides an operational analysis procedure for a passing lane on a two-lane highway. Here the procedure to determine average travel speed on the two-lane highway with a passing lane is presented.

The first step in the operation analysis of a passing lane is to apply the procedure for directional segment to the normal cross section without the passing lane. The result is the average travel speed, ATS_d, for the normal two-lane cross section. Installation of a passing lane provides operational benefits for some distance downstream before average travel speed returns to its former level. Thus, the effective length of a passing lane is greater than its actual length. The second step is to divide the analysis segment into four regions. Figure 5-2 shows the segment division and variation in average travel speed in a two-lane highway segment with a passing lane. These divided regions are:

1. Upstream of the passing lane
2. The passing lane
3. Downstream of the passing lane but with its effective length
4. Downstream of the passing lane but beyond its effective length
5. These four lengths add up to the total length of the analysis segment.

The HCM 2000 provides a table on the downstream length of roadway affected by passing lanes on directional segment, L_{de}. The lengths of other regions can be determined by the actual placement of the passing lane within the analysis section. Table 5-4 shows the downstream length of two-lane highway affected by a passing lane, based on the average travel speed.

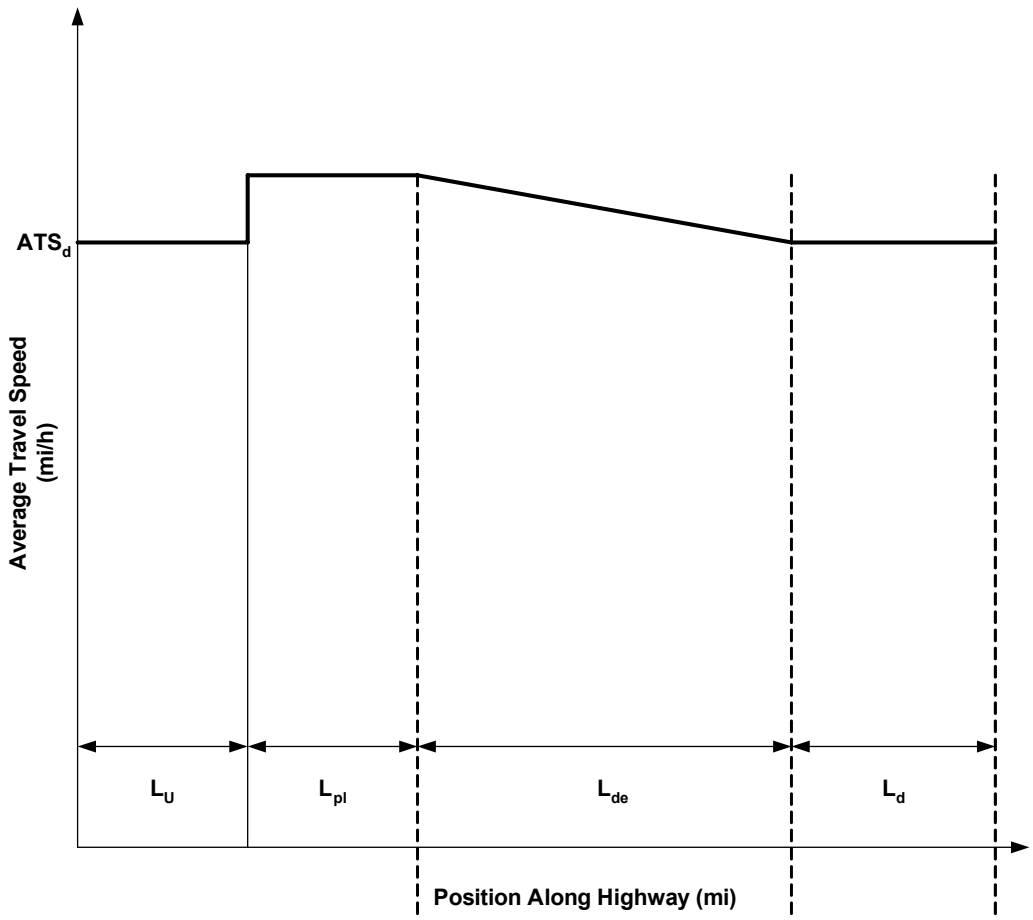


Figure 5-2. Effect of a passing lane on average travel speed

Table 5-4. Downstream length of roadway affected by passing lane

Directional Flow Rate (pc/h)	Downstream Length of Roadway Affected, L_{de} (mi) Average Travel Speed
≤200	1.7
400	1.7
700	1.7
≥1000	1.7

Source: HCM 2000, Chapter 20

The next step is to determine the average travel speed. Average travel speed with lengths L_u and L_d is assumed to equal ATS_d, as predicted by the directional segment procedure. Within the passing lane, average travel speed is generally 8 to 11 percent higher than its upstream value. This effect varies as a function of directional flow rate, as

shown in Table 5-5. Within the downstream length, L_{de}, average travel speed is assumed to decrease linearly with distance from the within-passing lane value to its normal upstream value. Thus, the average travel speed with the passing lane in place can be computed using equation 5-10 [1].

$$ATS_{pl} = \frac{ATS_d \times L_t}{L_u + L_d + \frac{L}{f_{pl}} + \frac{2L_{de}}{1 + f_{pl}}} \tag{5-10}$$

Where

ATS_{pl}: average travel speed for the entire segment including the passing lane (mi/h)

ATS_d: average travel speed for the entire segment without the passing lane

f_{pl}: factor for the effect of a passing lane on average travel speed

Table 5-5. Factors for estimation of average travel speed within a passing lane

Directional Flow Rate (pc/h)	Average Travel Speed
0 – 300	1.08
300 – 600	1.10
> 600	1.11

Source: HCM 2000, Chapter 20

5.5 Service Measure at a Signalized Intersection

The HCM 2000 uses control delay as the service measure for a signalized intersection. Control delay includes "movements at slower speed and stops on intersection approaches as vehicles move up in queue position or slow down upstream of an intersection".

The delay model incorporated in the HCM 2000 includes the uniform delay, a version of Akcelik's [1] overflow delay model, and a term covering delay from an existing or residual queue at the beginning of the analysis period. The control delay per vehicle for a given lane group is given by the following formulas, directly from the HCM

2000. Equation 5-12 gives an estimate of control delay assuming uniform arrivals and stable flow. It is based on the first term of Webster's delay formulation. Equation 5-13 gives an estimate of the incremental delay due to non-uniform arrivals and individual cycle failures, as well as delay caused by sustained periods of over-saturation. Equation 5-14 gives an estimate of the initial queue delay from the previous period at the start of the analysis.

$$d = d_1(PF) + d_2 + d_3 \tag{5-11}$$

$$d_1 = \frac{0.5C(1-\frac{g}{C})^2}{1-[\min(1,X)\frac{g}{C}]} \tag{5-12}$$

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8klX}{cT}}\right] \tag{5-13}$$

$$d_3 = \frac{1800Q_b(1+u)t}{cT} \tag{5-14}$$

Where:

d: control delay per vehicle, s/veh

d_1: uniform control delay, s/veh

d_2: incremental delay, s/veh

d_3: initial queue delay, s/veh

PF: progression adjustment factor

T: analysis period, h

X: v/c ratio

C: cycle length, s

k: incremental delay factor for actuated controller settings

I: upstream filtering/metering adjustment factor

c: capacity, veh/h

Q_b: initial queue at the start of analysis period

u: delay parameter

t: duration of unmet demand in analysis period

The progression adjustment factor is an important factor in calculating the control delay. It is an empirically calibrated adjustment to uniform delay that accounts for the effect of platooned arrival patterns or signal coordination. Progression primarily affects uniform delay, so the progression adjustment factor is applied only to d_1. The value of PF can be determined by Equation 5-15.

$$PF = \frac{(1-P)f_{PA}}{1-\frac{g}{C}} \tag{5-15}$$

Where:

PF: progression adjustment factor

P: proportion of all vehicles arriving during green

g/C: effective green to cycle length ratio

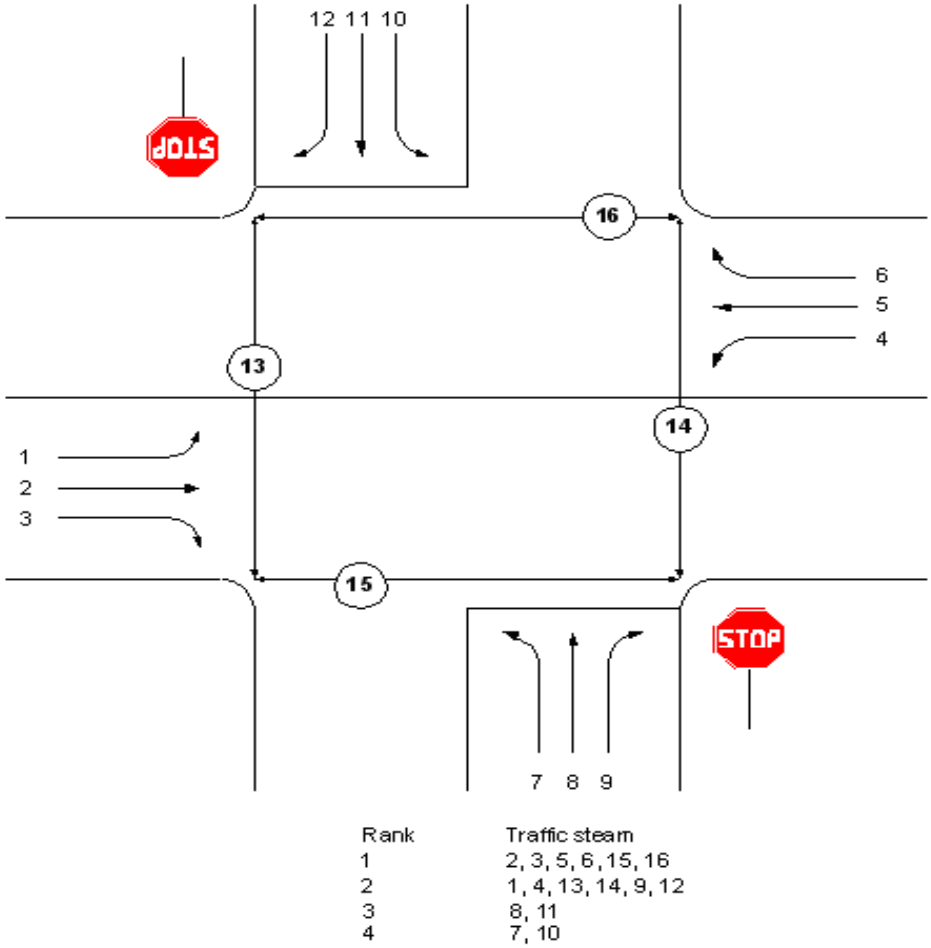
f_{PA}: supplemental adjustment factor for platoon arrival during the green

The procedure of calculating control delay at a signalized intersection is described in detail in the HCM 2000, Chapter 16, *Signalized Intersections*.

5.6 Service Measure at a TWSC Unsignalized Intersection

Usually there are three types of unsignalized intersections: two-way stop-controlled (TWSC), all-way stop-controlled (AWSC), and roundabouts. On a two-lane highway, the TWSC intersection is the most common type of unsignalized intersection. Stop signs are used to control vehicle movements at such intersections. At a TWSC intersection, the

intersection approaches that are not controlled by stop signs are referred to as the major street approaches. The stop-controlled approaches are referred to as the minor street approaches; they can be either public streets or private driveways.



Source: Chapter 17, *Highway Capacity Manual* 2000

Figure 5-3. Traffic streams at a TWSC intersection

In this study, the operational analysis methodology developed is for the traffic on the major two-lane highway. Figure 5-3 shows the right-of way hierarchy at a TWSC intersection. According to the right-of way hierarchy at a TWSC intersection, the major street through and right-turning movements are the highest-priority movements at a

TWSC intersection. These movements have the right-of-way over all minor-street movements at a STOP sign. Left-turning vehicles from the major street onto the minor street yield to opposing through and right-turning vehicles on the major street. Even though the right-of-way hierarchy suggests that the highest-priority movements experience no delay as they travel through a TWSC intersection, experience shows that their right-of-way is sometimes preempted by other conflicting movements. For example, when no exclusive left-turn lane is provided on the major street of a TWSC intersection, a delayed left-turn vehicle may block through and right-turn traffic on the major street (called Rank 1 vehicle). This will delay not only Rank 1 vehicles but also lower-ranked streams. While the delayed Rank 1 vehicles are discharged from the queue formed behind a left-turning vehicle, they impede lower ranked movements with which they conflict.

Chapter 17 of the HCM 2000 [1] provides a method to calculate the average delay to Rank 1 vehicles on the main-street approach. It is computed as follows:

$$d_{Rank1} = \begin{cases} \dfrac{(1-p^*_{0,j})d_{M,LT}\left(\dfrac{v_{i,1}}{N}\right)}{v_{i,1}+v_{i,2}} & \mathrm{N} > 1 \\ (1-p^*_{0,j})d_{M,LT} & \mathrm{N} = 1 \end{cases} \tag{5-16}$$

Where

d_{Rank1}: delay to Rank 1 vehicles, s/veh

N: number of through lanes per direction on the major street

$p^*_{0,j}$: proportion of Rank 1 vehicles not blocked

$d_{M,LT}$: delay to major street left-turning vehicles, s/veh

$v_{i,1}$: major-street through vehicles in shared lane, veh/h, and

$v_{i,2}$: major-street right-turning vehicles in shared lane, veh/h

On a two-lane highway, the number of through lanes per direction on the major street is always 1. $d_{M,LT}$ in Equation 5-16 is estimated using Equation 5-17 [1].

$$d = \frac{3600}{c_{m,x}} + 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right] + 5 \qquad (5\text{-}17)$$

Where

d: control delay, s/veh

v_x: flow rate for movement x, veh/h

$c_{m,x}$: capacity of movement x, veh/h

T: analysis time period, h (T=0.25 for a 15-min period)

The constant value of 5 s/veh included in Equation 5-17 accounts for the deceleration of vehicles from the free-flow speed to the speed of vehicles in queue and the acceleration of vehicles from the stop line to the free-flow speed [1]. The procedure to calculate the delay to Rank 1 vehicles is outlined in Worksheet 11 of Chapter 17 of the HCM 2000.

The control delay for all vehicles on a particular approach of the major street can be computed as the weighted average of the control delay estimated for each movement on the approach. The approach delay in the major street can be calculated using Equation 5-18 [1].

$$d_A = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l} \qquad (5\text{-}18)$$

Where

d_A: control delay on the approach, s/veh

d_r: computed control delay for the right-turn movements, s/veh

d_t: computed control delay for the through movements, s/veh

d_l: computed control delay for the left-turn movements, s/veh

v_r: flow rate of right-turn traffic on the approach, veh/h

v_t: flow rate of through traffic on the approach, veh/h

v_l: flow rate of left-turn traffic on the approach, veh/h

Here the influence length of the delayed left-turn vehicles on the through and right-turn traffic of the major street at the TWSC intersection without an exclusive left-turn lane is discussed. Just like the signalized intersection discussed in Chapter 4, the influence area of the TWSC intersection is also made up of deceleration distance, stopping distance (queue length), and acceleration distance. When calculating the average delay to Rank 1 vehicles on the main-street approach, the constant value of 5 s/veh is included to account for the deceleration of vehicles from the free-flow speed to the speed of vehicles in queue and the acceleration of vehicles from the stop line to the free-flow speed. The base free flow speed for a two-lane highway ranges from 45 to 60 mi/h. In 5 seconds, the distance for the deceleration of vehicles from the free-flow speed to the speed of vehicles in queue and the acceleration of vehicles from the stop line to the free-flow speed ranges from 260 ft to 370 ft. This estimate is based on an assumed acceleration/deceleration rate of 11.2 ft/s^2.

Regarding the queue length of a TWSC intersection, the method of simulation is used to determine the queue length of a TWSC intersection. Synchro and SimTraffic are selected as the simulation programs. Synchro is a complete software package for

modeling and optimizing traffic signal timing and unsignalized intersections. SimTraffic is a companion product to Synchro for performing microscopic simulation and animation. SimTraffic uses a Synchro file for input.

In Synchro, a vehicle is considered queued whenever it is traveling at less than 10 ft/s. A vehicle will only become "queued" when it is either at the stop bar or behind other queued vehicles [17]. In Synchro, there are two options for reporting the intersection statistical information, the Synchro method or the HCM method. To maintain fidelity to the existing HCM 2000 , the option of 'HCM Un-signalized' is chosen to create the HCM Un-signalized Intersection Report. The queue length can be obtained from the report [17].

Contributing factors selected for the determination of the queue length of a TWSC intersection influence area on the major approach due to the left-turn vehicles are presented in Table 5-6.

Table 5-6. Variable input values (TWSC intersections)

Peak Volume (pc/h)	D-Factor	Percentage of left-turn vehicles (%)
400	0.60	5%
700	0.70	10%
1000	0.80	15%

The simulation results on the queue length of the TWSC intersection are presented in Table 5-7. The preliminary simulation results show that the queue lengths of the TWSC intersection influence area on the major approach are less than 200 ft in most cases. Only when the left-turn vehicle volume on the major approach is much higher, and the opposing traffic volume is also much higher, the queue length on the main approach is above 200 ft.

Table 5-7. Simulation results on the queue length of the TWSC intersection

Peak Volume (pc/h)	D-Factor	Percentage of left-turn vehicles (%)	Queue Length (ft)
400	0.6	5%	0
400	0.6	10%	0
400	0.6	15%	0
400	0.7	5%	0
400	0.7	10%	0
400	0.7	15%	0
400	0.8	5%	0
400	0.8	10%	0
400	0.8	15%	0
700	0.6	5%	30
700	0.6	10%	85
700	0.6	15%	169
700	0.7	5%	16
700	0.7	10%	86
700	0.7	15%	97
700	0.8	5%	11
700	0.8	10%	14
700	0.8	15%	17
1000	0.6	5%	83
1000	0.6	10%	221
1000	0.6	15%	432
1000	0.7	5%	60
1000	0.7	10%	69
1000	0.7	15%	185
1000	0.8	5%	21
1000	0.8	10%	36
1000	0.8	15%	49

When the percentage of left-turn vehicles on the major approach increases, the effective length also increases. However, when the percentage of left-turn vehicles on the major approach increases to some point, an exclusive left-turn lane is usually provided at the TWSC intersection. NCHRP REPORT 457 [18] provides the guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection. A left-turn lane is recommended on the major road of any intersection when the combination of intersection volumes intersect above or to the right of the appropriate trend line shown in Figure 5-4. When a left-turn bay is added to the major-road approach, the assumption is that there would be no delay for the Rank 1 vehicles.

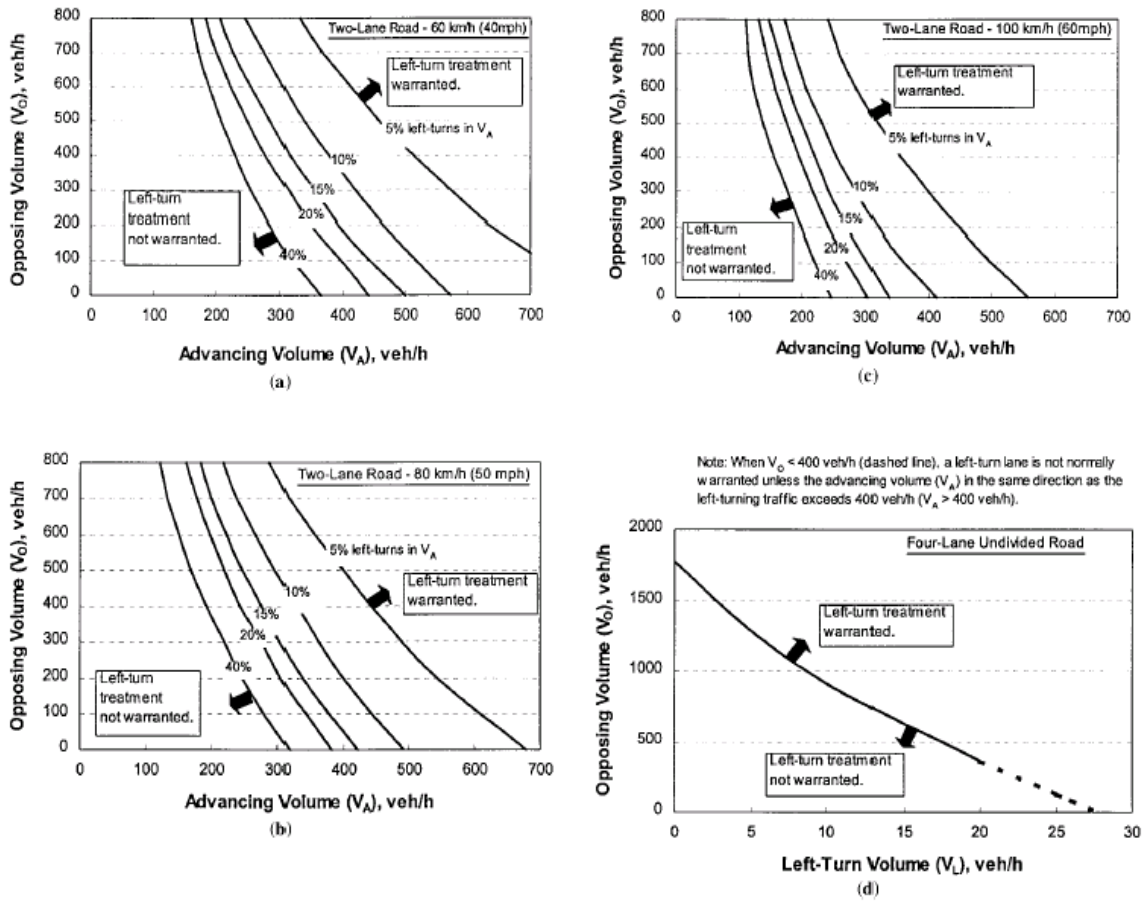


Figure 5-4. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection

Source: Reference 18.

The above analysis shows that the sum of deceleration distance and acceleration distance of the TWSC intersection on the major approach ranges from 260 ft to 370 ft, and the queue length is less than 200 ft. So the sum of deceleration distance, queue length, and acceleration distance of the TWSC intersection without a left-turn bay on the main approach due to left-turn vehicles approaches 500 ft. In this dissertation, the effective length of the influence area due to left-turn vehicles at the TWSC intersection without an exclusive left-turn lane on the through and right-turn vehicles is recommend as extending 0.1 miles (528 ft) in length centered on the midpoint of the crossing facility.

Figure 5-5 shows an example of how to determine the effective length on a two-lane highway facility of a TWSC intersection without an exclusive left-turn lane. In this example, a two-lane highway with a TWSC intersection extends 5 miles, and the isolated intersection is located at the 3-mile point. The first 2.95 miles would be regarded as a two-lane highway segment, the next 0.1 miles would be regarded as the intersection area, and the last 1.95 miles would be regarded as a two-lane highway segment.

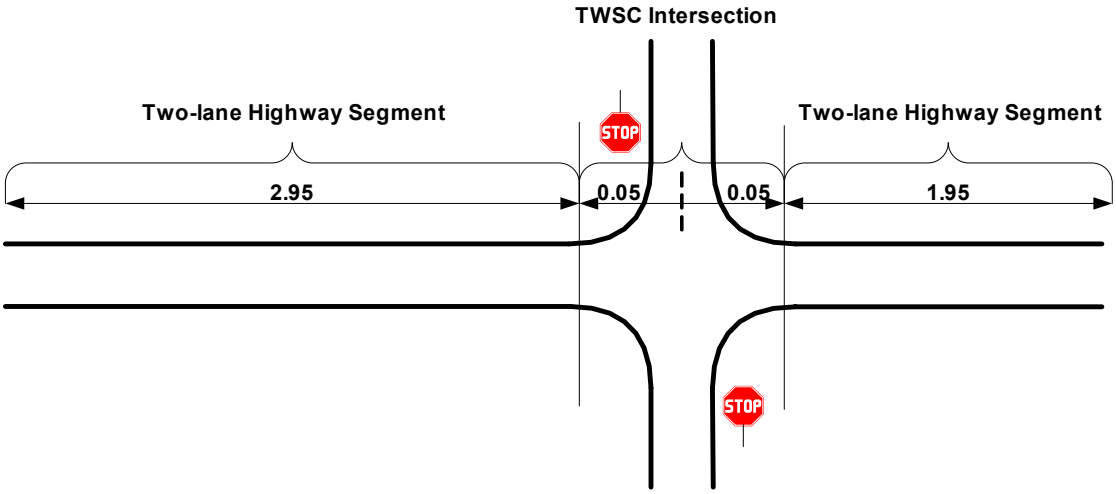


Figure 5-5. Length of TWSC intersection area

5.7 Level of Service Thresholds

To complete a level of service (LOS) methodology, it is necessary to define values of the service measure that serve to delineate between the various levels of service, A-F. The selected values, or thresholds, should be chosen such that they correspond to drivers' level of satisfaction with the operating conditions for the given level of the service measure.

However, without the benefit of research that directly investigates driver satisfaction on these facilities under varying levels of the service measure, there is no way to be sure that the chosen thresholds correlate with driver expectations. Nonetheless,

threshold values must still be chosen such that a level of service value can be assigned to the analysis results.

Although the selection of these threshold values is somewhat arbitrary without the benefit of driver perception-based research, there is some existing information that can be used to guide the threshold value selection. This information is the existing LOS thresholds for basic two-lane highway segments. It is important to recognize that adding a signalized intersection along a two-lane highway should not result in an improvement to the LOS that would be estimated for an equivalent two-lane highway with no signalized intersection(s) present.

Preliminary LOS threshold values have been selected and are shown in Table 5-8. The application of these thresholds is demonstrated in the example problems of Chapter 6.

Table 5-8. LOS criteria for two-lane highway facilities

Level of Service	**Percent Time-Delayed**
A	< 7.5%
B	7.5%-15%
C	15%-25%
D	25%-35%
E	35%-45%
F	> 45%

CHAPTER 6
APPLICATION EXAMPLES

The methodologies developed in this research can be used to assess the operational performance of an extended length of a two-lane highway facility, comprised of multiple segments, with segment delineations occurring with a change in either roadway or control attributes. A common application of the methodologies is to determine the LOS of a current or a changed facility in the near term or in the future. The primary outputs are the performance measure values of delay time and percent time-delayed, as well as a level of service ranking for the analyzed facility.

This dissertation focuses the efforts on developing the methodology for operational analysis of a two-lane highway with an isolated signalized intersection at the facility level. It is intended that this research will also provide a model for the basic structure of a facility level analysis that will be amenable to the incorporation of a variety of segment types. In this chapter, two examples are provided to illustrate the application of the developed methodology.

6.1 Input Parameters

Table 6-1 provides a summary of the input information required to conduct an operational analysis for the two-lane highway facility composed different types of segments. This information forms the basis for selecting computational values and procedures in the methodology developed in the dissertation. The data needed are detailed and varied and fall into two main categories: facility-wide data, and segment-wide data. The facility-wide data includes geometric and traffic conditions.

Table 6-1. Input data needs for the facility-wide operational analysis

Facility-wide information	
(1) Geometric Conditions	(2) Traffic conditions
Roadway length, (ft)	Demand volume by movements, (veh/h)
Lane width, (ft)	Base saturation flow rate, (pc/h/ln)
Shoulder width, (ft)	Base FFS, (mi/h)
Terrain	Directional distribution
Density of access points	Peak-hour factor, PHF
Proportion of no-passing zones, (%)	Proportion of trucks and buses, (%)
	Proportion of RVs, (%)
Segment-wide information	
(1) Signalized intersection	(2) Unsignalized intersection
Location of intersection	Location of Intersection
Traffic volume from minor streets, (veh/h)	Traffic volume from minor streets, (veh/h)
Existence of exclusive LT lane	Existence of exclusive LT lanes
Proportion of left-turn vehicles, (%)	Proportion of left-turn vehicles, (%)
Proportion of right-turn vehicles, (%)	Proportion of right-turn vehicles, (%)
Signal phasing plan	
Signal timing data	(3) Passing lane
Arrival type	Location of the beginning of passing lane
Area type	Length of passing lane, (ft)
Length of storage lanes, (ft)	

6.2 Example 1

The Facility:

A rural two-lane highway facility extends 7 miles with an isolated signalized intersection at the 3-mile point. Figure 6.1(a) illustrates the components of the facility.

Determine:

The percent time-delayed and level of service on this two-lane highway facility for the peak hour

The Facts:

- Roadway Data

➢ Level terrain ➢ 6-ft shoulder width ➢ 5 access points/mi	➢ 60 mi/h base FFS ➢ 12-ft lane width ➢ 50% no-passing zones

➢ Downstream with 4-mile length	➢ Upstream with 3-mile length

- Traffic Data

➢ 1000 veh/h (two-way volume) ➢ 3 percent trucks and buses ➢ 0.95 PHF	➢ 60/40 split ➢ 2 percent RVs

- Control Data

➢ EB and WB HV = 5 percent ➢ 0.95 PHF ➢ EB-WB green = 54 s, NB-SB green = 26 s ➢ Cross street has one lane in each direction ➢ 6-ft shoulder width ➢ No parking at intersection	➢ NB and SB HV = 5 percent ➢ Two-phase signal ➢ Yellow = 4 s ➢ Main street has one lane and a left-turn bay in each direction. ➢ Movement lost time = 5 s ➢ 12-ft lane width ➢ Level terrain

Outline of Solution:

1. Divide the facility into segments.

2. Determine segment lengths.

3. Calculate the free-flow speed.

4. Calculate the average travel speed on the two-lane highway segment upstream of the signal influence area.

5. Calculate control delay at the signalized intersection influence area.

6. Determine average travel speed on the affected downstream segment.

7. Determine average travel speed on the unaffected downstream segment.

8. Determine the delay of every segment.

9. Determine the percent time-delayed and LOS of the entire facility.

INPUT WORKSHEET

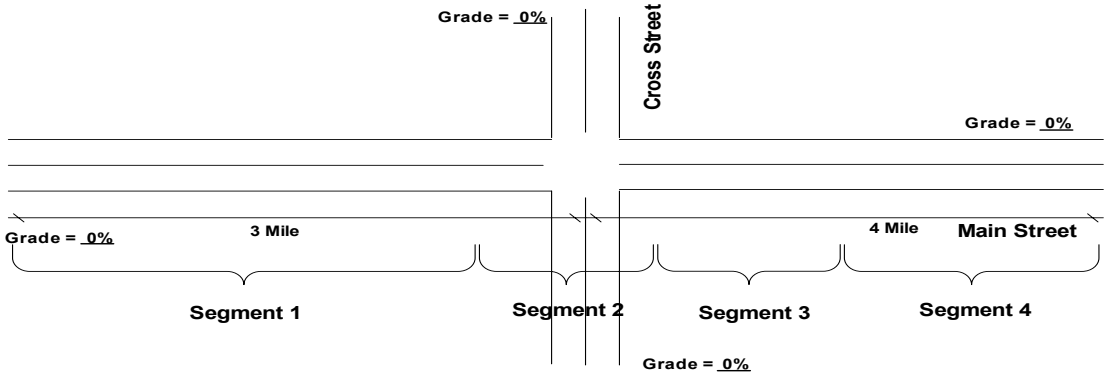
General Information

Analyst _____ .
Date Performed _____ .
Analysis Time Period _____ .

Site Information

Intersection _____ .
Area Type _____ .
Analysis Year _____ .

Facility Geometry



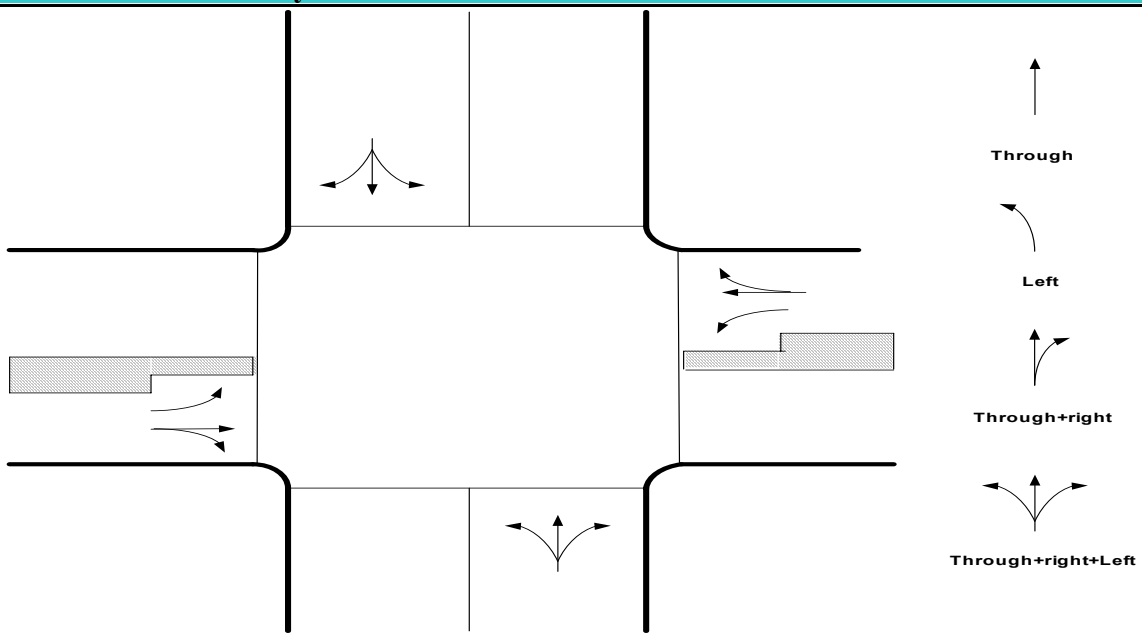
Input Data of Two-Lane Highways

Class II highway

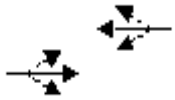
Two-way hourly volume	1000 veh/h	Directional split	60/40
Peak-hour factor, PHF	0.95	Access points/mi	5 /mi
% Trucks	3 %	Base FFS	60 mi/h
% Recreational vehicles	2 %	Shoulder width	6 ft
% No-passing zone	50 %	Lane width	12 ft
Upstream segment length	3 mi	Downstream segment length	4 mi
Terrain	Level		

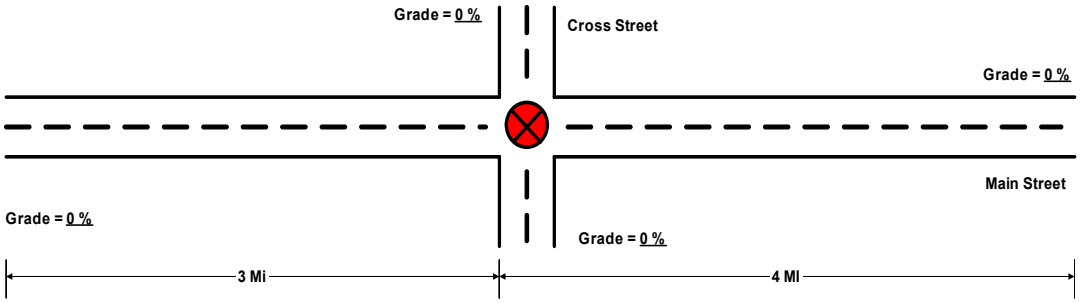
Input Data of Signalized Intersection

Intersection Geometry

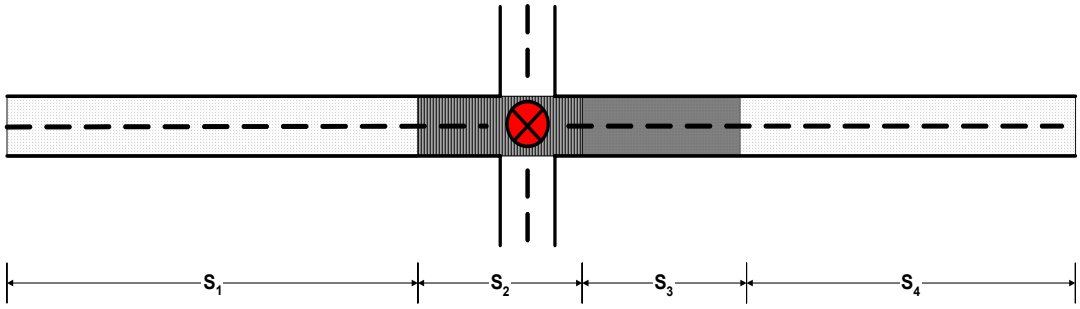


Volume and Signal Timing Input												
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	50	500	50	50	300	50	50	100	50	50	100	50
% heavy vehicle, %HV	5	5	5	5	5	5	5	5	5	5	5	5
Peak-hour factor, PHF	0.95			0.95			0.95			0.95		
Pretimed(P), Actuated (A)	P			P			P			P		
Start-up Lost time, l (s)	2			2			2			2		
Arrival type, AT	3			3			3			3		
Parking (Y or N)	N			N			N			N		
Parking maneuvers,	0			0			0			0		
Bus stopping	0			0			0			0		

Signal Phasing Plan			
Diagram	 Phase 1	 Phase 2	
Time	G= 54.0 Y+R=5.0	G= 26.0 Y+R=5.0	Cycle Length, C = 90.0 s



(A)



(B)

Figure 6-1. A two-lane highway with an isolated signalized intersection

Step 1: Divide the facility into segments

The first step is to divide the facility into uninterrupted and interrupted flow segments. Each segment has homogenous characteristics. Figure 6-1 (B) shows the segmentation of a two-lane highway with an isolated signalized intersection. The whole two-lane highway facility is divided into four segments. These segments are:

- Segment 1: the basic two-lane highway segment. It is located upstream of the signalized intersection. This segment is not affected by the downstream signalized intersection.
- Segment 2: the influence area of the signalized intersection. It is composed of not only its own actual length, but also the deceleration distance, stopping distance and acceleration distance.
- Segment 3: the affected downstream two-lane highway segment. It is located downstream of the signalized intersection, and affected by the upstream signalized intersection. Note that this segment length does not include the acceleration distance of the signal influence area.
- Segment 4: the basic two-lane highway segment, it is located downstream of the signalized intersection. Just like Segment 1, this segment is not affected by the upstream signalized intersection.

These four lengths add up to the total length of the analysis facility.

Step 2: Determine segment lengths

The length of the signalized intersection influence area, L_2, is calculated as the sum of upstream effective length and acceleration length.

The upstream effective length is calculated using Equation 4-17.

$$\begin{aligned} Len_{eff\_up} &= 43.2463 + 4.2688 \times (V/100)^2 + 5.2178 \times Cycle \\ &\quad - 57.3041 \times (V/100) \times \%LT - 5.2444 \times Cycle \times g\_C \\ &= 43.2463 + 4.2688 \times (600/100)^2 + 5.2178 \times 90 \\ &\quad - 57.3041 \times (600/100) \times 8.3\% - 5.2444 \times 90 \times 0.6 \\ &= 355 \text{ ft} \end{aligned}$$

The acceleration length can be determined using the linearly-decreasing acceleration model, or obtained from the reference table in *Transportation and Traffic*

Engineering Handbook, which is presented in Chapter 4, Table 4-1. Here the acceleration distance uses the value from this reference table.

$L_A = 574$ ft

The length of Segment 2, L_2, is:

$$L_2 = L_{eff\_up} + L_A$$
$$= 355 + 574 = 929 \text{ ft} = 0.18 \text{ mi}$$

The length of the conventional two-lane highway segment upstream of the signalized intersection, L_1, is calculated by subtracting the upstream part of the signalized intersection influence area from the length of the two-lane highway upstream of the signalized intersection. That is,

$$L_1 = L_{U1} - L_{eff\_up}$$
$$= 3.0 - 355/5280 = 2.93 \text{ mi}$$

The length of the affected downstream segment of the signalized intersection, L_3, is calculated by subtracting the acceleration length of the signal influence area from the downstream effective length. The downstream effective length is calculated using Equation 4-25.

$$Len_{eff\_down} = 2.218584 - 0.122942 \times (V/100)$$
$$= 2.218584 - 0.122942 \times (600/100)$$
$$= 1.48 \text{ mi}$$

The length of Segment 3, L_3, is,

$$L_3 = Len_{eff\_down} - L_A$$
$$L_3 = 1.48 - (574/5280) = 1.37 \text{ mi}$$

Segment 4 is located downstream of the signalized intersection. The length of Segment 4 can be calculated by subtracting the total length of Segment 1, 2 and 3 from the total length of the facility. The length of Segment 4, L_4, is,

$$L_4 = L_T - (L_1 + L_2 + L_3)$$

$$L_4 = (3.0 + 4.0) - (2.93 + 0.18 + 1.37) = 2.52 \text{ mi}$$

Step 3: Calculate the free-flow speed

$$\begin{aligned} FFS &= BFFS - f_{LS} - f_A \\ &= 60 - 0 - 1.3 \\ &= 58.7 \text{ mi/h} \end{aligned}$$

The above equation is Equation 20-2 in the HCM 2000. The f_{LS} and f_A values are from Exhibits 20-5 and 20-6, respectively, in Chapter 20 of the HCM 2000.

Step 4: Calculate the average travel speed on the unaffected upstream segment.

Use the HCM 2000 methodology to calculate the *ATS* (Chapter 20)

ATS_1 = 48.4 mi/h

Step 5: Calculate control delay at the signalized intersection influence area.

Use the HCM 2000 methodology to calculate the control delay (Chapter 16)

Control Delay = 12.8 sec/veh

Step 6: Determine average travel speed on the unaffected downstream segment

Use the 2000 HCM methodology to calculate the average travel speed.

ATS_4 = 48.4 mi/h

Step 7: Determine average travel speed on the affected downstream segment.

For the affected downstream segment of the signalized intersection, an adjustment factor for the effect of a signalized intersection on average travel speed will be applied to the average travel speed without a signalized intersection to compute the average travel speed on the affected downstream segment of the signalized intersection. The adjustment factors can be obtained from Table 5-3 based on the traffic flow rate. The adjustment factor for Segment 4 can be interpolated as

$$f_{ATS} = 1.32 + (600 - 440) \times \frac{1.800 - 1.320}{660 - 440} = 1.669 \text{ mi/h}$$

So the average travel speed of Segment 4 is

$$ATS_2 = ATS_4 - f_{ATS}$$
$$= 48.4 - 1.669 = 46.73 \text{ mi/h}$$

Step 8: Determine the delay of every segment.

The delay at Segment 1, D_1, is

$$L_1 = 2.93 \text{ mi}$$
$$S_1 = 48.4 \text{ mi/h}$$
$$FFS_1 = 58.7 \text{ mi/h}$$
$$D_1 = \frac{L_1}{S_1} - \frac{L_1}{FFS_1} = (\frac{2.93}{48.4} - \frac{2.93}{58.7})(3600) = 38.24 \text{ sec/veh}$$

The delay at segment 2, D_2, is

$$L_2 = 0.18 \text{ mi}$$
$$D_2 = 12.8 \text{ sec/veh}$$

The delay at Segment 3, D_3, is

$$L_3 = 1.37 \text{ mi}$$
$$S_3 = 46.73 \text{ mi/h}$$
$$FFS_3 = 58.7 \text{ mi/h}$$
$$D_3 = \frac{L_3}{S_3} - \frac{L_3}{FFS_3} = (\frac{1.37}{46.73} - \frac{1.37}{58.7})(3600) = 21.52 \text{ sec/veh}$$

The delay at Segment 4, D_4, is

$$L_4 = 2.52 \text{ mi}$$
$$S_4 = 48.4 \text{ mi/h}$$
$$FFS_4 = 58.7 \text{ mi/h}$$
$$D_4 = \frac{L_4}{S_4} - \frac{L_4}{FFS_4} = (\frac{2.52}{48.4} - \frac{2.52}{58.7})(3600) = 32.89 \text{ sec/veh}$$

Step 9: Determine the percent time-delayed of the entire facility.

1. The total length of the facility

Length of Segment 1, L_1	2.93 miles
Length of Segment 2, L_2	0.18 miles
Length of Segment 3, L_3	1.37 miles
Length of Segment 4, L_4	2.52 miles
Total Length, L_t	$L_t = L_1 + L_2 + L_3 + L_4$ $L_t = 2.93 + 0.18 + 1.37 + 2.52 = 7$ miles

2. The total delay of the facility

Delay of Segment 1, D_1	38.24 sec
Delay of Segment 2, D_2	12.80 sec
Delay of Segment 3, D_3	21.52 sec
Delay of Segment 4, D_4	32.89 sec
Total Delay, D_T	$D_T = D_1 + D_2 + D_3 + D_4$ $= 38.24 + 12.8 + 21.52 + 32.89 = 105.45$ sec/veh

3. Calculate the total travel time of the facility based on the free flow speed

$$T_{tFFS} = \frac{L}{FFS} = \frac{7}{58.7}(3600) = 429 \text{ sec/veh}$$

4. Calculate the percent time-delayed of the facility

$$PTD = \frac{\sum_{H,S}(D_H + D_S)}{\sum_{H,S}(\frac{L_H}{FFS_H} + \frac{L_S}{FFS_S})} = \frac{D_T}{\frac{L_t}{FFS}}$$

$$= \frac{105.45}{429} = 24.58\%$$

From Table 5-8, this value of percent time-delayed gives an LOS value of 'C', albeit barely, as the value of 24.6% is just under the 'C/D' threshold of 25.0%. By comparison, an average travel speed of 48.4 mph, for a two-lane highway with no signalized intersection, would also yield an LOS of 'C' using the criteria in Exhibit 20-2 of the HCM 2000 (for *ATS* only). The LOS 'C' range in this table is 45-50 mph. Although the level of service is the same in this situation, as opposed to being made worse due to the presence of the signal, it is still reasonable given that the average signal delay is only 12.8 seconds over a 7-mile length of highway. Thus, the relatively good signal conditions in this case are probably having a very minor impact on the overall trip quality over this length of highway.

6.2 Example 2

The Facility:

A rural two-lane highway facility extends 20 miles with two isolated signalized intersections, a two-way stop-controlled intersection, and a passing lane. The first isolated signalized intersection is installed beginning at a location 3 mile downstream from the beginning of the 20-mile two-lane highway in the analysis direction; the second one is at the16-mile point. The two-way stop-controlled intersection is located at the 13-mile point. A 1.5-mile passing lane is also added at the 7-mile point. Figure 6.2-(a) illustrates the components of the facility.

Determine:

The percent time-delayed and level of service on this two-lane highway facility for the peak hour.

The Facts:

- Two-lane highway segments
  - 1200 veh/h (two-way volume)
  - 60/40 split
  - 3 percent trucks and buses
  - 2 percent RVs
  - 0.95 PHF
  - 65 mi/h base FFS
  - Level terrain
  - 12-ft lane width
  - 6-ft shoulder width
  - 40% no-passing zones
  - 5 access points/mi
  - 20-mile roadway Length
  - Three intersections and one passing lane
  - The first signalized intersection is located 3 mi downstream
  - A 1.5-mile length of passing lane including tapers at a location 7 mi downstream from the beginning of the 20-mile two-lane highway in the analysis direction
  - A two-way stop-controlled intersection is located 13 mi downstream
  - The second signalized intersection at a location 16 mi downstream

- The First Signalized intersection:
  - EB and WB HV = 5 percent
  - NB and SB HV = 5 percent
  - 0.95 PHF
  - Two-phase signal
  - EB-WB green = 54 s,
  - Yellow = 4 s
  - NB-SB green = 26 s
  - Main street has one lane in each direction.
  - Cross street has one lane in each direction
  - Movement lost time = 5 s
  - 6-ft shoulder width
  - 12-ft lane width
  - No parking at intersection
  - Level terrain

- The Second Signalized Intersection
  - EB and WB HV = 5 percent
  - NB and SB HV = 5 percent
  - 0.95 PHF
  - Two-phase signal
  - EB-WB green = 63 s,
  - Yellow = 4 s
  - NB-SB green = 17 s
  - Main street has one lane and a left-

- Cross street has one lane in each direction
- 6-ft shoulder width
- No parking at intersection
- turn bay in each direction.
- Movement lost time = 5 s
- 12-ft lane width
- Level terrain

- The Two-Way Stop-Controlled Intersection
  - Two-lane major street
  - Two-lane minor street
  - Stop-controlled on minor street approaches
  - 5 percent HV
  - No special intersection geometry
  - No pedestrians
  - Level grade

Outline of Solution:

1. Divide the facility into segments.

2. Determine segment lengths.

3. Calculate the free-flow speed.

4. Determine the average travel speed on the unaffected basic two-lane segments (**Segment 1, Segment 4, Segment 7, Segment 9, and Segment 12).**

5. Determine the average travel speed of the affected downstream segments of the signalized intersection **(Segment 3, Segment 11).**

6. Determine the average travel speed within the passing lane and its affected downstream segment **(Segment 5 and Segment 6).**

7. Determine the control delay of the signal influence areas **(Segment 2, and Segment 10)**.

8. Determine the delay of the unsignalized intersection **(Segment 8).**

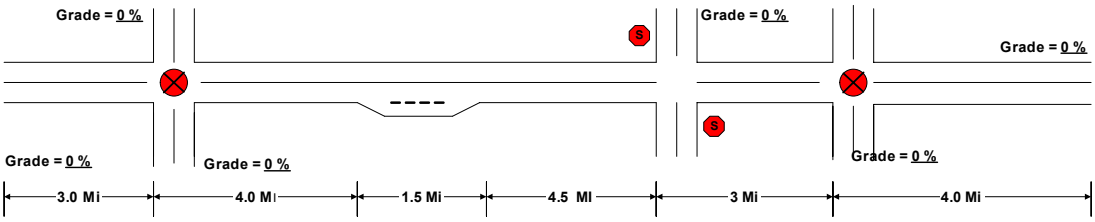
9. Determine the performance measure of every segment.

10. Determine the performance measure and LOS of the entire facility.

INPUT WORKSHEET

General Information		Site Information	
Analyst	_____ .	Intersection	_____ .
Date Performed	_____ .	Area Type	_____ .
Analysis Time Period	_____ .	Analysis Year	_____ .

Facility Geometry



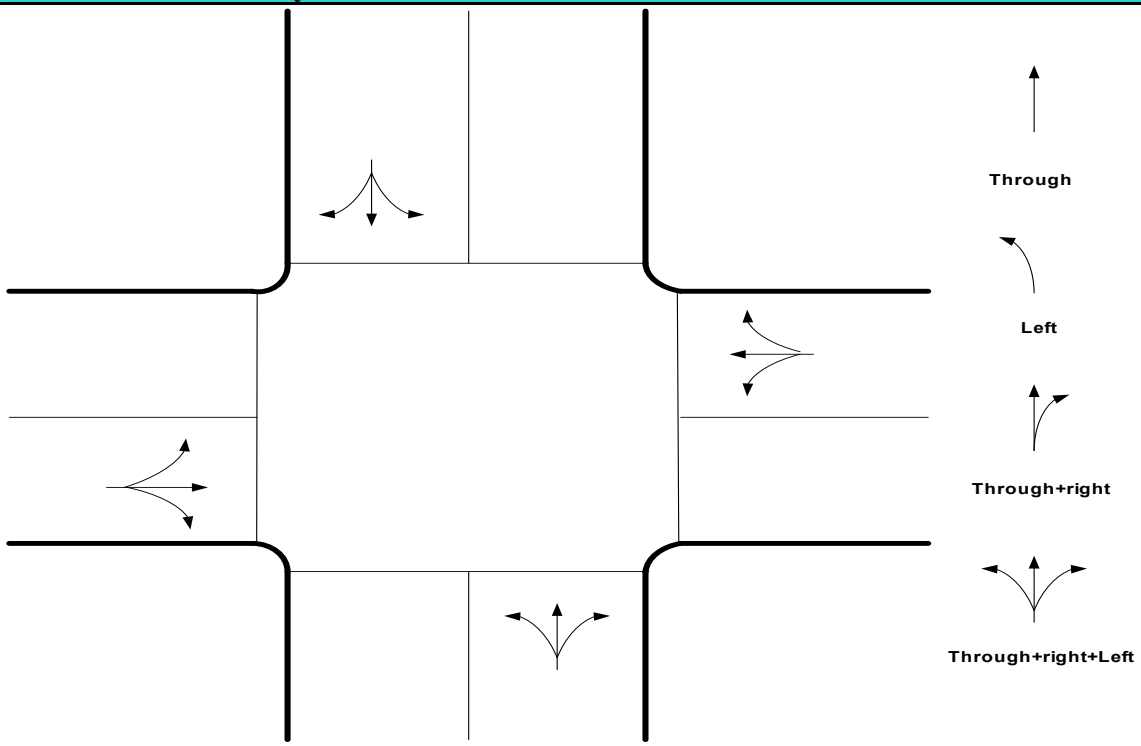
Input Data of Two-Lane Highways

Class I highway

Two-way hourly volume	1200 veh/h	Directional split	60/40
Peak-hour factor, PHF	0.95	Access points/mi	5 /mi
% Trucks and	3 %	Base FFS	60 mi/h
% Recreational vehicles	2 %	Shoulder width	6 ft
% No-passing zone	40 %	Lane width	12 ft
Facility length	20 mi	Terrain	Level

Input Data of the First Signalized Intersection

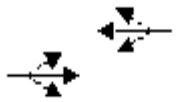
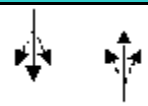
Intersection Geometry



Volume and Time Input

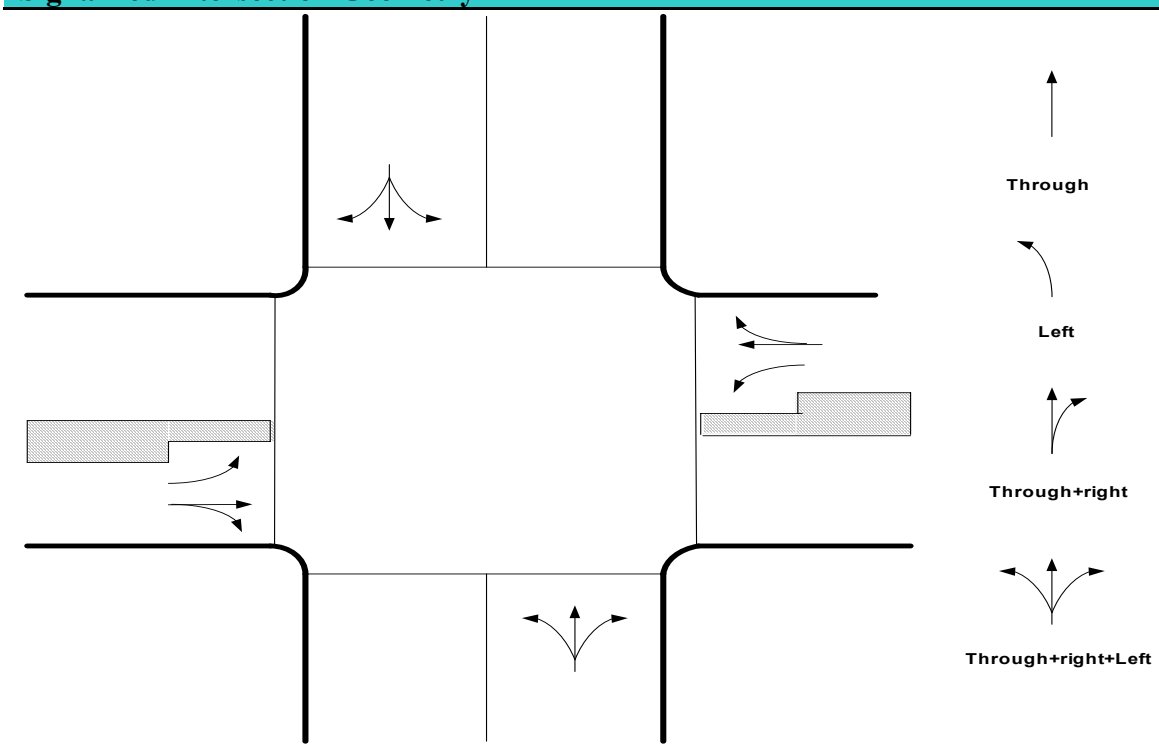
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, v (veh/h)	50	620	50	50	230	50	40	50	115	115	90	40
% heavy vehicle, %HV	5	5	5	5	5	5	5	5	5	5	5	5
Peak-hour factor, PHF	0.95			0.95			0.95			0.95		
Pre-time(P), Actuated (A)	P			P			P			P		
Start-up Lost time, l (s)	2			2			2			2		
Arrival type, AT	3			3			3			3		
Parking (Y or N)	N			N			N			N		
Parking maneuvers,	0			0			0			0		
Bus stopping	0			0			0			0		

Signal Phasing Plan

Diagram	 Phase 1	 Phase 2	
Time	G= 54.0 Y+R=5.0	G= 26.0 Y+R=5.0	Cycle Length, C = 90.0 s

Input Data of the Second Signalized Intersection

Signalized Intersection Geometry

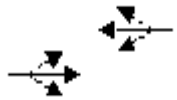


Volume and Time Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, v (veh/h)	100	700	50	120	240	120	60	100	85	85	120	60
% heavy vehicle, %HV	5	5	5	5	5	5	5	5	5	5	5	5
Peak-hour factor, PHF	0.95			0.95			0.95			0.95		

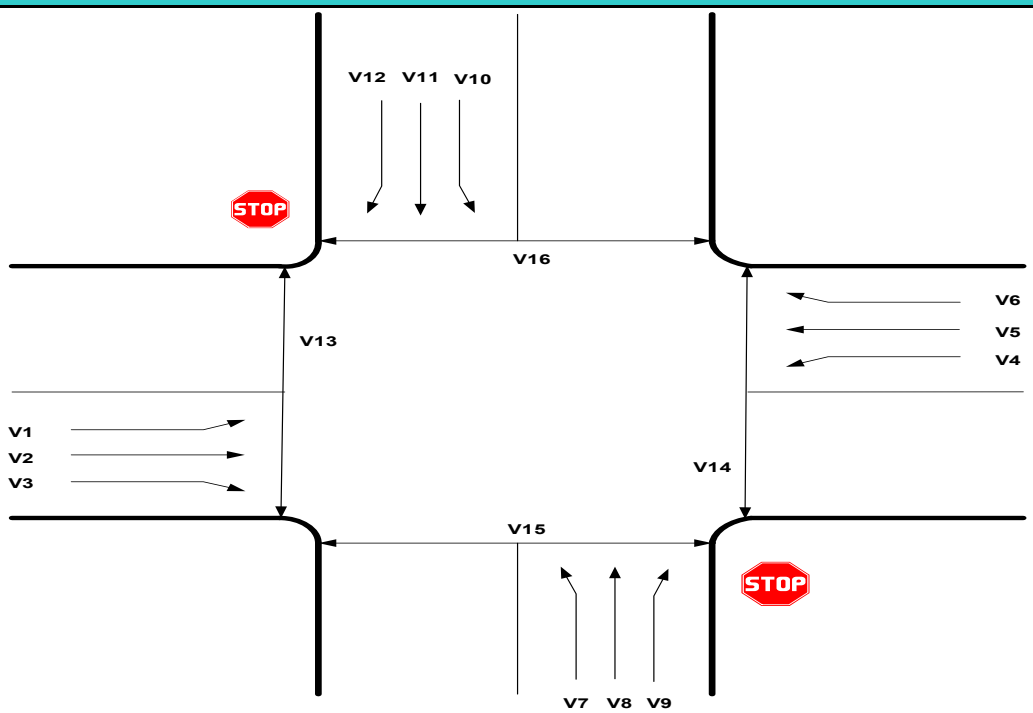
Pre-time(P), Actuated (A)	P	P	P	P
Start-up Lost time, l (s)	2	2	2	2
Arrival type, AT	3	3	3	3
Parking (Y or N)	N	N	N	N
Parking maneuvers,	0	0	0	0
Bus stopping	0	0	0	0

Signal Phasing Plan

Diagram	 Phase 1	 Phase 2	
Time	G= 63.0 Y+R=5.0	G= 26.0 Y+R=5.0	Cycle Length, C = 90.0 s

Input Data of the TWSC Intersection

Geometrics and Movements



Vehicle Volumes and Adjustments

Movement	Vehicles Volumes and Adjustments											
	1	2	3	4	5	6	7	8	9	10	11	12
Volume (veh/h)	120	700	30	65	250	45	40	40	80	70	40	40
Peak-hour factor, PHF	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Hourly flow rate, (veh/h)	100	630	30	30	360	80	40	40	90	80	40	50
Proportion of heavy vehicles, P_{HV}	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1

Pedestrian Volume and Adjustments

Movement	13	14	15	16
Flow, V_X (ped/h)	0	0	0	0

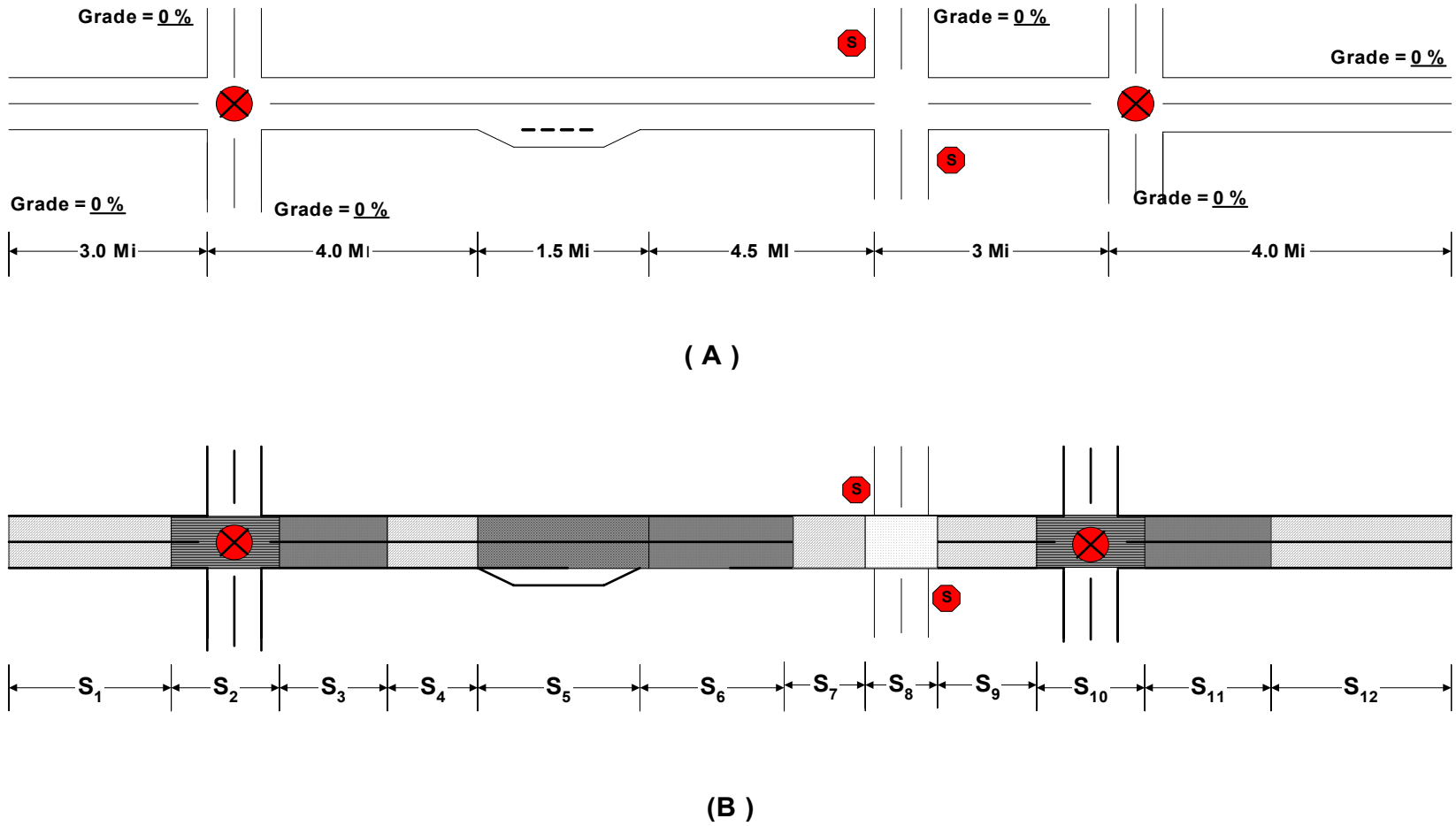


Figure 6-2. Segmentation for a two-lane highway facility

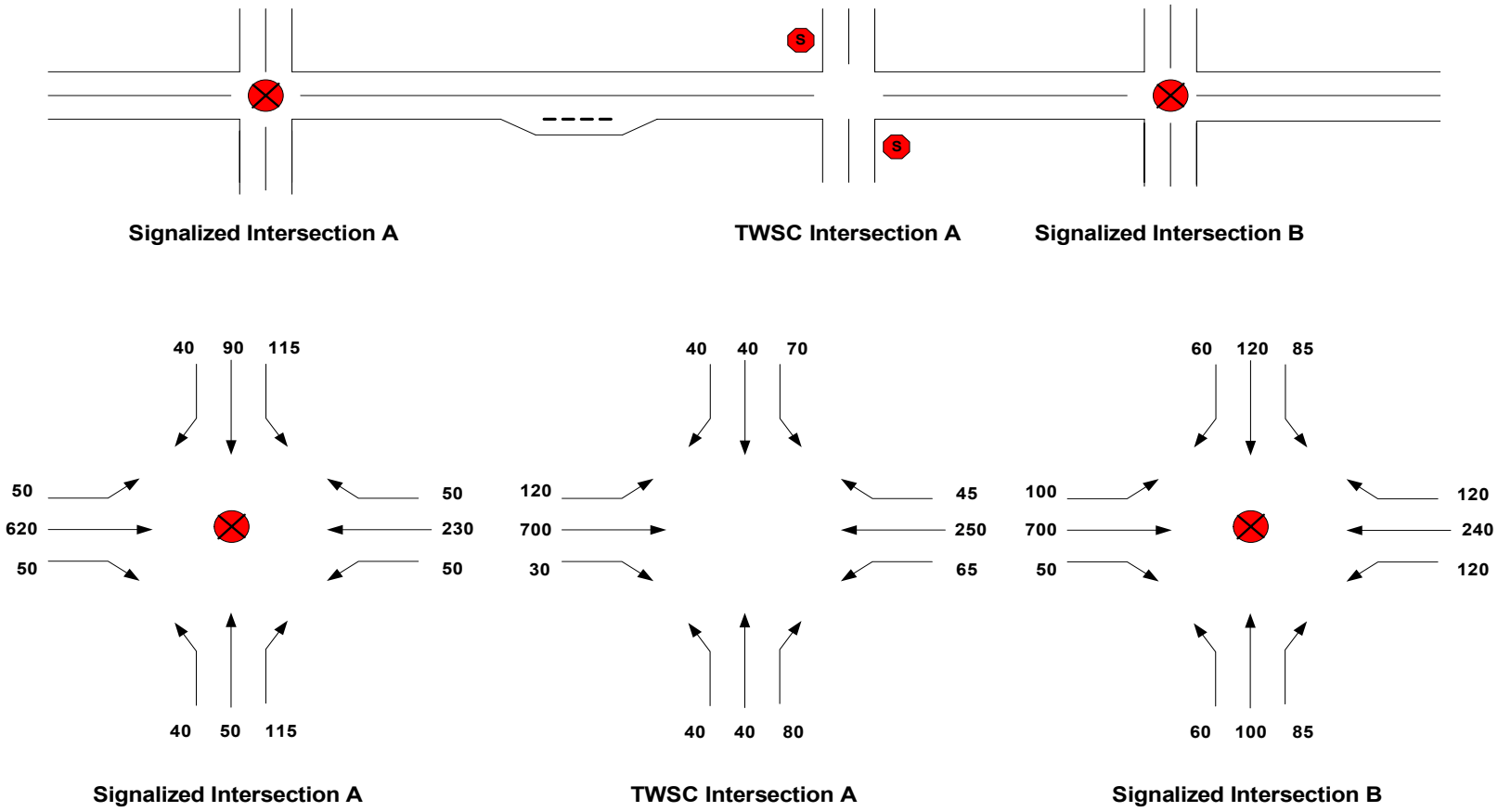


Figure 6-3. Traffic flow rates at three intersections

Step 1: Divide the facility into Segments

To a two-lane highway with two isolated signalized intersections, one two-way stop-controlled intersection, and a passing lane, the signalized and unsignalized intersections affect the operations of the upstream and downstream two-lane highway segments; the added passing lane also has effects on the downstream two-lane highway segment. The first step is to break the facility into uninterrupted and interrupted flow segments. Each segment has the homogenous features. Figure 6-2 (B) shows the division of a two-lane highway with two isolated signalized intersections, one two-way stop-controlled intersection, and one passing lane. The whole two-lane highway facility is divided into twelve segments. These segments are:

- Segment 1: the unaffected two-lane highway segment. It is located upstream of the first signalized intersection. This segment is not affected by the downstream signalized intersection.
- Segment 2: the influence area of the first signalized intersection. It is composed of not only its own actual length, but also the deceleration distance, stopping distance and acceleration distance, which is needed for the vehicles through the first signalized intersection.
- Segment 3: the affected downstream two-lane highway segment. It is located downstream of the first signalized intersection, and affected by the upstream signalized intersection. Potential operational effects on this segment are produced by the first signalized intersection and traffic flows coming from the cross street at the first signalized intersection. Note this segment length does not include the acceleration length of the first signal influence area.
- Segment 4: the unaffected two-lane highway segment, it is located downstream of the first signalized intersection. This segment is not affected by the upstream signalized intersection, and the downstream passing lane.
- Segment 5: the passing lane. The passing lane length should include the length of the lane addition and lane drop tapers.
- Segment 6: the affected downstream segment. It is the downstream segment of the passing lane but within its effective length.

- Segment 7: the unaffected two-lane highway segment. It is located upstream of the two-way stop-controlled intersection. This segment is not affected by the upstream passing lane and the downstream unsignalized intersection.

- Segment 8: the influence area of the two-way stop-controlled intersection. Its length is longer than the actual length of this unsignalized intersection.

- Segment 9: the unaffected two-lane highway segment, it is located upstream of the second signalized intersection, and the downstream of the TWSC intersection. This segment is not affected by the upstream TWSC intersection and the downstream signalized intersection.

- Segment 10: the influence area of the second signalized intersection. It is composed of not only its own actual length, but also the deceleration lengths, stopping lengths and acceleration lengths, which is needed for the vehicles through the second signalized intersection.

- Segment 11: the affected downstream segment. It is located downstream of the second signalized intersection, and affected by the upstream signalized intersection. Potential operational effects on this segment are produced by the second signalized intersection and traffic flows coming from the cross street at the second signalized intersection. Note this segment length does not include the acceleration length of the second signal influence area.

- Segment 12: the unaffected two-lane highway segment. It is located downstream of the second signalized intersection. This segment is not affected by the upstream signalized intersection.

These twelve lengths added up to the total length of the analysis facility.

Step 2: Determine segment lengths

Because there is not a left-turn bay present at the first signalized intersection, the upstream effective length of the first signalized intersection, Len_{eff_up1}, is calculated using Equation 4-18.

$$
\begin{aligned}
Len_{eff\_up1} &= 3074.49 + 5.89 \times (V/100)^2 - 440.00 \times DFactor \\
&\quad + 1.69 \times C - 7336.59 \times g\_C + 4758.52 \times (g\_C)^2 \\
&\quad + 1171.01 \times (V/100) \times (\%LT)^2 \\
&= 3074.49 + 5.89 \times (720/100)^2 - 440.00 \times 0.6 \\
&\quad + 1.69 \times 90 - 7336.59 \times 0.6 + 4758.52 \times (0.6)^2 \\
&\quad + 1171.01 \times (720/100) \times (50/720)^2 \\
&= 620 \text{ ft}
\end{aligned}
$$

The acceleration length can be determined using the linearly-decreasing acceleration model, or obtained from the reference table in *Transportation and Traffic Engineering Handbook*, which is presented in Chapter 4, Table 4-1. Here the acceleration distance uses the value from this reference table.

$L_A = 574$ ft

The length of Segment 2, L_2, is:

$$
\begin{aligned}
L_2 &= L_{eff\_up1} + L_A \\
&= 620 + 574 = 1194 \text{ ft} = 0.23 \text{ mi}
\end{aligned}
$$

The length of the unaffected two-lane highway segment upstream of the first signalized intersection, L_1, is calculated by subtracting the upstream part of the first signalized intersection influence area from length of the two-lane highway upstream of the first signalized intersection.

$$
\begin{aligned}
L_1 &= L_{UFS} - L_{eff\_up1} \\
&= 3.0 - 620/5280 = 2.88 \text{ mi}
\end{aligned}
$$

The length of the affected downstream segment of the first signalized intersection, L_3, is calculated by subtracting the acceleration length of the first signal influence area from the downstream effective length of the first signalized intersection. The downstream effective length, Len_{eff_down1} is calculated using Equation 4-25.

$Len_{eff\_down1} = 2.218584 - 0.122942 \times (V/100)$

$= 2.218584 - 0.122942 \times (850/100)$

$= 1.17$ mi

The length of Segment 3, L_3, is:

$$L_3 = Len_{eff\_down1} - L_A$$
$$L_3 = 1.17 - (574/5280) = 1.06 \text{ mi}$$

Segment 4 is located in the downstream of the first signalized intersection, and the upstream of the passing lane. The length of Segment 4 can be calculated by subtracting the total length of Segment 1, 2 and 3 from the total upstream length of the passing lane as shown in Equation 6-1.

$$L_4 = (L_{UFS} + L_{UP}) - (L_1 + L_2 + L_3) \tag{6-1}$$

Where:

L_{UFS}: upstream length of the first signalized intersection, mi, and

L_{UP}: the length between the stop line of the first signalized intersection and the beginning point of the passing lane, mi

So the length of Segment 4, L_4, is:

$$L_4 = (3.0 + 4.0) - (2.88 + 0.23 + 1.06)$$
$$= 2.83 \text{ mi}$$

The length of the passing lane, L_5, includes the lengths of the lane addition and lane drop tapers. A typical passing lane is shown in Figure 6-4.

In this analysis, the passing lane length, L_5, is:

$L_5 = 1.5$ mi

The length of the downstream highway segment within the effective length of the passing lane, L_6, is determined from Table 5-4. From this table, the downstream length of roadway affected by the upstream passing lane, L_6, is:

L_6 = 1.7 mi

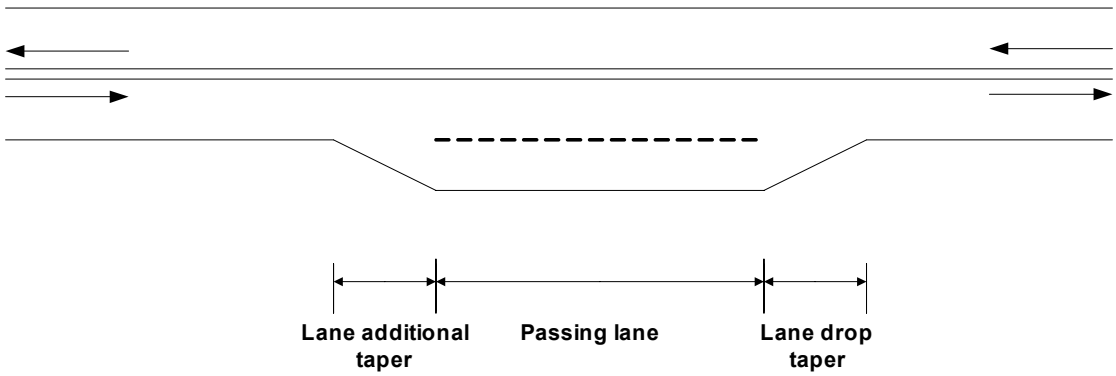


Figure 6-4. A view of a typical passing lane

Segment 8 is a two-way stop-controlled intersection. The effect of a shared lane on the major-street approach where left-turn vehicles may block Rank 1 through or right-turning vehicles can be significant. While the delayed Rank 1 vehicles are discharging from the queue formed behind a left-turning vehicle, they impede lower-ranked movements with which they conflict. Field observations have shown that such a blockage effect is very small [1]. CORSIM simulation runs have shown the same results. In this methodology, the interrupted flow unsignalized intersection segments, "unsignalized intersection influence are," extend 0.1 miles in length centered on the midpoint of the crossing facility. The delay for this influence area is determined by the delay to Rank 1 vehicles. The length of the two-way stop-controlled intersection influence area, L_8, is:

L_8 = 0.1 mi

Segment 7 is an unaffected two-lane highway segment, located downstream of the passing plan, and upstream of the TWSC intersection. The length of Segment 7 can be calculated by subtracting the total length of Segment 1, Segment 2, Segment 3, Segment

4, Segment 5, Segment 6, and half of Segment 8 from the total upstream length of the TWSC intersection as shown below.

$$L_7 = (L_{US} + L_{UP} + L_{PL} + L_{UTWSC}) - (L_1 + L_2 + L_3 + L_4 + L_5 + L_6 + \frac{L_8}{2})$$
$$= (3.0 + 4.0 + 1.5 + 4.5) - (2.88 + 0.23 + 1.06 + 2.83 + 1.50 + 1.70 + \frac{0.1}{2})$$
$$= 2.75 \text{ mi}$$

The length of the second signalized intersection influence area, L_{10}, is also calculated as the sum of its upstream effective length and acceleration length. Because there is a left-turn bay present at the second signalized intersection, the upstream effective length of the second signalized intersection, Len_{eff_up2}, is calculated using Equation 4-17.

$$\begin{aligned} Len_{eff\_up2} &= 43.2463 + 4.2688 \times (V/100)^2 + 5.2178 \times Cycle \\ &\quad - 57.3041 \times (V/100) \times \%LT - 5.2444 \times Cycle \times g\_C \\ &= 43.2463 + 4.2688 \times (850/100)^2 + 5.2178 \times 90 \\ &\quad - 57.3041 \times (850/100) \times (100/850) - 5.2444 \times 90 \times 0.7 \\ &= 434\text{ft} \end{aligned}$$

L_A = 574 ft

The length of Segment 10, L_{10}, is:

$$\begin{aligned} L_{10} &= L_{eff\_up2} + L_A \\ &= 434 + 574 = 1008\text{ft} = 0.19 \text{ mi} \end{aligned}$$

Segment 9 is an unaffected two-lane highway segment, located downstream of the two-way stop-controlled intersection and upstream of the signalized intersection. The length of Segment 9 can be calculated by subtracting the total length of Segment 1, Segment 2, Segment 3, Segment 4, Segment 5, Segment 6, Segment 7, Segment 8, and

the upstream effective length of the second signal influence area from the total upstream length of the second signalized intersection. That is,

$$\begin{aligned} L_9 &= (L_{UFS} + L_{UP} + L_{PL} + L_{UTWSC} + L_{USS}) \\ &\quad - (L_1 + L_2 + L_3 + L_4 + L_5 + L_6 + L_7 + L_8 + Len_{eff\_up2}) \\ &= (3.0 + 4.0 + 1.5 + 4.5 + 3) \\ &\quad - (2.88 + 0.23 + 1.06 + 2.83 + 1.50 + 1.70 + 2.75 + 0.1 + 434/5280) \\ &= 2.97 \text{ mi} \end{aligned} \tag{6-2}$$

The length of the affected downstream segment of the signalized intersection, L_{11}, is calculated by subtracting the acceleration length of the second signal influence area from the downstream effective length. The downstream effective length, Len_{eff_down2}, is calculated using Equation 4-25.

$$Len_{eff\_down2} = 2.218584 - 0.122942 \times (V/100)$$

$$= 2.218584 - 0.122942 \times (870/100)$$

$$= 1.15 \text{ mi}$$

The length of Segment 11, L_{11}, is:

$$L_{11} = Len_{eff\_down2} - L_A$$
$$L_{11} = 1.15 - (574/5280) = 1.04 \text{ mi}$$

Segment 12 is located downstream of the second signalized intersection. The length of Segment 12 can be calculated by subtracting the downstream effective length of the second signalized intersection from the distance between the stop line of the second signalized intersection and the ending point of the analyzed facility. The length of Segment 12 can be calculated using Equation 6-3.

$$L_{12} = L_{DSS} - Len_{eff\_down2} \tag{6-3}$$

Where:

L_{DSS} : the downstream length of the second signalized intersection from the stop line of

the second signalized to the ending point of the facility, mi

The length of Segment 12, $L_{12,}$ is:

$$L_{12} = L_{DSS} - Len_{eff\_down2}$$
$$= 4.0 - 1.15 = 2.85 \text{ mi}$$

Step 3: Calculate the free-flow speed

$$FFS = BFFS - f_{LS} - f_A$$
$$= 60 - 0 - 1.3$$
$$= 58.7 \text{ mi/h}$$

The above equation is Equation 20-2 in the HCM 2000. The f_{LS} and f_A values are from Exhibits 20-5 and 20-6, respectively, in Chapter 20 of the HCM 2000.

Step 4: Determine the average travel speed on the unaffected two-lane segments (*Segment 1, Segment 4, Segment 7, Segment 9, and Segment 12*)

Use the HCM 2000 methodology to calculate the average travel speeds on the unaffected two-lane segments. The average travel speeds of these basic two-lane segments are summarized in the Table 6-2.

Table 6-2. Average travel speed for the unaffected two-lane segments

	Average Travel Speed (mi/h)
Segment 1	ATS_1 = 47.0 mi/h
Segment 4	ATS_4 = 46.9 mi/h
Segment 7	ATS_7 = 46.9 mi/h
Segment 9	ATS_9 = 46.8 mi/h
Segment 12	ATS_{12} = 46.1 mi/h

Step 5: Determine the average travel speed of the affected downstream segments of the signalized intersection (Segment 3 and Segment 11)

For the affected downstream segment of the signalized intersection, an adjustment factor for the effect of a signalized intersection on average travel speed will be applied to the average travel speed without a signalized intersection to compute the average travel

speed on the affected downstream segment of the signalized intersection. The adjustment factors can be obtained from Table 5-3 based on the traffic flow rate. The adjustment factor for Segment 3 can be interpolated as

$$f_{ATS} = 1.8 + (850 - 660) \times \frac{2.281 - 1.800}{880 - 660} = 2.215 \text{ mi/h}$$

So the average travel speed of Segment 3 is

$$\begin{aligned} ATS_3 &= ATS_4 - f_{ATS} \\ &= 46.9 - 2.215 = 44.7 \text{ mi/h} \end{aligned}$$

The adjustment factor for Segment 11 can be interpolated as

$$f_{ATS} = 1.8 + (870 - 660) \times \frac{2.281 - 1.800}{880 - 660} = 2.259 \text{ mi/h}$$

So the average travel speed of Segment 11 is

$$\begin{aligned} ATS_{11} &= ATS_{12} - f_{ATS} \\ &= 46.1 - 2.259 = 43.8 \text{ mi/h} \end{aligned}$$

The average travel speeds of these affected downstream segments of the signalized intersection are summarized in Table 6-3.

Table 6-3. Average travel speed for the affected downstream two-lane segments

Segment No.	**Average Travel Speed (mi/h)**
Segment 3	ATS_3 = 44.7 mi/h
Segment 11	ATS_{11} = 43.8 mi/h

Step 6: Determine the average travel speed within the passing lane and its affected downstream segment (Segment 5 and Segment 6)

Equation 20-21 in the HCM 2000 is used to compute the average travel speed with the passing lane and its affected downstream two-lane highway. The average travel speeds of Segment 5 and Segment 6 are summarized in Table 6-4.

Table 6-4. Average travel speed for the affected passing lane segments

Segment No.	Average Travel Speed (mi/h)
Segment 5	ATS_5 = 46.1 mi/h
Segment 6	ATS_6 = 46.1 mi/h

Step 7: Determine the control delay of the signal influence areas (Segment 2 and Segment 10)

Use the HCM 2000 analysis methodology to calculate the control delay (Chapter 16). The control delays of the signalized intersection influence areas are summarized in Table 6-5.

Table 6-5. Control delays at signalized intersections

Segment No.	Control Delay (sec/veh)
Segment 2	D_2 = 17.8 sec/veh
Segment 10	D_{10} = 16.4 sec/veh

Step 8: Determine the main approach delay at the Two-Way Stop-Controlled Intersection (Segment 8)

1. Calculate the critical gap for left-turn movement from the major street, $t_{c,1}$

$$t_{c,x} = t_{c,base} + t_{c,HV}P_{HV} + t_{c,G}G - t_{c,T} - t_{3,LT}$$
$$t_{c,1} = 4.1 + 1.0(0.05) + 0 - 0 - 0 = 4.15\,\text{s}$$

2. Calculate the follow-up time for the left-turn movement from the major street, $t_{f,1}$

$$t_{c,x} = t_{f,base} + t_{f,HV}P_{HV}$$
$$t_{c,1} = 2.2 + 1.0(0.05) = 2.25\,\text{s}$$

3. Calculate the potential capacity of the left-turn movement from the major street, $c_{p,1}$

$$c_{p,x} = v_{c,x}\frac{e^{-v_{c,x}t_{c,x}/3600}}{1-e^{-v_{c,x}t_{f,x}/3600}}$$
$$v_{c,1} = v_5 + v_6 + v_{16} = 250 + 45 = 295\ \text{veh}$$
$$c_{p,1} = 295\frac{e^{-295\times4.15/3600}}{1-e^{-295\times2.25/3600}} = 1247\ \text{veh/h}$$

4. Calculate the delay to major left-turn vehicle, $d_{M,LT}$

$$d_{MLT} = \frac{3600}{c_{m,x}} + 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right] + 5$$

$$d_{M,LT} = \frac{3600}{1247} + 900 \times 0.25\left[\frac{120}{1247} - 1 + \sqrt{\left(\frac{120}{1247} - 1\right)^2 + \frac{\left(\frac{3600}{1247}\right)\left(\frac{120}{1247}\right)}{450 \times 0.25}}\right] + 5$$

$$= 8.19 \text{ s}$$

5. Calculate the probability of queue-free state for left-turning movement from the major approach assuming an exclusive left-turn lane on the major street, $p_{o,1}$

$$p_{o,j} = 1 - \frac{v_j}{c_{m,j}}$$

$$p_{o,1} = 1 - \frac{v_1}{c_{m,1}} = 1 - \frac{120}{1247} = 0.904$$

6. Calculate the proportion of through vehicles and right-turn vehicles on the major approach not blocked,

$$p^*_{o,j} = 1 - \frac{1 - p_{o,j}}{1 - \left(\frac{v_{j1}}{s_{j1}} + \frac{v_{j2}}{s_{j2}}\right)}$$

$$p^*_{o,1} = 1 - \frac{1 - 0.904}{1 - \left(\frac{700}{1800} + \frac{30}{1800}\right)} = 0.839$$

7. Calculate the delay to through vehicles and right-turn vehicles on the major approach, $d_{T,RT}$

$$d_{T,RT} = (1 - p^*_{0,j})d_{M,LT}$$

$$d_{T,RT} = (1 - 0.839) \times 8.194 = 1.32 \text{ s}$$

8. Calculate control delay on the major approach, d_A

$$d_A = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l}$$

$$d_A = \frac{120 \times 8.194 + 1.319 \times 730}{120 + 730} = 2.30\,\text{s}$$

Step 9: Determine the performance measure of the entire facility.

The delay at Segment 1, D_1, is:

$$L_1 = 2.88\,\text{mi}$$
$$S_1 = 47.0\,\text{mi/h}$$
$$FFS_1 = 58.7\,\text{mi/h}$$
$$D_1 = \frac{L_1}{S_1} - \frac{L_1}{FFS_1} = (\frac{2.88}{47.0} - \frac{2.88}{58.7})(3600) = 43.97\,\text{sec/veh}$$

The delay at Segment 2, D_2, is:

$$L_2 = 0.23\,\text{mi}$$
$$D_2 = 17.8\,\text{sec/veh}$$

The delay at Segment 3, D_3, is:

$$L_3 = 1.06\,\text{mi}$$
$$S_3 = 44.7\,\text{mi/h}$$
$$FFS_3 = 58.7\,\text{mi/h}$$
$$D_3 = \frac{L_3}{S_3} - \frac{L_3}{FFS_3} = (\frac{1.06}{44.7} - \frac{1.06}{58.7})(3600) = 20.36\,\text{sec/veh}$$

The delay at Segment 4, D_4, is:

$$L_4 = 2.83\,\text{mi}$$
$$S_4 = 46.9\,\text{mi/h}$$
$$FFS_4 = 58.7\,\text{mi/h}$$
$$D_4 = \frac{L_4}{S_4} - \frac{L_4}{FFS_4} = (\frac{2.83}{46.9} - \frac{2.83}{58.7})(3600) = 43.68\,\text{sec/veh}$$

The delay at Segment 5 and Segment 6 is:

$$L_5 = 1.50 \text{ mi}$$
$$L_6 = 1.70 \text{ mi}$$
$$S_5 = 46.1 \text{ mi/h}$$
$$S_6 = 46.1 \text{ mi/h}$$
$$FFS_5 = 58.7 \text{ mi/h}$$
$$FFS_6 = 58.7 \text{ mi/h}$$
$$D_{5,6} = \frac{L_5 + L_6}{S_{5,6}} - \frac{L_5 + L_6}{FFS_{5,6}} = (\frac{1.5 + 1.7}{46.1} - \frac{1.5 + 1.7}{58.7})(3600) = 53.64 \text{ sec/veh}$$

The delay at Segment 7 is:

$$L_7 = 2.75 \text{ mi}$$
$$S_7 = 46.9 \text{ mi/h}$$
$$FFS_7 = 58.7 \text{ mi/h}$$
$$D_7 = \frac{L_7}{S_7} - \frac{L_7}{FFS_7} = (\frac{2.75}{46.9} - \frac{2.75}{58.7})(3600) = 42.43 \text{ sec/veh}$$

The delay at segment 8 is

$$L_8 = 0.1 \text{ mi}$$
$$D_8 = 2.3 \text{ sec/veh}$$

The delay at Segment 9 is

$$L_9 = 2.97 \text{ mi}$$
$$S_9 = 46.8 \text{ mi/h}$$
$$FFS_9 = 58.7 \text{ mi/h}$$
$$D_9 = \frac{L_9}{S_9} - \frac{L_9}{FFS_9} = (\frac{2.97}{46.8} - \frac{2.97}{58.7})(3600) = 46.32 \text{ sec/veh}$$

The delay at segment 10 is:

$$L_8 = 0.19 \text{ mi}$$
$$D_8 = 16.4 \text{ sec/veh}$$

The delay at Segment 11 is:

$$L_9 = 1.04 \text{ mi}$$
$$S_9 = 43.8 \text{ mi/h}$$
$$FFS_9 = 58.7 \text{ mi/h}$$
$$D_9 = \frac{L_9}{S_9} - \frac{L_9}{FFS_9} = (\frac{1.04}{43.8} - \frac{1.04}{58.7})(3600) = 21.70 \text{ sec/veh}$$

The delay at segment 12 is:

$$L_{10} = 2.85 \text{ mi}$$
$$S_{10} = 46.1 \text{ mi/h}$$
$$FFS_{10} = 58.7 \text{ mi/h}$$
$$D_{10} = \frac{L_{10}}{S_{10}} - \frac{L_{10}}{FFS_{10}} = (\frac{2.85}{46.1} - \frac{2.85}{58.7})(3600) = 47.77 \text{ sec/veh}$$

Step 10: Determine the performance measure and LOS of the entire facility.

1. The total length of the facility

Length of Segment 1, L_1	2.88 miles
Length of Segment 2, L_2	0.23 miles
Length of Segment 3, L_3	1.06 miles
Length of Segment 4, L_4	2.83 miles
Length of Segment 5, L_5	1.50 miles
Length of Segment 6, L_6	1.70 miles
Length of Segment 7, L_7	2.75 miles
Length of Segment 8, L_8	0.10 miles
Length of Segment 9, L_9	2.97 miles
Length of Segment 10, L_{10}	0.19 miles
Length of Segment 11, L_{11}	1.04 miles
Length of Segment 12, L_{12}	2.85 miles
Total Length	$L_t = L_1 + L_2 + L_3 + L_4 + L_5 + L_6 + L_7 + L_8 + L_9 + L_{10} + L_{11} + L_{12}$ $L_t = 2.88 + 0.23 + 1.06 + 2.83 + 1.50 + 1.70 + 2.75 + 0.10$ $+ 2.97 + 0.19 + 1.04 + 2.85 = 20 \text{ miles}$

2. The total delay of the facility

Delay of Segment 1, D_1	43.97 sec
Delay of Segment 2, D_2	17.80 sec
Delay of Segment 3, D_3	20.36 sec
Delay of Segment 4, D_4	43.68 sec
Delay of Segment 5, D_5	53.64 sec

Delay of Segment 6, D_6	
Delay of Segment 7, D_7	42.43 sec
Delay of Segment 8, D_8	2.30 sec
Delay of Segment 9, D_9	46.32 sec
Delay of Segment 10, D_{10}	16.40 sec
Delay of Segment 11, D_{11}	21.70 sec
Delay of Segment 12, D_{12}	47.77 sec
Total Delay	$D_t = D_1 + D_2 + D_3 + D_4 + D_5 + D_6 + D_7 + D_8 + D_9 + D_{10} + D_{11} + D_{12}$ $D_t = 43.97 + 17.80 + 20.36 + 43.68 + 53.64 + 42.43 + 2.30 + 46.32 + 16.40 + 21.70 + 47.77 = 356\text{sec/veh}$

3. Calculate the total travel time of the facility based on the free flow speed

$$T_{tFFS} = \frac{L}{FFS} = \frac{20}{58.7}(3600) = 1227 \text{ sec/veh}$$

4. Calculate the percent time-delayed of the facility

$$PTD = \frac{\sum_{H,S}(D_H + D_S)}{\sum_{H,S}(\frac{L_H}{FFS_H} + \frac{L_S}{FFS_S})} = \frac{D_T}{\frac{L_t}{FFS}}$$

$$= \frac{356}{1227} = 29.02\%$$

From Table 5-8, this value of percent time-delayed gives an LOS value of ‘D’. By comparison, if the two signalized intersections and one Unsignalized intersection were removed, an average travel speed of 46.9 mph would be estimated with the HCM 2000 Chapter 20 methodology, which would result in an LOS of ‘C’, again using the criteria in Exhibit 20-2 of the HCM 2000 (for *ATS* only). So in this case, the LOS is one grade worse due to the presence of the signals. Even though the operations of the signals are still relatively good, with borderline LOS B/C (from LOS criteria in HCM 2000 Exhibit

16-2), they are adding just enough of a penalizing effect to reduce the LOS over the length of the facility.

CHAPTER 7
SUMMARY AND RECOMMENDATIONS

This chapter summarizes the dissertation, discusses the findings, and outlines future research directions. Developing a method for the assessment of traffic operations on two-lane highway facilities has proven to be a challenging and complex issue. Despite the progress of this dissertation, many potential research extensions deserve consideration.

7.1 Summary

This dissertation makes some contributions in the area of traffic operations assessment for the highway system. In this dissertation, the focus is on the development of a method for the assessment of traffic operations on two-lane highways that include occasional traffic signals and TWSC intersections. This particular type of facility is not currently addressed by any of the methodology chapters within the HCM 2000.

The features of the methodology developed in this dissertation are summarized as follows:

- The method maintains some fidelity to the HCM 2000 by using the existing methodologies for the two-lane highway segments and signalized intersections. For the two-lane highway methodology (HCM Chapter 20), the method for calculation of average travel speed (ATS) is utilized. The average travel speed of two-lane highway segments with a passing lane is also calculated using the methodology of the HCM Chapter 20. For signalized intersections (HCM Chapter 16), the current method for the calculation of control delay is utilized. For unsignalized intersections, the method for the calculation of delay to the vehicles with highest priority is utilized (HCM Chapter 17).

- In this method, a common time/delay based service measure is used for each segment and point of the entire facility. The service measure values for each segment are

aggregated to obtain an estimate of the service measure value for the entire facility. The LOS of the entire facility is determined by the aggregated service measure. The unified facility-wide service measure not only avoids the anomalies caused by the application of multiple service measures, but also provides a MOE describing traffic operations in terms discernible by motorists from the viewpoint of the entire facility. By using a time/delay based service measure, this method is similar to the current HCM methodology for urban streets with signalized intersection. For transportation agencies looking to analyze the impacts of adding a lane (or lanes) to a two-lane highway, along with adding some signalized intersections, thus possibly changing the classification to an urban arterial in some segments, this will provide for consistency in the LOS analyses.

- The primary contribution of the work in this dissertation is the development of equation and analytical relationships to determine the upstream and downstream boundary points that define the transition between the basic two-lane highway segment and the effective influence area of the signalized intersection and the TWSC intersection as well as the corresponding method for calculating an overall facility service measure based on percent time-delayed and then translating to a level of service grade.

- In the proposed segmentation method, a signalized intersection is not regarded as an isolated point. The interaction between a signalized intersection and upstream and downstream highway segments are taken into account by considering the signalized intersection as a segment and determining its effective length. The components of the signal influence area include deceleration distance, stopping distance, and acceleration distance, which are consistent with those of control delay defined in the HCM 2000–deceleration delay, stop delay, and acceleration delay. The segment delay time for the signalized intersection influence area is determined by the intersection control delay. Two regression models are developed to determine the relationship between the length of signal influence area and corresponding traffic, roadway, and signal characteristics. One is for the signalized intersection with an exclusive left-turn lane; the other is for without a left-turn lane.

- In determining the length of the affected downstream segment, the parameter, EPF was used to represent the effects of a signalized intersection on the downstream two-lane highway segment. Shifted negative exponential and composite distributions were used to calculate the parameter of EPF. Under very low volume conditions, the shifted negative exponential distribution was used to define the time headway distribution for the EPF calculation; under higher volume conditions, the composite distribution was used. The calculated values of EPF were input into the TWOPAS simulation model to determine the length of the affected downstream segment. Traffic volume is the most significant factor for the affected downstream length of a signalized intersection and this relationship was determined to be inversely proportional.

- This structure easily allows for including other two-lane highway segment types (e.g., passing lanes, which already has a method for calculating ATS in the HCM) and

other intersection types (e.g., unsignalized, which also uses a control delay service measure). Meanwhile, this methodological approach provides a balance between one that is intuitive and logical from both an analytical and traveler's perspective and complexity.

- The method of analysis for multilane highways could be modified to fit into the proposed framework such that combinations of two-lane highway segments, multilane highway segments, and occasional traffic signals can be analyzed as an overall facility.

Personnel with the FDOT System Planning Office have indicated that a facility-based evaluation methodology for two-lane highways would be much more useful to them than just the individual segment and point analysis methodologies. With the continual increase in growth and development, combinations of various types of two-lane highway segments, multilane highway segments, signalized intersections, and unsignalized intersections are also increasing; as such, transportation planners and engineers need tools to perform more comprehensive roadway facility evaluations. This is consistent with the fact that drivers typically evaluate the quality of their trip over its entire length, not just in separate pieces.

This type of methodology will be particularly useful to those managing the transportation system in more rural areas. These areas are often faced with decisions on whether to add a signal along a two-lane highway through a small town, or go to a multilane configuration. Finally, the methodology has been incorporated into the existing HIGHPLAN software program. Currently, HIGHPLAN only allows for the input of data specific to one segment. The updated version incorporating the results from this research includes a segment and intersection data input screen to allow for the calculation of a facility LOS value.

7.2 Further Research

In this research, the regression models for determining the upstream effective length of a signalized intersection were developed using the data obtained from the CORSIM simulation program. Although the coefficients of the regression models show logical relationships among all the variables considered, the model validation was somewhat limited. On analyzing the potential effect of a signalized intersection on the downstream two-lane highway operation, the EPF parameter was used to represent the effects of a signalized intersection. Although the EPF calculation takes into account many factors such as the signal timing plan, travel speed, and the time headway distribution, the single EPF value seems overstressed to some degree. A simulation program capable of simulating the combination of two-lane highway segments and signalized intersections (one was still not available at the time of publication of this dissertation) would provide the capability to perform some validation of the model proposed in this research. Obviously, validation with field data is the most desirable option, and it is hoped that this could be done in a future study.

With regard to the two-way stop-controlled intersection incorporated in this methodology, additional study on the full effects of these types of intersections on two-lane highways should be investigated, particularly as it relates to driver behavior in reacting to left-turning vehicles from the major highway. Additionally, all-way stop controlled intersections are sometimes used in small towns instead of a signal, and even roundabouts to a lesser extent. These intersection types should be considered for inclusion into the methodology as part of a future project.

With regard to the application of this methodology to multilane highways, multilane highway segments and freeway segments currently share the same service

measure, density. Given the similarity in operational features of these segments, this makes some sense. However, in some respects, multilane highways that include the presence of signalized intersections have more in common with signalized arterials or two-lane highways (with occasional signals) than freeways. Since freeways, by definition, are always uninterrupted, it becomes more difficult to make comparisons between an interrupted facility and one that includes occasional interruptions.

Combining multilane highway segments, that use density for the service measure, with signalized intersections, that use delay as the service measure, poses the same challenge as that for two-lane highways and signalized intersections. That is, the service measures for each facility type are somewhat disparate, but even more so in the case of multilane highways and signals, as the density measure has no time component to it, whereas the signal delay measure is strictly time-based.

As was done for this two-lane facility analysis methodology, a speed/delay measure can be implemented for multilane highways. However, as indicated by current empirical evidence, average speeds on multilane highways (as well as freeways) are relatively constant up to fairly high flow rates, and thus LOS based upon speed does not reflect the likely discomfort experienced by travelers for increasing flow rates. Density, on the other hand, always increases with increasing flow rate (for under-saturated conditions) and thus reflects traveler discomfort more adequately. To some extent, the use of speed for two-lane highways has the same limitations (compared to percent time-spent-following); however, speed is more readily impacted by flow rate on two-lane highways due to the need to use the oncoming lane for passing maneuvers.

Percent time-spent-following may be more applicable than speed on multilane highways, as it correlates better with increasing density (decreasing headways) at lower volumes. Although this measure has a time component to it, it would still require some manipulation to be compatible with the delay measure of signalized intersections. Alternatively, the signal delay measure might be manipulated to yield a comparable *PTSF* value. While these manipulations may also have been possible for the two-lane highway facility methodology, it should be noted again that one of the recognized drawbacks of the *PTSF* measure was its potential lack of applicability to certain types of two-lane highways. Furthermore, these manipulations of either the *PTSF* measure or the signal delay measure would potentially lead to a less intuitive level of service methodology. Further research needs to be done on the application of the methodological approach described in this dissertation to multilane highways facilities.

In this dissertation, a LOS threshold table for the two-lane facility is presented based on the service measure, percent time-delayed. The threshold value selection is guided by the concept that adding a signalized intersection along a two-lane highway should not result in an improvement to the LOS that would be estimated for an equivalent two-lane highway with no signalized intersection(s) present. Nonetheless, the selection of the threshold values was still somewhat arbitrary. Ideally, the selected thresholds should be chosen such that they correspond to actual drivers' level of satisfaction with the operating conditions for the given level of service value. Future research that directly investigates driver satisfaction on these facilities under operating conditions should be conducted to ensure threshold values are truly appropriate.

APPENDIX A
PASSING PERCENTAGE AND AVERAGE TRAVEL SPEED ON TWO-LANE HIGHWAYS

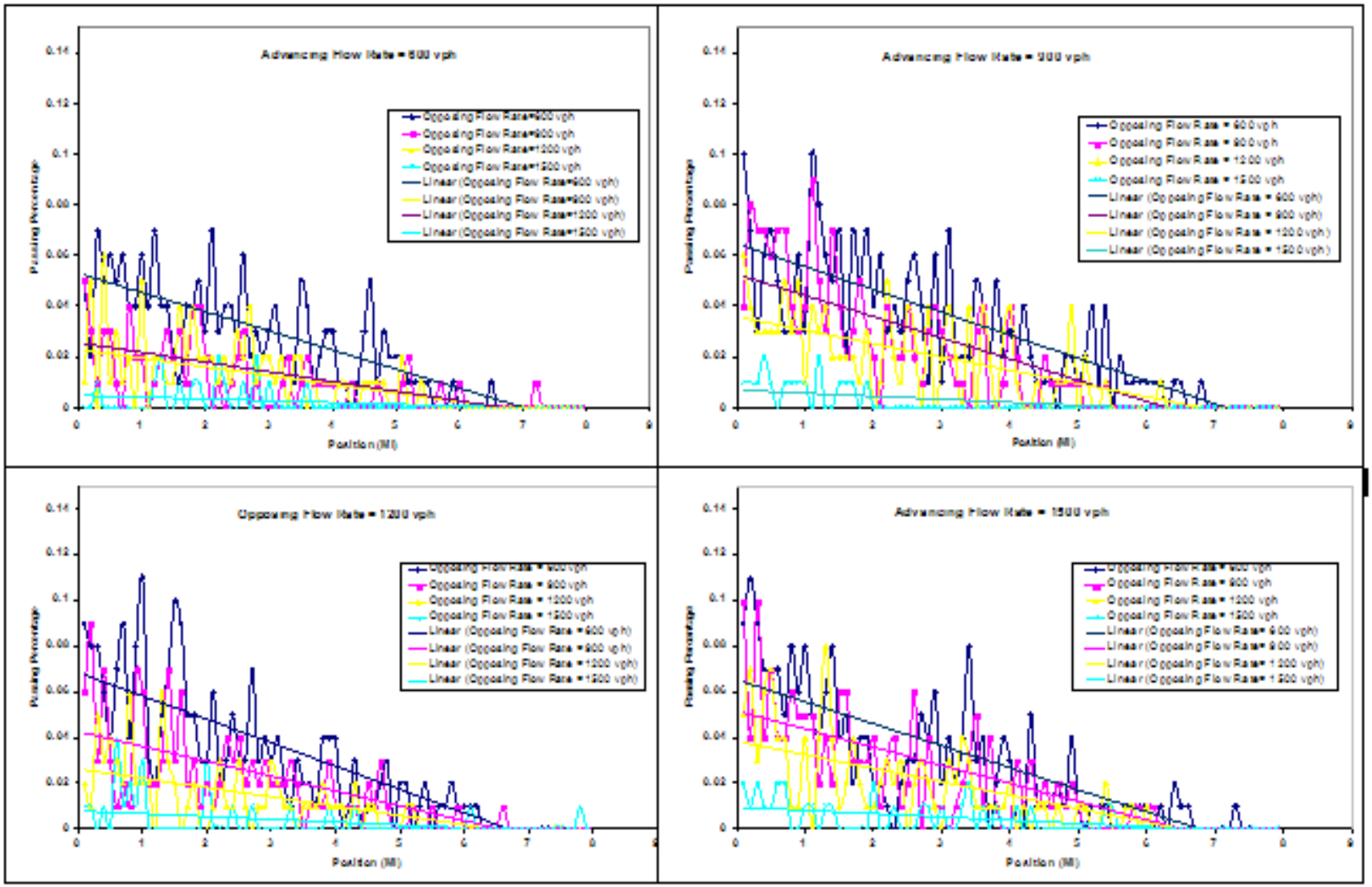


Figure A.1. Passing percentage for various levels of advancing and opposing traffic volume on two-lane highways

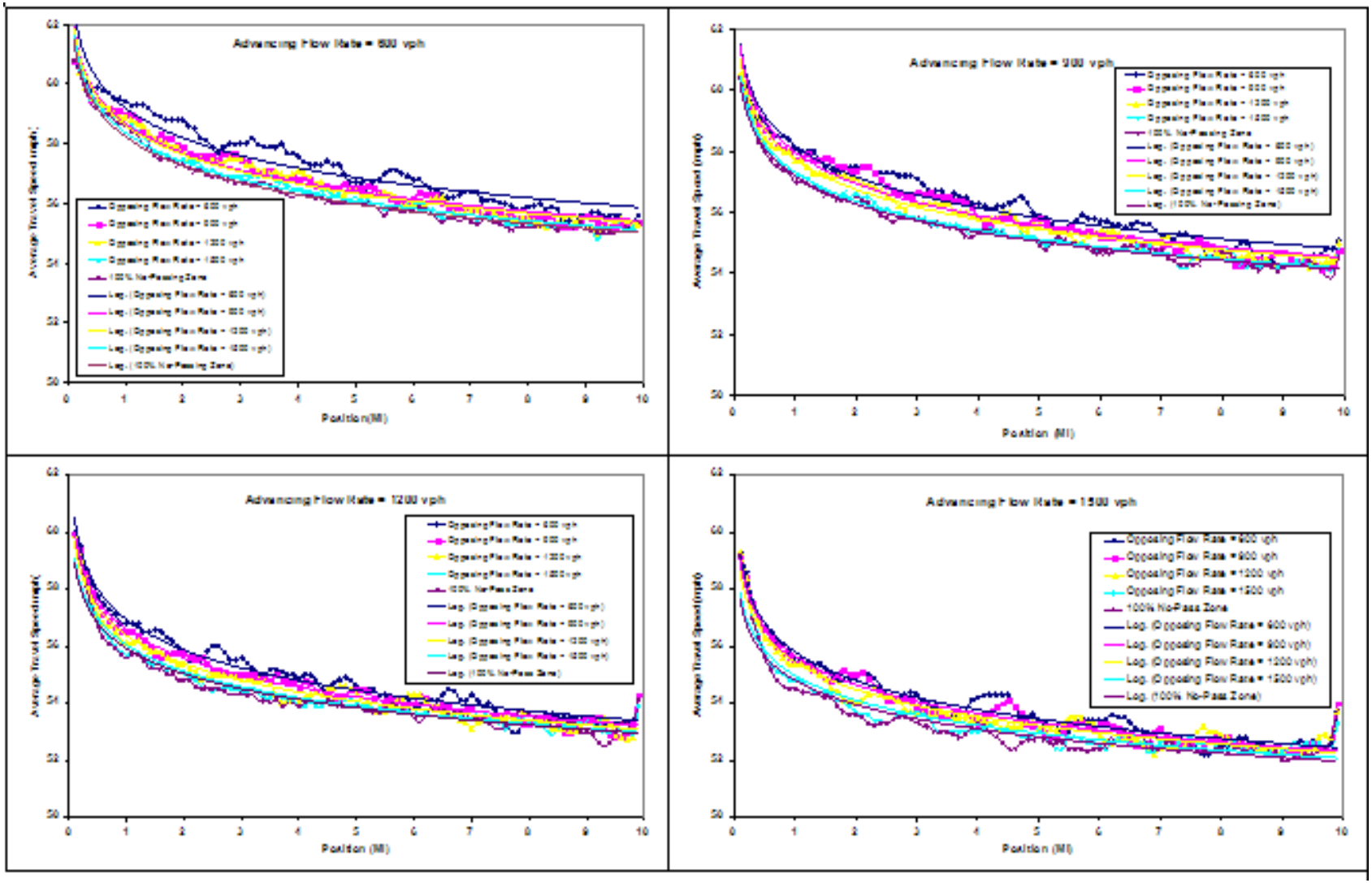


Figure A-2. Average travel speed for various levels of advancing and opposing traffic volume on two-lane highways

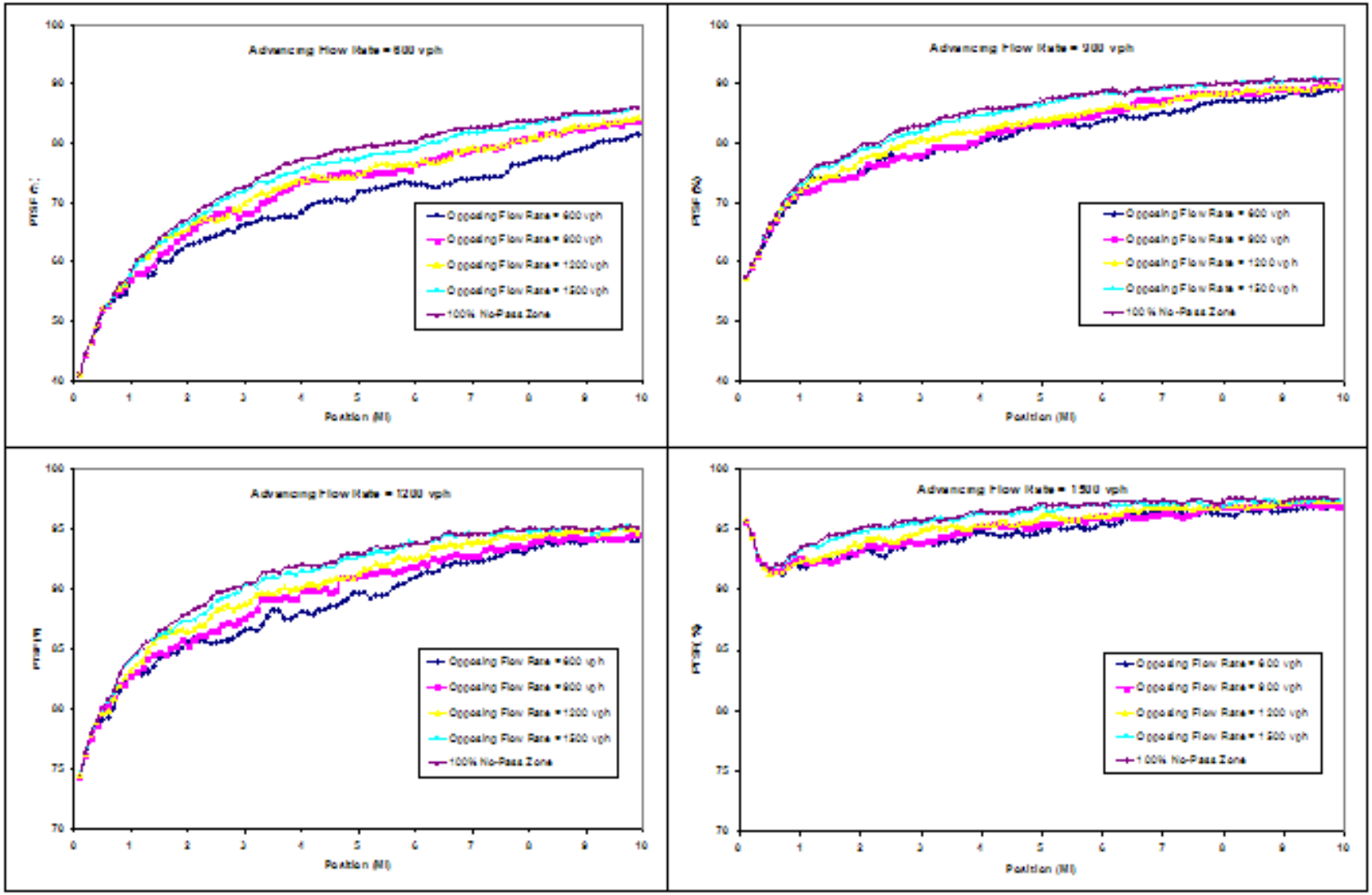


Figure A-3. Percent time-spent-following for various levels of advancing and opposing traffic volume on two-lane highways

APPENDIX B
HEADWAY DISTRIBUTION

Table B-1. Time headway distribution (traffic flow rate = 900 veh/h ~ 1140 veh/h)

	Vehicles Not in Platoons						Platoon Vehicles				Composite Distribution	
t	$t-\alpha$	$\bar{t}-\alpha$	$\bar{t}-\alpha$	$e^{-\frac{(t-\alpha)}{(\bar{t}-\alpha)}}$	100%	P	z	P(t<z)	100%	1-P	(P)	(F)
0.0	0.0	4				0.0000	-2.00	0.0228	0.0440	0.0251	0.0251	33
0.5	0.0	4				0.0000	-1.50	0.0668	0.0919	0.0525	0.0525	69
1.0	0.0	4				0.0000	-1.00	0.1587	0.1500	0.0857	0.0857	113
1.5	0.0	4	0.000	1.0000	0.1175	0.0504	-0.50	0.3087	0.1913	0.1093	0.1597	211
2.0	0.5	4	0.125	0.8825	0.1037	0.0444	0.00	0.5000	0.1913	0.1093	0.1538	203
2.5	1.0	4	0.250	0.7788	0.0915	0.0392	0.50	0.6913	0.1500	0.0857	0.1249	165
3.0	1.5	4	0.375	0.6873	0.0808	0.0346	1.00	0.8413	0.0919	0.0525	0.0871	115
3.5	2.0	4	0.500	0.6065	0.0713	0.0305	1.50	0.9332	0.0440	0.0251	0.0557	74
4.0	2.5	4	0.625	0.5353	0.0629	0.0270	2.00	0.9772	0.0166	0.0094	0.0364	48
4.5	3.0	4	0.750	0.4724	0.0555	0.0238	2.50	0.9938	0.0049	0.0028	0.0266	35
5.0	3.5	4	0.875	0.4169	0.0490	0.0210	3.00	0.9987	0.0013	0.0007	0.0217	29
5.5	4.0	4	1.000	0.3679	0.0432	0.0185	3.50	1.0000			0.0185	24
6.0	4.5	4	1.125	0.3247	0.0381	0.0163					0.0163	22
6.5	5.0	4	1.250	0.2865	0.0337	0.0144					0.0144	19
7.0	5.5	4	1.375	0.2528	0.0297	0.0127					0.0127	17
7.5	6.0	4	1.500	0.2231	0.0262	0.0112					0.0112	15
8.0	6.5	4	1.625	0.1969	0.0231	0.0099					0.0099	13
8.5	7.0	4	1.750	0.1738	0.0204	0.0088					0.0088	12
9.0	7.5	4	1.875	0.1534	0.0180	0.0077					0.0077	10
9.5	8.0	4	2.000	0.1353	0.1353	0.0580					0.0580	77

$\bar{t}_{NP} = 5.5\,\text{sec}, \alpha = 1.5\,\text{sec}, s_{NP} = 4.0\,\text{sec}, P_{NP} = 0.4686, E_P = 2.0\,\text{sec}, s_p = 1\,\text{sec}, P_p = 0.5414$

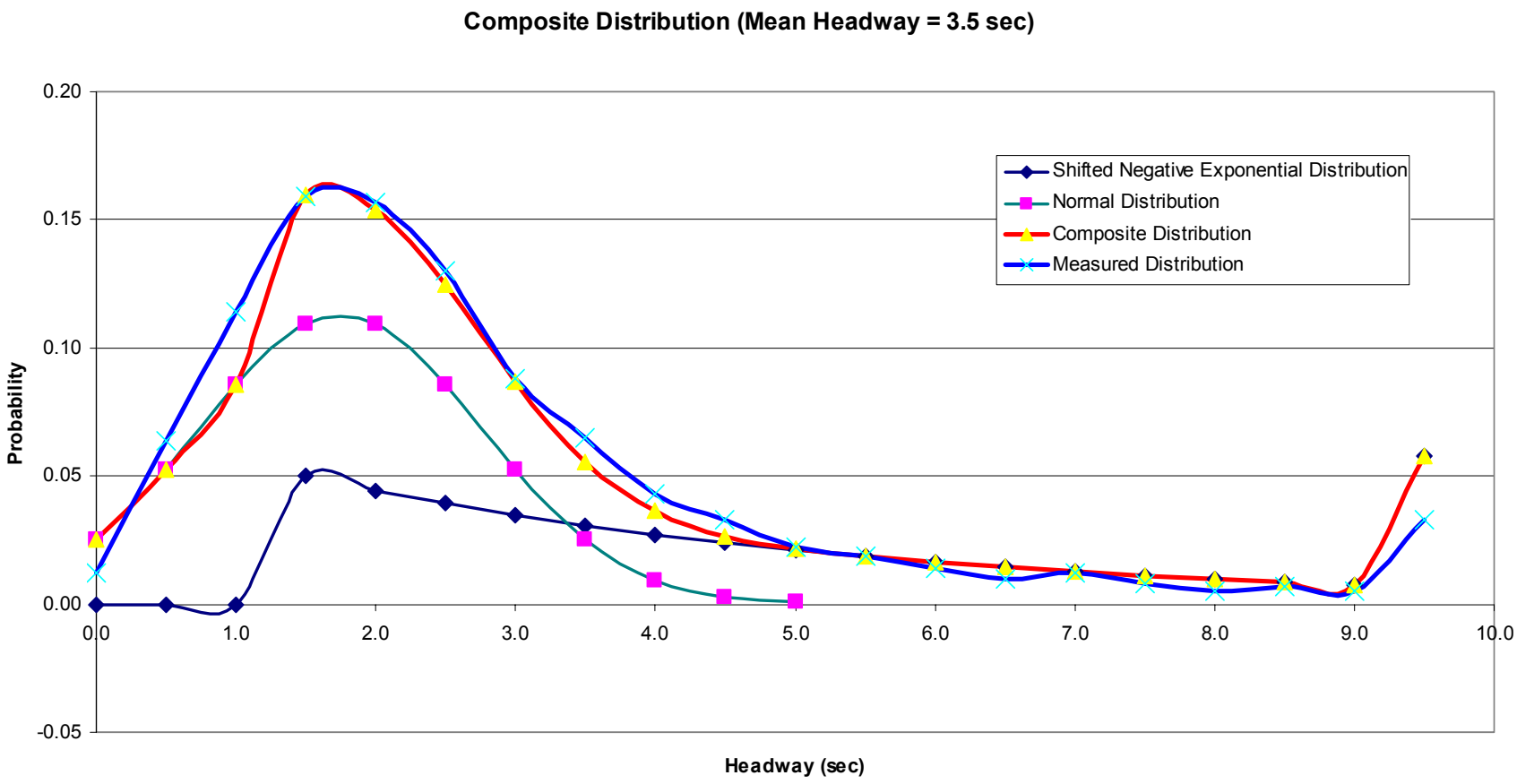


Figure B-1. Composite time headway distribution (based on Table B-1)

Table B-2. Chi-Square test calculation (based on Table B-1)

Time Headway Group	f_0	f_t	$f_0 - f_t$	$(f_0 - f_t)^2$	$\frac{(f_0 - f_t)^2}{f_t}$
0.0 - 0.5	16	33	-17	301	9.0686
0.5 – 1.0	84	69	15	230	3.3160
1.0 - 1.5	150	113	37	1394	12.3213
1.5 – 2.0	210	211	-1	1	0.0037
2.0 – 2.5	207	203	4	18	0.0904
2.5 – 3.0	172	165	7	45	0.2712
3.0 - 3.5	116	115	1	1	0.0116
3.5 – 4.0	86	74	12	151	2.0560
4.0 - 4.5	57	48	9	75	1.5585
4.5 – 5.0	44	35	8	72	2.0414
5.0 - 5.5	29	29	0	0	0.0043
5.5 – 6.0	25	24	1	0	0.0160
6.0 - 6.5	18	22	-3	10	0.4455
6.5 – 7.0	13	19	-6	34	1.7938
7.0 – 7.5	16	17	-1	1	0.0556
7.5 – 8.0	11	15	-4	18	1.2305
8.0 – 8.5	7	13	-6	42	3.2172
8.5 – 9.0	9	12	-2	5	0.4625
9.0 - 9.5	7	10	-4	13	1.2671
> 9.5	44	77	-33	1089	14.2249
	1320	1320	0		$\chi^2_{CALC} = 53.4561$

n = (I – 1 - p) = (20 – 1) – 4 = 15. Significant Level = 0.05, $\chi^2_{TABLE} = 25.0$.

$\chi^2_{CALC} > \chi^2_{TABLE}$, $53.4561 < 25.0$; Therefore, reject hypothesi

Table B-3. Time headway distribution (traffic flow rate = 1200 veh/h ~ 1440 veh/h)

	Vehicles Not in Platoons						Platoon Vehicles				Composite Distribution	
t	$t-\alpha$	$\bar{t}-\alpha$	$\bar{t}-\alpha$	$e^{-\frac{(t-\alpha)}{(\bar{t}-\alpha)}}$	100%	P	z	P(t<z)	100%	1-P	(P)	(F)
0.0	0.0	2.0	0.00			0.0000	-2.667	0.0039	0.0297	0.0176	0.0176	23
0.5	0.0	2.0	0.00			0.0000	-1.833	0.0336	0.1251	0.0741	0.0741	98
1.0	0.0	2.0	0.00			0.0000	-1.000	0.1587	0.2753	0.1631	0.1631	215
1.5	0.0	2.0	0.00	1.0000	0.0000	0.0000	-0.167	0.4340	0.3138	0.1860	0.1860	245
2.0	0.0	2.0	0.00	1.0000	0.0952	0.0388	0.667	0.7478	0.1854	0.1099	0.1486	196
2.5	0.2	2.0	0.10	0.9048	0.2001	0.0815	1.500	0.9332	0.0569	0.0337	0.1153	152
3.0	0.7	2.0	0.35	0.7047	0.1559	0.0635	2.333	0.9901	0.0097	0.0057	0.0693	91
3.5	1.2	2.0	0.60	0.5488	0.1214	0.0495	3.167	0.9998	0.0002	0.0001	0.0496	65
4.0	1.7	2.0	0.85	0.4274	0.0945	0.0385	4.000	1.0000			0.0385	51
4.5	2.2	2.0	1.10	0.3329	0.0736	0.0300					0.0300	40
5.0	2.7	2.0	1.35	0.2592	0.0573	0.0234					0.0234	31
5.5	3.2	2.0	1.60	0.2019	0.0447	0.0182					0.0182	24
6.0	3.7	2.0	1.85	0.1572	0.0348	0.0142					0.0142	19
6.5	4.2	2.0	2.10	0.1225	0.0271	0.0110					0.0110	15
7.0	4.7	2.0	2.35	0.0954	0.0211	0.0086					0.0086	11
7.5	5.2	2.0	2.60	0.0743	0.0164	0.0067					0.0067	9
8.0	5.7	2.0	2.85	0.0578	0.0128	0.0052					0.0052	7
8.5	6.2	2.0	3.10	0.0450	0.0100	0.0041					0.0041	5
9.0	6.7	2.0	3.35	0.0351	0.0078	0.0032					0.0032	4
9.5	7.2	2.0	3.60	0.0273	0.0273	0.0111					0.0111	15
					1.0000							

$\bar{t}_{NP} = 4.3\,\text{sec}, \alpha = 2.3\,\text{sec}, s_{NP} = 2.0\,\text{sec}, P_{NP} = 0.4074, E_P = 1.6\,\text{sec}, s_p = 0.6\,\text{sec}, P_p = 0.5926$

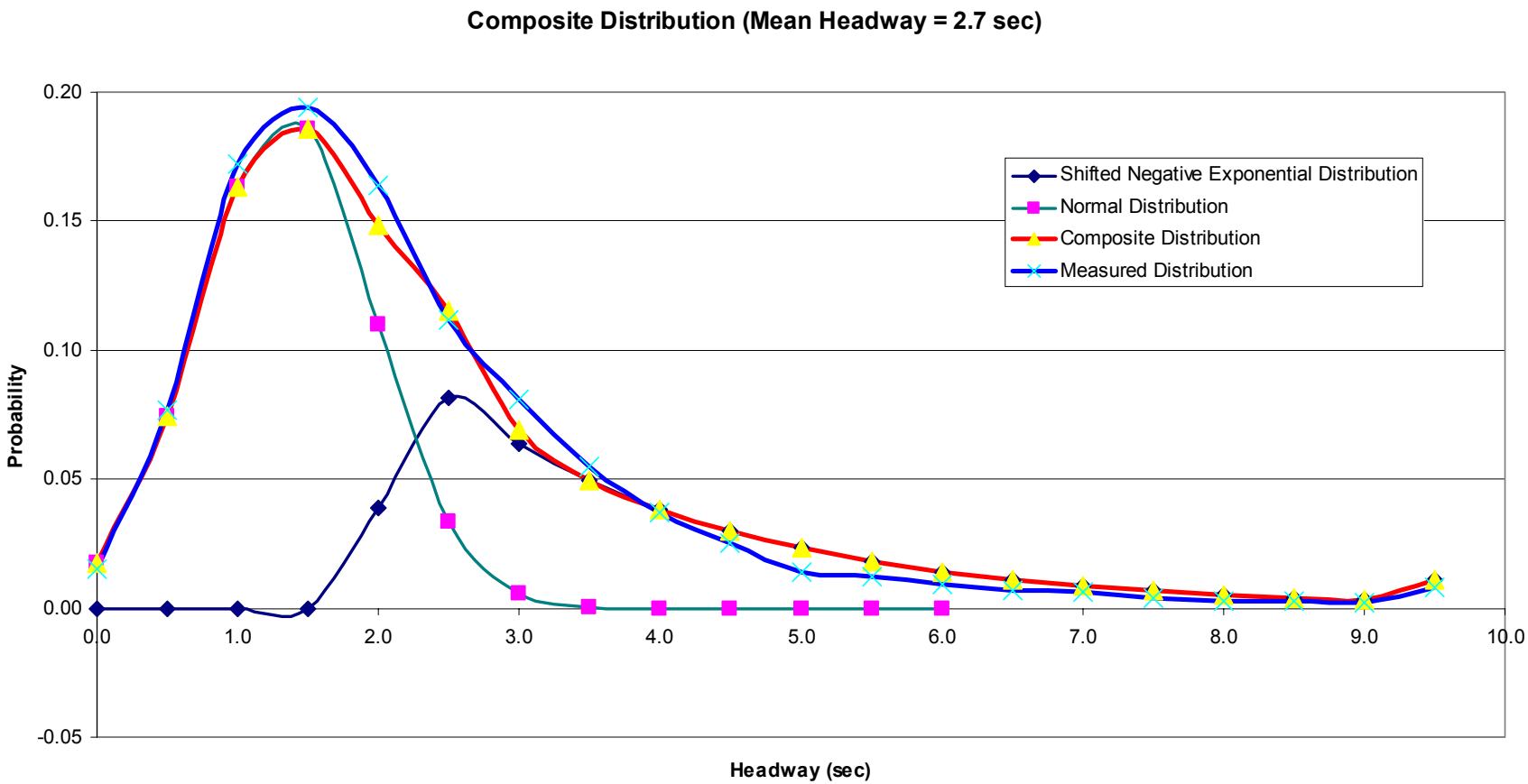


Figure B-2. Composite time headway distribution (based on Table B-3)

Table B-4. Chi-Square test calculation (based on Table B-3)

Time Headway Group	f_0	f_t	$f_0 - f_t$	$(f_0 - f_t)^2$	$\frac{(f_0 - f_t)^2}{f_t}$
0.0 - 0.5	20	23	-3	12	0.5070
0.5 – 1.0	102	98	4	14	0.1463
1.0 - 1.5	227	215	12	137	0.6350
1.5 – 2.0	256	245	11	113	0.4594
2.0 – 2.5	216	196	20	411	2.0962
2.5 – 3.0	148	152	-4	19	0.1218
3.0 - 3.5	107	91	16	240	2.6300
3.5 – 4.0	73	65	7	51	0.7832
4.0 - 4.5	49	51	-2	4	0.0790
4.5 – 5.0	33	40	-7	44	1.0991
5.0 - 5.5	18	31	-12	153	4.9525
5.5 – 6.0	16	24	-8	67	2.7839
6.0 - 6.5	12	19	-7	47	2.4899
6.5 – 7.0	9	15	-5	28	1.9480
7.0 – 7.5	8	11	-3	12	1.0339
7.5 – 8.0	5	9	-4	13	1.4306
8.0 – 8.5	4	7	-3	9	
8.5 – 9.0	4	5	-1	2	0.7823
9.0 - 9.5	3	4	-2	2	
> 9.5	11	15	-4	17	1.1631
	1320	1320	0		$\chi^2_{CALC} = 25.141$

n = (I – 1 - p) = (18 – 1) – 4 = 13. Significant Level = 0.05, $\chi^2_{TABLE} = 22.4$.

$\chi^2_{CALC} > \chi^2_{TABLE}$, 25.141 > 22.4 ; Therefore, reject hypothesi

Table B-5. Time headway distribution (traffic flow rate = 1500 veh/h ~ 1740 veh/h)

	Vehicles Not in Platoons						Platoon Vehicles				Composite Distribution	
t	$t-\alpha$	$\bar{t}-\alpha$	$\bar{t}-\alpha$	$e^{-\frac{(t-\alpha)}{(\bar{t}-\alpha)}}$	100%	P	z	P(t<z)	100%	1-P	(P)	(F)
0.0	0.0	1.5	0.0000	----	----	0.0000	-3.000	0.0013	0.0215	0.0147	0.0147	19
0.5	0.0	1.5	0.0000	----	----	0.0000	-2.000	0.0228	0.1359	0.0927	0.0927	122
1.0	0.0	1.5	0.0000	----	----	0.0000	-1.000	0.1587	0.3413	0.2327	0.2327	307
1.5	0.0	1.5	0.0000	1.0000	0	0.0000	0.0000	0.5	0.3413	0.2327	0.2327	307
2.0	0.0	1.5	0.0000	1.0000	0.1813	0.0577	1.0000	0.8413	0.1359	0.0927	0.1503	198
2.5	0.3	1.5	0.2000	0.8187	0.2321	0.0738	2.0000	0.9772	0.0215	0.0147	0.0885	117
3.0	0.8	1.5	0.5333	0.5866	0.1663	0.0529	3.0000	0.9987	0.0013	0.0009	0.0538	71
3.5	1.3	1.5	0.8667	0.4204	0.1192	0.0379	4.0000	1			0.0379	50
4.0	1.8	1.5	1.2000	0.3012	0.0854	0.0272					0.0272	36
4.5	2.3	1.5	1.5333	0.2158	0.0612	0.0195					0.0195	26
5.0	2.8	1.5	1.8667	0.1546	0.0438	0.0139					0.0139	18
5.5	3.3	1.5	2.2000	0.1108	0.0314	0.0100					0.0100	13
6.0	3.8	1.5	2.5333	0.0794	0.0225	0.0072					0.0072	9
6.5	4.3	1.5	2.8667	0.0569	0.0161	0.0051					0.0051	7
7.0	4.8	1.5	3.2000	0.0408	0.0116	0.0037					0.0037	5
7.5	5.3	1.5	3.5333	0.0292	0.0083	0.0026					0.0026	3
8.0	5.8	1.5	3.8667	0.0209	0.0059	0.0019					0.0019	2
8.5	6.3	1.5	4.2000	0.0150	0.0043	0.0014					0.0014	2
9.0	6.8	1.5	4.5333	0.0107	0.003	0.0010					0.0010	1
9.5	7.3	1.5	4.8667	0.0077	0.0077	0.0024					0.0024	3

$\bar{t}_{NP} = 3.7\,\text{sec}, \alpha = 2.2\,\text{sec}, s_{NP} = 1.5\,\text{sec}, P_{NP} = 0.3182, E_P = 1.5\,\text{sec}, s_p = 0.5\,\text{sec}, P_p = 0.6818$

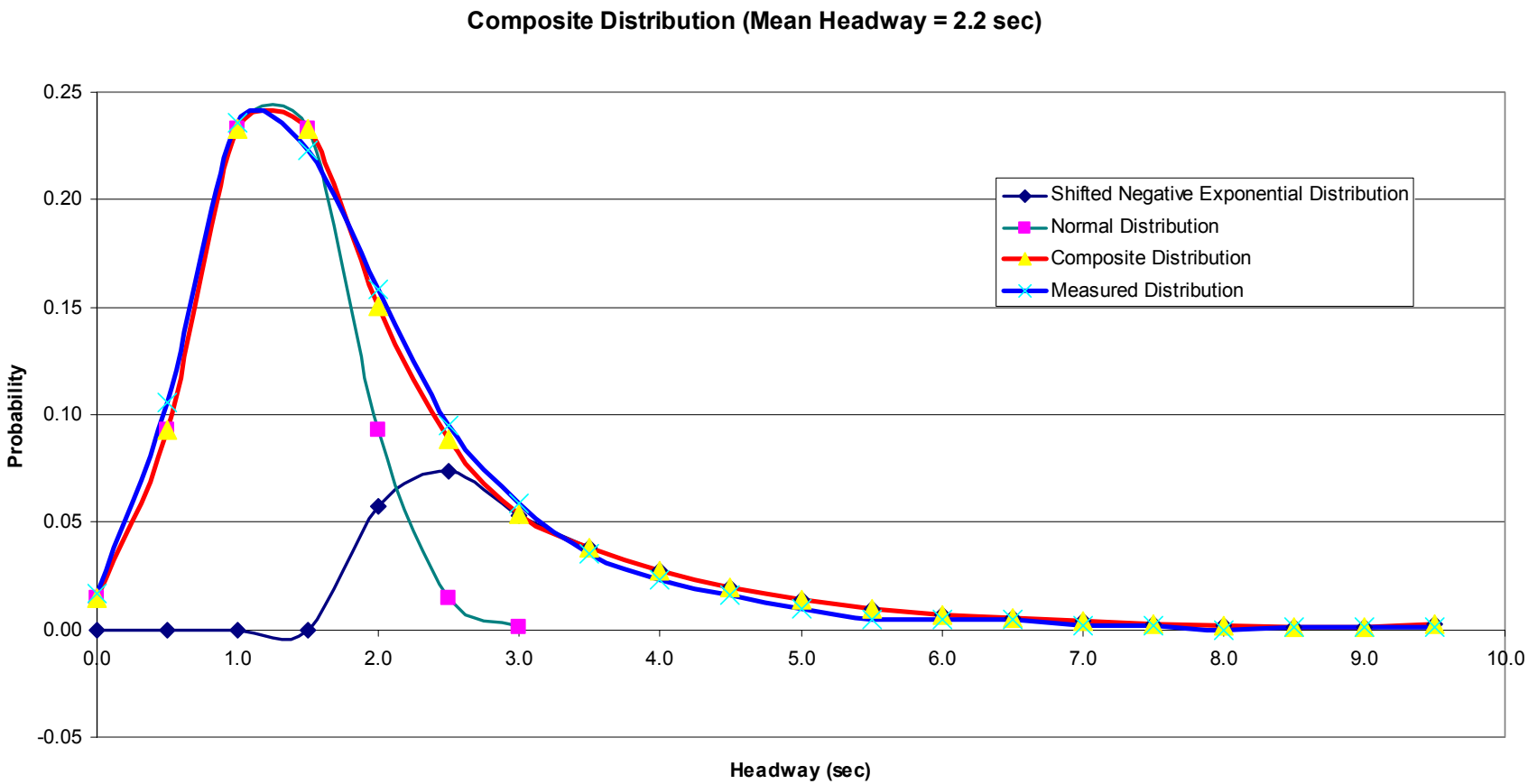


Figure B-3. Composite time headway distribution (based on Table B-5)

Table B-6. Chi-Square test calculation (based on Table B-5)

Time Headway Group	f_0	f_t	$f_0 - f_t$	$(f_0 - f_t)^2$	$\frac{(f_0 - f_t)^2}{f_t}$
0.0 - 0.5	22	19	3	10	0.4934
0.5 – 1.0	140	122	18	310	2.5355
1.0 - 1.5	312	307	4	19	0.0616
1.5 – 2.0	294	307	-13	164	0.5342
2.0 – 2.5	209	198	10	102	0.5158
2.5 – 3.0	125	117	9	74	0.6293
3.0 - 3.5	78	71	7	47	0.6638
3.5 – 4.0	46	50	-4	15	0.2955
4.0 - 4.5	30	36	-5	30	0.8433
4.5 – 5.0	21	26	-5	21	0.8143
5.0 - 5.5	13	18	-5	27	1.4748
5.5 – 6.0	7	13	-7	43	3.2939
6.0 - 6.5	7	9	-3	8	0.8607
6.5 – 7.0	7	7	0	0	0.0044
7.0 – 7.5	3	5	-2	5	
7.5 – 8.0	3	3	-1	1	1.9678
8.0 – 8.5	0	2	-2	6	
8.5 – 9.0	2	2	0	0	
9.0 - 9.5	1	1	0	0	0.4183
> 9.5	3	1	-2	4	
	1320	1320	0		$\chi^2_{CALC} = 15.4068$

n = (I – 1 - p) = (16 – 1) – 4 = 11. Significant Level = 0.05, $\chi^2_{TABLE} = 19.70$.

$\chi^2_{CALC} < \chi^2_{TABLE}$, $15.41 < 19.70$; Therefore, not reject hypothesis.

APPENDIX C
ENTERING PERCENT FOLLOWING (EPF) CALCULATION

C.1 Methods to Calculate EPF Downstream of Signal

On calculating EPF downstream of signal, shifted negative exponential distribution and composite distribution are introduced into Dixon et al.'s methodology [3].

Estimation of the percentage of entering traffic following is based on a flow profile immediately downstream of the signalized intersection. A flow profile immediately downstream of the signalized intersection at location "A" is shown in the Figure C-1. The "A" denotes a location immediately downstream of the signalized intersection. As shown in the Figure C-1, there are three movements that contribute to the flow profile:

- Movement 1: Primary contributing movement. They are through movements from the upstream major street,
- Movement 2: Secondary contributing movement. They are right-turn movements from the minor street, and
- Movement 3: Secondary contributing movement. They are left-turn movements from the minor street.

As shown in the Figure C-1, the total cycle-length is divided into three states. The above three movements are discharged through the three states. They are:

- First state: Discharge from the through movement queue during the first phase
- Second state: Discharge from the through movement without a queue plus any right-turn on red executed during the first phase
- Third state: Discharge from the right and left turn movements during the second phase

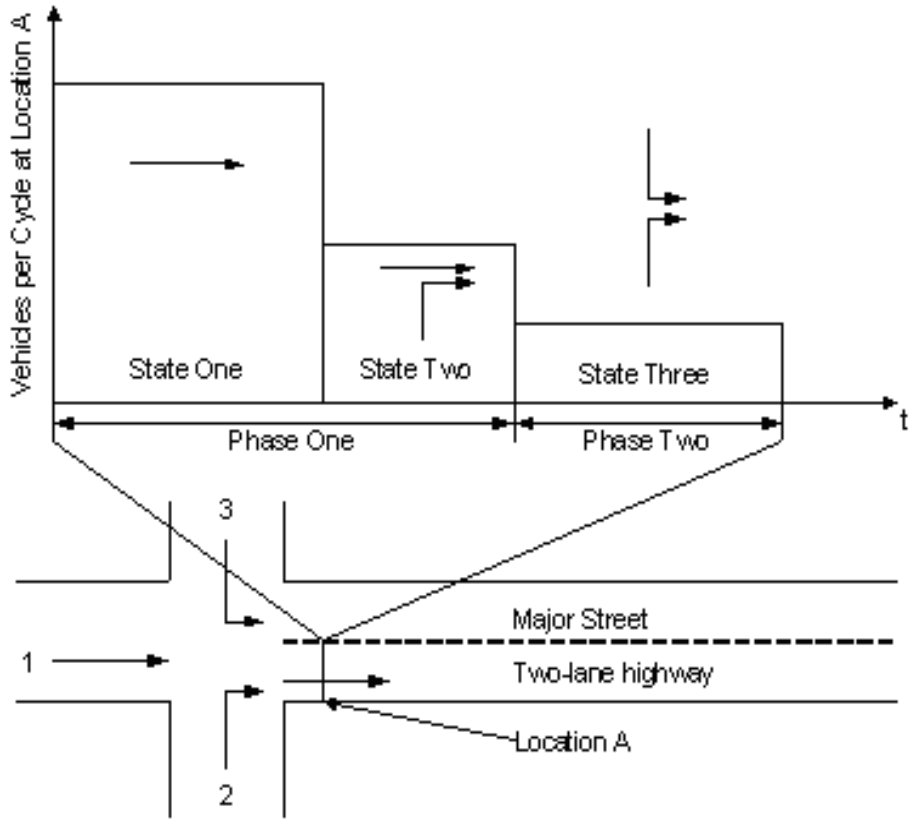


Figure C-1. Two-lane highway traffic flow downstream of a signalized intersection

Source: Dixon, Michael P., Michael Kyte, and Satya Sai Kumar Sarepali. *Effects of Upstream Signalized Intersections on Two-Lane Highway Operations, Transportation Research Board, Washington D.C., 2004*

Entering percent following at location A can be estimated using equation C-1:

$$EPF_a = \frac{VF_a}{V_a} \tag{C-1}$$

$$VF_a = \sum_i VF_i \tag{C-2}$$

Where:

EPF_a: percent of vehicles following at Point A, immediately downstream of a signalized intersection,

VF_a: total number of vehicles following per cycle at location A, veh

VF_i: total number of vehicles following per cycle from movement i, veh and

V_a: total number of vehicles per cycle at location A, veh

To determine the EPF_a, the key point is to decide the denominator, V_a, and nominator, VF_a. Because V_a is the summation of the cycle-by-cycle volumes from movements 1, 2, and 3, it can be determined if volumes for movements 1, 2 and 3 and the cycle length are known. This leaves the estimation of VF_a, the number of vehicles following at location A, which can be estimated by movement.

Vehicles following in movement 1

Movement 1 is discharged during phase one, which is divided into flow state 1 and flow state 2. During state 1, since vehicles discharged are in a platoon, it can be assumed that all of the queued vehicles are in a platoon at location A, and are following, except the platoon leader, as represented in Equation C-4. Note that Equation C-4 is based on a deterministic queue length model, consistent with queue length equations used in the HCM 2000 signalized intersection analysis procedure. Equation C-3 is used to calculate t_{Q1} in Equation C-4 based on traffic flow rates and the timing plan of the signal. Here, uniform arrivals are assumed. Stochastic queue length models could be used instead to provide more realistic representations of the effects of random vehicle arrivals on queue length.

$$t_{Q1} = \frac{v_1}{s_1 - v_1}(C - g_1) \qquad \text{(C-3)}$$

Where:

t_{Q1}: time for queue at movement 1 to dissipate, sec

v_1: movement 1 demand flow rate, veh/h

s_1: saturation flow rate for movement 1, veh/h

C: cycle length, sec and

g_1: phase one effective green time, sec

$$VF_{Q1} = \mathrm{s}_1(\frac{t_{Q1}}{3600}) - 1 \tag{C-4}$$

Where:

VF_{Q1}: number of vehicles following in movement 1 that are discharging from a queue, veh/h

During State 2, vehicles discharging from movement 1 when no queue is present may or may not be following at location A. If it is assumed that they are following if their headway is less than or equal to 3.0 seconds, the number of vehicles following at location A can be estimated using the shifted negative exponential distribution or composite distribution. When traffic flow rate is low, the shifted negative exponential distribution is used through Equation C-5; when traffic flow rate is high, the composite distribution is used through Equation C-6.

$$VF_{NQ1} = \mathrm{v}_1\left(\frac{g_1 - t_{Q1}}{3600}\right).\left(1 - e^{-(t-\alpha)/(\frac{3600}{v_{nq}^{a,2}} - \alpha)}\right) \tag{C-5}$$

Where:

VF_{NQ1}: number of vehicles following in movement 1 that are discharging when no queue is present, veh

$v_{nq}^{a,2}$: flow rate at location A during flow rate 2, when no queue is present, veh/h

α : user-selected parameter, which affects the shift of the distribution, sec

Or

$$VF_{NQ1} = \mathrm{v}_1(\frac{g_1 - t_{Q1}}{3600})\left[1 - (P_{NP}e^{-(t-\alpha)/(\bar{t}_{NP}-\alpha)} + P_P\int_t^{\infty}\frac{1}{s\sqrt{2\pi}}e^{-\frac{1}{2}(\frac{t-\bar{t}_P}{s})^2}dt)\right] \tag{C-6}$$

Where:

P_p: proportion of vehicle in platoon, %

P_{NP}: proportion of vehicles not in platoon, %

$\bar{t}_p$: mean headway of the vehicles in platoon, sec

$\bar{t}_{Np}$: mean headway of the vehicles not in platoon, sec

α : the minimum time headway for vehicles not in platoon, sec

s: standard deviation of normal distribution,

t: time headway being investigated, sec

$\bar{t}$: mean headway, sec/v

$$V_{NQ}^{a,2} = v_1 + v_2 P_{RTOR} \frac{C}{g_1 - t_{Q1}} \qquad \text{(C-7)}$$

Where:

v_2: movement two demand flow rate, veh/h

P_{RTOR}: proportion of vehicles turning right on red

When using Equation C-5 or Equation C-6, the following items should be noted:

- The through movement volume, v_1, is used in Equation C-5 or Equation C-6 because it is estimating the number of vehicles following in movement 1.
- The time during the green time when no queue is present for movement 1 is $g_1 - t_{Q1}$. The proportion of these non-queued vehicles that are following, given a headway criterion of t, is estimated by the value in the brackets containing the shifted negative exponential distribution in Equation C-5 or the composite distribution Equation C-6.
- The volume, $V_{NQ}^{a,2}$ shown in Equation C-7, is used to calculate the mean headway because it is the volume occurring during flow state 2 at location A. Also note that Equation C-7 is comprised of the movement 1 flow rate (the first element) and the right-turn-on-red flow rate (the second element), which are the movements arriving at location A during flow state 2.

- Proportion of right turn vehicles turning on red can be determined from the field measurement or by estimation

Finally, the total number of through vehicles that are following at location A can be estimated as shown in Equation C-8. Note that an alternative to the estimation of t_{Q1}, as given by Equation C-4, is available as part of the signalized intersections procedure in the HCM 2000 and should be considered as part of a formal procedure implementing the methodology.

$$VF_1 = \mathrm{VF}_{\mathrm{Q1}} + \mathrm{VF}_{\mathrm{NQ1}} \tag{C-8}$$

Where:

VF_1: total number of vehicles following per cycle from movement 1, veh

Vehicles following in Movement 2

In the cross street, one lane is shared by through vehicles, left-turn vehicles, and right-turn vehicles. During the phase two, the left-turn vehicles need to yield to the through vehicles and right-turn vehicles in the opposite direction to drive into the main street. Here it is assumed that the secondary nature of left-turn movement and right-turn movement do not justify determining how these movements interact with other movements during the second phase.

Vehicles in movement 2 are dissipated in State 2 and State 3. In the flow state 2, the right-turn vehicles are turning on red. When traffic flow rate in the flow state 2 is low, the number of vehicles following from movement two is estimated using Equation C-9, assuming a shifted negative exponential distribution of headways in the flow state 2. When the traffic flow rate in the flow state 2 is high, Equation C-10 is used for calculation, assuming a composite distribution of headway in the flow state 2.

$$VF_2^{'} = \mathrm{v}_2 \frac{C}{3600}\left[P_{PROR}(1-e^{-(t-\alpha)/(\frac{3600}{v_{nq}^{a,2}}-\alpha)})\right] \quad \text{(C-9)}$$

Where:

$VF_2^{'}$: number of vehicles following in movement 2 that are discharging on red, veh

Or

$$VF_2^{'} = \mathrm{v}_2 \frac{C}{3600}\left[P_{PTOR}(1-(P_{NP}e^{-(t-\alpha)/(\bar{t}_{NP}-\alpha)} + P_P\int_t^{\infty}\frac{1}{s\sqrt{2\pi}}e^{-\frac{1}{2}(\frac{t-\bar{t}_P}{s})^2}dt))\right] \quad \text{(C-10)}$$

In the flow state 3, the number of vehicles following from movement two is estimated using Equation C-11 or Equation C-12, respectively, assuming a negative exponential distribution of headways or composite distribution. When traffic flow rate is low, the shifted negative exponential distribution is used through Equation C-11; when traffic flow rate is high, the composite distribution is used through Equation C-12.

$$VF_2^{''} = \mathrm{v}_2 \frac{C}{3600}\left[(1-P_{PTOR})(1-e^{-(t-\alpha)/(\frac{3600}{v^{a,3}}-\alpha)})\right] \quad \text{(C-11)}$$

Where:

$VF_2^{''}$: number of vehicles following in movement 2 that are discharging not on red, veh

$v^{a,3}$: flow rate at location A during flow state 3, veh/h and

Or

$$VF_2^{''} = \mathrm{v}_2 \frac{C}{3600}\left[(1-P_{PTOR})(1-(P_{NP}e^{-(t-\alpha)/(\bar{t}_{NP}-\alpha)} + P_P\int_t^{\infty}\frac{1}{s\sqrt{2\pi}}e^{-\frac{1}{2}(\frac{t-\bar{t}_P}{s})^2}dt))\right] \quad \text{(C-12)}$$

$$v^{a,3} = \frac{C}{C-g_1}\left[v_3 + v_2(1-P_{RTOR})\right] \quad \text{(C-13)}$$

Where:

v^3 : movement 3 demand flow rate, veh/h

When using Equation C-9, C-10, C-11 or C-12, the following items should be noted:

- The right-turn movement volume, $\mathbf{v}_2$, is used in Equation C-9, C-10, C-11 or C-12, because it is estimating the number of vehicles following in movement 2.
- The volume, $v^{a,3}$ shown in Equation C-9, is used to calculate the mean headway because it is the volume occurring during flow state 3 at location A. Also note that Equation C-11 is comprised of the movement 3 flow rate (the first element) and the right-turn-on-green flow rate (the second element), which are the movements arriving at location A during flow state 3.
- The variable, $(1 - P_{RTOR})$, is the portion of right-turn vehicle dissipated on green in the flow state 3.

Finally, the total number of right-turn vehicles that are following at location A can be estimated as shown in Equation C-13.

$$VF_2 = VF_2^{'} + VF_2^{''} \tag{C-13}$$

Where:

VF_2 : total number of vehicle following per cycle from movement 2, veh

Vehicles following in movement 3

Vehicles following from the left-turn movement are determined for flow state 3 only. When traffic flow rate in the flow state 3 is low, the number of vehicles following from movement 3 is estimated using Equation C-14, assuming a shifted negative exponential distribution of headways in the flow state 3. When traffic flow rate in the flow state 3 is high, Equation C-15 is used for calculation, assuming a composite distribution of headway in the flow state 3.

$$VF_3 = \mathrm{v}_3 \frac{C}{3600}\left[1 - e^{-(t-\alpha)/(\frac{3600}{v^{a,3}}-\alpha)}\right] \quad \text{(C-14)}$$

Where:

VF_3 : total number of vehicle following per cycle from movement 3, veh

Or

$$VF_3 = \mathrm{v}_3 \frac{C}{3600}\left[(1-P_{PTOR})(1-(P_{NP}e^{-(t-\alpha)/(\bar{t}_{NP}-\alpha)} + P_P\int_t^{\infty}\frac{1}{s\sqrt{2\pi}}e^{-\frac{1}{2}(\frac{t-\bar{t}_P}{s})^2}dt))\right] \quad \text{(C-15)}$$

When using Equation C-14 or C-15, the following items should be noted:

- The right-turn movement volume, v_3, is used in Equation C-14 and C-15, because it is estimating the number of vehicles following in movement 3.

- The volume, $v^{a,3}$ shown in Equation C-14, is used to calculate the mean headway because it is the volume occurring during flow state 3 at location A. Also note that Equation C-14 is comprised of the movement 3 flow rate (the first element) and the right-turn-on-green flow rate (the second element), which are the movements arriving at location A during flow state 3.

When determining the number of vehicles following at a location immediately downstream of the signalized intersection, either the shifted negative exponential distribution or composite distribution of headway is used. In Equation C-5, Equation C-9, Equation C-11, and Equation C-14, the shifted negative exponential distribution of headways is assumed. The traffic flow rates used to calculate mean headway are different under different flow states for different movements. In Equation C-6, Equation C-10, Equation C-12, and Equation C-15, the composite distribution of headways is assumed. Note that the variables $\mathrm{P}_\mathrm{P}, P_{NP}, \bar{t}_p, \bar{t}_{np}, \alpha, s$ in these equations are expressed in the same ways, but their values are different. All these values are decided by the vehicle volumes under the different conditions. It is summarized as follows:

- In Equation C-6, vehicles following from the through movement that are discharging when no queue is present are determined. The flow rate at location A during flow state 2, $v_{NQ}^{a,2}$ is used to determine these parameters of $\mathrm{P_P}, P_{NP}, \bar{t}_p, \bar{t}_{np}, \alpha, s$ in the composite distribution. Note that $v_{NQ}^{a,2}$ is composed of the movement 1 flow rate and the right-turn-on red flow rate.

- In Equation C-10, vehicles following from the right-turn movement that are discharging on red during flow state 2 are determined. The flow rate at location A during flow state 2, $v_{NQ}^{a,2}$ is still used to determine these parameters of $\mathrm{P_P}, P_{NP}, \bar{t}_p, \bar{t}_{np}, \alpha, s$ in the composite distribution.

- In Equation C-12, vehicles following from the right-turn movement that are discharging on green during flow state 3 are determined, and the flow rate at location A during flow state 3, $v^{a,3}$ is used to determine these parameters of $\mathrm{P_P}, P_{NP}, \bar{t}_p, \bar{t}_{np}, \alpha, s$ in the composite distribution. Note that the flow rate 3 volume used is comprised of left-turn vehicles and right-turn-on-green vehicles.

- In Equation C-15, vehicles following from the left-turn movement during flow state 3 are determined, and the flow at location A during flow state 3, $v^{a,3}$ is still used to determine these parameters of $P_P, P_{NP}, \bar{t}_p, \bar{t}_{np}, \alpha, s$ in the composite distribution.

After VF_1, VF_2, and VF_3 are decided, the EPF at location A, can be calculated using the following equation:

$$EPF_a = \frac{VF_1 + VF_2 + VF_3}{V_a} \tag{C-16}$$

Table C-1 summarized the values of EPF under the different traffic conditions and signal timing plans. Due to plenty of calculation, a program was developed to calculate the Entering Percent Following using the Visual Basic Language. The Visual Basic code is presented in Appendix D.

C.2 Method to Calculate EPF Without Signal

Dixon et al. concluded that it was appropriate to represent the effects of a signalized intersection on the downstream two-lane highway operations through the EPF parameters, as long as the percent following immediately downstream of a signalized intersection can be determined. In this study, the methodology for determining Entering

Percent Following referred to Dixon et al.'s methodology [3]. The main difference from their methodology reflects on the application of distributions for time headway. As discussed in Chapter 4, the shifted negative exponential distribution and composite distribution are introduced into this methodology. The detailed procedure determining Entering Percent Following downstream of a signalized intersection are presented as follows.

The entering percent following at the location A, without experiencing the effects of a signalized intersection can be calculated using Equation C-17 or C-18. When the traffic flow rate is low, the headway distribution of entering traffic can be assumed to be shifted negative exponential and Equation C-17can be used to estimate EPF; when the traffic flow is high, the headways of entering traffic can be assumed to following composite distribution ad Equation C-18 can be used to estimate EPF.

$$EPF_a = 100\left[1 - e^{-(t-\alpha)/(\frac{3600}{q_a}-\alpha)}\right] \tag{C-17}$$

Where:

EPF_a: entering percent following at the location a

q_a : flow rate at location a, vph

Or

$$EPF_a = 100\left[(1 - P_{PTOR})(1 - (P_{NP}e^{-(t-\alpha)/(\bar{t}_{NP}-\alpha)} + P_P\int_t^{\infty}\frac{1}{s\sqrt{2\pi}}e^{-\frac{1}{2}(\frac{t-\bar{t}_P}{s})^2}dt))\right] \tag{C-18}$$

$$\bar{t} = \frac{3600}{q_a} \tag{C-19}$$

Where:

$\bar{t}$: average time headway, sec

In Equation C-18, the average time headway is calculated using Equation C-19, and other parameters ($P_P, P_{NP}, \bar{t}_p, \bar{t}_{np}, \alpha$) in the composite distribution can be determined referring to the detailed procedure presented in Chapter 4.

Table C-1. Entering Percent Following with a signalized intersection

Through Vehicles (veh/h)	Right/Left Turn Vehicles (veh/h)	Cycle Length (sec)	g/C	EPF	Through Vehicles (veh/h)	Right/Left Turn Vehicles (veh/h)	Cycle Length (sec)	g/C	EPF
200	20	60	0.6	0.2069	200	20	60	0.7	0.1192
200	20	60	0.8	0.0324	200	20	75	0.6	0.2615
200	20	75	0.7	0.1737	200	20	75	0.8	0.0869
200	20	90	0.6	0.2978	200	20	90	0.7	0.2101
200	20	90	0.8	0.1233	200	20	105	0.6	0.3238
200	20	105	0.7	0.2361	200	20	105	0.8	0.1493
200	20	120	0.6	0.3433	200	20	120	0.7	0.2556
200	20	120	0.8	0.1687	200	40	60	0.6	0.1984
200	40	60	0.7	0.1205	200	40	60	0.8	0.0457
200	40	75	0.6	0.2484	200	40	75	0.7	0.1705
200	40	75	0.8	0.0957	200	40	90	0.6	0.2818
200	40	90	0.7	0.2038	200	40	90	0.8	0.1291
200	40	105	0.6	0.3056	200	40	105	0.7	0.2276
200	40	105	0.8	0.1529	200	40	120	0.6	0.3234
200	40	120	0.7	0.2455	200	40	120	0.8	0.1707
200	60	60	0.6	0.1958	200	60	60	0.7	0.1275
200	60	60	0.8	0.0655	200	60	75	0.6	0.242
200	60	75	0.7	0.1737	200	60	75	0.8	0.1117
200	60	90	0.6	0.2727	200	60	90	0.7	0.2044
200	60	90	0.8	0.1425	200	60	105	0.6	0.2947
200	60	105	0.7	0.2264	200	60	105	0.8	0.1644
200	60	120	0.6	0.3112	200	60	120	0.7	0.2429
200	60	120	0.8	0.1809	400	40	60	0.6	0.4515
400	40	60	0.7	0.3662	400	40	60	0.8	0.2826
400	40	75	0.6	0.4788	400	40	75	0.7	0.3935
400	40	75	0.8	0.3099	400	40	90	0.6	0.4969
400	40	90	0.7	0.4117	400	40	90	0.8	0.3281
400	40	105	0.6	0.5099	400	40	105	0.7	0.4246
400	40	105	0.8	0.3411	400	40	120	0.6	0.5197
400	40	120	0.7	0.4344	400	40	120	0.8	0.3508
400	80	60	0.6	0.4309	400	80	60	0.7	0.3573
400	80	60	0.8	0.2896	400	80	75	0.6	0.4559
400	80	75	0.7	0.3823	400	80	75	0.8	0.3146
400	80	90	0.6	0.4725	400	80	90	0.7	0.399
400	80	90	0.8	0.3313	400	80	105	0.6	0.4844
400	80	105	0.7	0.4109	400	80	105	0.8	0.3432
400	80	120	0.6	0.4934	400	80	120	0.7	0.4198
400	80	120	0.8	0.3521	400	120	60	0.6	0.4219
400	120	60	0.7	0.3607	400	120	60	0.8	0.3103
400	120	75	0.6	0.445	400	120	75	0.7	0.3838
400	120	75	0.8	0.3334	400	120	90	0.6	0.4604
400	120	90	0.7	0.3992	400	120	90	0.8	0.3488
400	120	105	0.6	0.4714	400	120	105	0.7	0.4102
400	120	105	0.8	0.3597	400	120	120	0.6	0.4796
400	120	120	0.7	0.4184	400	120	120	0.8	0.368

600	60	60	0.6	0.5988	600	60	60	0.7	0.5149
600	60	60	0.8	0.4334	600	60	75	0.6	0.6169
600	60	75	0.7	0.533	600	60	75	0.8	0.4516
600	60	90	0.6	0.6291	600	60	90	0.7	0.5452
600	60	90	0.8	0.4637	600	60	105	0.6	0.6377
600	60	105	0.7	0.5538	600	60	105	0.8	0.4724
600	60	120	0.6	0.6442	600	60	120	0.7	0.5603
600	60	120	0.8	0.4789	600	120	60	0.6	0.5736
600	120	60	0.7	0.5033	600	120	60	0.8	0.4408
600	120	75	0.6	0.5903	600	120	75	0.7	0.52
600	120	75	0.8	0.4574	600	120	90	0.6	0.6014
600	120	90	0.7	0.5311	600	120	90	0.8	0.4685
600	120	105	0.6	0.6093	600	120	105	0.7	0.539
600	120	105	0.8	0.4765	600	120	120	0.6	0.6153
600	120	120	0.7	0.545	600	120	120	0.8	0.4824
600	180	60	0.6	0.5643	600	180	60	0.7	0.5083
600	180	60	0.8	0.4722	600	180	75	0.6	0.5797
600	180	75	0.7	0.5237	600	180	75	0.8	0.4876
600	180	90	0.6	0.5899	600	180	90	0.7	0.5339
600	180	90	0.8	0.4978	600	180	105	0.6	0.5973
600	180	105	0.7	0.5413	600	180	105	0.8	0.5051
600	180	120	0.6	0.6028	600	180	120	0.7	0.5468
600	180	120	0.8	0.5106	800	80	60	0.6	0.7213
800	80	60	0.7	0.6372	800	80	60	0.8	0.5562
800	80	75	0.6	0.735	800	80	75	0.7	0.6508
800	80	75	0.8	0.5698	800	80	90	0.6	0.7441
800	80	90	0.7	0.6599	800	80	90	0.8	0.5789
800	80	105	0.6	0.7506	800	80	105	0.7	0.6664
800	80	105	0.8	0.5854	800	80	120	0.6	0.7554
800	80	120	0.7	0.6713	800	80	120	0.8	0.5903
800	160	60	0.6	0.6951	800	160	60	0.7	0.6261
800	160	60	0.8	0.5664	800	160	75	0.6	0.7076
800	160	75	0.7	0.6386	800	160	75	0.8	0.5789
800	160	90	0.6	0.7159	800	160	90	0.7	0.6469
800	160	90	0.8	0.5873	800	160	105	0.6	0.7219
800	160	105	0.7	0.6529	800	160	105	0.8	0.5932
800	160	120	0.6	0.7263	800	160	120	0.7	0.6573
800	160	120	0.8	0.5977	800	240	60	0.6	0.6878
800	240	60	0.7	0.6346	800	240	60	0.8	0.6205
800	240	75	0.6	0.6993	800	240	75	0.7	0.6461
800	240	75	0.8	0.6321	800	240	90	0.6	0.707
800	240	90	0.7	0.6538	800	240	90	0.8	0.6398
800	240	105	0.6	0.7125	800	240	105	0.7	0.6593
800	240	105	0.8	0.6453	800	240	120	0.6	0.7166
800	240	120	0.7	0.6634	800	240	120	0.8	0.6494
1000	100	60	0.6	0.8435	1000	100	60	0.7	0.7798
1000	100	60	0.8	0.7184	1000	100	75	0.6	0.8544
1000	100	75	0.7	0.7907	1000	100	75	0.8	0.7294
1000	100	90	0.6	0.8617	1000	100	90	0.7	0.798
1000	100	90	0.8	0.7366	1000	100	105	0.6	0.8669

1000	100	105	0.7	0.8032	1000	100	105	0.8	0.7418
1000	100	120	0.6	0.8708	1000	100	120	0.7	0.8071
1000	100	120	0.8	0.7457	1000	200	60	0.6	0.8121
1000	200	60	0.7	0.7656	1000	200	60	0.8	0.7389
1000	200	75	0.6	0.8221	1000	200	75	0.7	0.7756
1000	200	75	0.8	0.7489	1000	200	90	0.6	0.8288
1000	200	90	0.7	0.7823	1000	200	90	0.8	0.7556
1000	200	105	0.6	0.8335	1000	200	105	0.7	0.787
1000	200	105	0.8	0.7603	1000	200	120	0.6	0.8371
1000	200	120	0.7	0.7906	1000	200	120	0.8	0.7639
1000	300	60	0.6	0.8027	1000	300	60	0.7	0.7888
1000	300	60	0.8	0.7749	1000	300	75	0.6	0.812
1000	300	75	0.7	0.798	1000	300	75	0.8	0.7841
1000	300	90	0.6	0.8181	1000	300	90	0.7	0.8042
1000	300	90	0.8	0.7903	1000	300	105	0.6	0.8225
1000	300	105	0.7	0.8086	1000	300	105	0.8	0.7947
1000	300	120	0.6	0.8258	1000	300	120	0.7	0.8119
1000	300	120	0.8	0.798	1200	120	60	0.7	0.8657
1200	120	60	0.8	0.803	1200	120	75	0.7	0.8748
1200	120	75	0.8	0.8121	1200	120	90	0.7	0.8808
1200	120	90	0.8	0.8182	1200	120	105	0.7	0.8852
1200	120	105	0.8	0.8225	1200	120	120	0.7	0.8884
1200	120	120	0.8	0.8258	1200	240	60	0.7	0.8495
1200	240	60	0.8	0.827	1200	240	75	0.7	0.8579
1200	240	75	0.8	0.8353	1200	240	90	0.7	0.8634
1200	240	90	0.8	0.8409	1200	240	105	0.7	0.8674
1200	240	105	0.8	0.8449	1200	240	120	0.7	0.8704
1200	240	120	0.8	0.8478	1200	360	60	0.7	0.8815
1200	360	60	0.8	0.8521	1200	360	75	0.7	0.8892
1200	360	75	0.8	0.8598	1200	360	90	0.7	0.8943
1200	360	90	0.8	0.8649	1200	360	105	0.7	0.8979
1200	360	105	0.8	0.8686	1200	360	120	0.7	0.9007
1200	360	120	0.8	0.8713	1400	140	60	0.8	0.8915
1400	140	75	0.8	0.8993	1400	140	90	0.8	0.9045
1400	140	105	0.8	0.9082	1400	140	120	0.8	0.911
1400	280	60	0.8	0.9134	1400	280	75	0.8	0.9205
1400	280	90	0.8	0.9253	1400	280	105	0.8	0.9287
1400	280	120	0.8	0.9313	1400	420	60	0.8	0.9249
1400	420	75	0.8	0.9315	1400	420	90	0.8	0.9359
1400	420	105	0.8	0.9391	1400	420	120	0.8	0.9414

Table C-2. Entering Percent Following with no signalized intersection

Volume (veh/h)	EPF	Volume (veh/h)	EPF	Volume (veh/h)	EPF
220	0.1458	240	0.1584	260	0.1708
440	0.2778	480	0.3003	520	0.3224
660	0.3962	720	0.4262	780	0.4553
880	0.5015	960	0.5366	1040	0.5701
1100	0.5941	1200	0.6321	1300	0.6677
1320	0.6745	1440	0.7135	1560	0.7492
1540	0.7434	1680	0.7817	1820	0.8157

APPENDIX D
VISUAL BASIC CODE

D.1 Calculate Entering Percent Following

```
Sub Calculate_EPF()

    Dim EPF As Double
    Dim VFA_Total, VA, VF1, VF2, VF3, V1, V2, V3 As Double
    Dim VFQ1, VFNQ1, VNQ_a_2, V_a_3, Prtor As Double
    Dim Tq1, S1, C, g1, t As Double
    Dim Percent_Following_1, Percent_Following_2 As Double

    ' set values for some parameters
    S1 = 1800
    V1 = G_Volume
    V2 = G_R_L
    V3 = G_R_L
    C = G_Cycle
    g1 = G_Cycle * G_g_C
    g_c = g1 / C
    Prtor = 0.1
    t = 3

     ' set the propability of headway greater than 3 at the different flow rates
    PF_800 = 0.4647
    PF_1028 = 0.6889
    PF_1333 = 0.7741
```

```
PF_1636 = 0.8654

' tell if the demand is above capacity

If ((V1) / (S1 * g_c)) > 1 Then
    G_Comment = "Over Capacity!"
    G_EPF = 0.99999
    GoTo LAB1
End If

' vehicle following in movement one
Tq1 = V1 / (S1 - V1) * (C - C * g_c)
VFQ1 = S1 * (Tq1 / 3600) - 1

If ((V1) / (S1 * g_c)) = 1 Then
    VNQ_a_2 = V1
Else
    VNQ_a_2 = V1 + V2 * Prtor * (C / (C * g_c - Tq1))
End If

If VNQ_a_2 < 800 Then
    Percent_Following_1 = 1 - Exp(-(t - 0.5) / (3600 / VNQ_a_2 - 0.5))

ElseIf VNQ_a_2 < 1028 Then
    Percent_Following_1 = PF_800 + (VNQ_a_2 - 800) / (1028 - 800) * (PF_1028 - PF_800)

ElseIf VNQ_a_2 < 1333 Then
    Percent_Following_1 = PF_1028 + (VNQ_a_2 - 1028) / (1333 - 1028) * (PF_1333 - PF_1028)

ElseIf VNQ_a_2 < 1636 Then
    Percent_Following_1 = PF_1333 + (VNQ_a_2 - 1333) / (1636 - 1333) * (PF_1636 - PF_1333)
Else
```

```
    Percent_Following_1 = PF_1636

End If

VFNQ1 = V1 * ((C * g_c - Tq1) / 3600) * Percent_Following_1

VF1 = VFQ1 + VFNQ1

' vehicle following in movement Two and Three
V_a_3 = (C / (C - C * g_c)) * (V3 + V2 * (1 - Prtor))

If V_a_3 < 800 Then
   Percent_Following_2 = 1 - Exp(-(t - 0.5) / (3600 / V_a_3 - 0.5))

ElseIf V_a_3 < 1028 Then
   Percent_Following_2 = PF_800 + (V_a_3 - 800) / (1028 - 800) * (PF_1028 - PF_800)

ElseIf V_a_3 < 1333 Then
   Percent_Following_2 = PF_1028 + (V_a_3 - 1028) / (1333 - 1028) * (PF_1333 - PF_1028)

ElseIf V_a_3 < 1636 Then
   Percent_Following_2 = PF_1333 + (V_a_3 - 1333) / (1636 - 1333) * (PF_1636 - PF_1333)

Else
   Percent_Following_2 = PF_1636

End If

VF2 = V2 * (C / 3600) * ((1 - Prtor) * Percent_Following_2 + Prtor * Percent_Following_1)
VF3 = V3 * C / 3600 * Percent_Following_2

' The Entering Percent Following
EPF = (VF1 + VF2 + VF3) / ((V1 + V2 + V3) / 3600 * C)
```

```
LAB1:

End Sub
```

D.2 Extract Average Travel Speed From CORSIM Output

Sub Processing_CORSIM_Output(Common_directory)

```
Dim OutputFile_1, OutputFile_2, OutputFile_3, OriginalFile As String
Dim A, B, C, D, E, file_number As Integer
Dim Switch, key_word, lineoftext As String
Dim i, j, k, point, length_string As Integer
Dim k1 As Double
Dim Array_speed(1 To 1200, 1 To 2), Array_1(1 To 120, 1 To 2) As String
Dim speed, link_info, link_1, link_2, Print_String, Sum_Speed, As String

For A = 400 To 1000 Step 300
        For B = 0.5 To 0.6 Step 0.05
                For C = 60 To 90 Step 15
                        For D = 55 To 75 Step 10
                                For E = 8 To 11 Step 3
                                    FileHeader_1 = CStr(A) + "_" + CStr(B) + "_" + CStr(C) + "_0" + _ CStr(D) + "_" +
                                CStr(E) + "%"

                                    For file_number = 1 To 10
                                        OriginalFile = Common_directory + FileHeader_1 + "_" + CStr(file_number) + ".out"
                                        OutputFile_1 = Common_directory + FileHeader_1 + ".txt"
                                        OutputFile_2 = Common_directory + FileHeader_1 + "_2.txt"
                                        OutputFile_3 = Common_directory + FileHeader_1 + ".csv"

                                        Open OriginalFile For Input As #1
```

```
Open OutputFile_1 For Output As #2

Switch = "Close"
Do Until EOF(1)
        Line Input #1, lineoftext
        lineoftext = Trim(lineoftext)
        key_word = Left(lineoftext, 12)

        If key_word = "VEHICLE MINU" Then
                Switch = "Open"
                i = 1
                GoTo Lab1
        End If
        If key_word = "0SUBNETWORK=" Then
                Switch = "Close"
                GoTo Lab2
        End If
        If Switch = "Open" And i > 5 And i < 51 Then
                Print #2, lineoftext
        End If

Lab1:
        i = i + 1
Loop
Lab2:
Close #1
Close #2

Open OutputFile_1 For Input As #3
Open OutputFile_2 For Append As #4

i = 1
Do Until EOF(3)
```

```
                    Line Input #3, lineoftext
                    lineoftext = Trim(lineoftext)
                    point = InStr(lineoftext, ")")
                    link_info = Left(lineoftext, point - 1)
                    point = InStr(lineoftext, ",")
                    length_string = Len(link_info)
                    link_1 = Trim(Mid(link_info, 2, length_string - point))
                    link_2 = Trim(Right(link_info, length_string - point))
                    point = InStrRev(lineoftext, " ")
                    length_string = Len(lineoftext)
                    speed = Right(lineoftext, length_string - point)

                    If CLng(link_1) >= 1184 And CLng(link_2) <= 1304 _
                       And CLng(link_1) < CLng(link_2) Then
                            Array_1(i, 1) = link_1
                            Array_1(i, 2) = speed
                            i = i + 1
                    End If
            Loop

            For j = 1184 To 1304
                For i = 1 To 120
                    If Array_1(i, 1) = CStr(j) Then
                            Print #4, CStr((j - 1184) * (132 / 5280)) + "," + string_1(i, 2)
                            GoTo Lab3
                     End If
                Next i
Lab3:
            Next j
            Close #3
            Close #4
        Next file_number
```

```
            Open OutputFile_2 For Input As #5
            Open OutputFile_3 For Output As #6

            i = 1
            Do Until EOF(5)
                    Line Input #5, lineoftext
                    lineoftext = Trim(lineoftext)
                    point = InStr(lineoftext, ",")
                    Array_speed(i, 1) = Left(lineoftext, point - 1)
                    Array_speed(i, 2) = Right(lineoftext, Len(lineoftext) - point)
                    i = i + 1
            Loop

            For k = 0 To 3000 Step 25
                    Print_String = ""
                    Sum_Speed = 0
                    k1 = CDbl(k) / 1000

                    For i = 1 To 1200
                            If Array_speed(i, 1) = CStr(k1) Then
                                    Print_String =  Print_String + "," + CStr(Array_speed(i, 2))
                                    Sum_Speed = Sum_Speed + CDbl(Array_speed(i, 2))
                            End If
                    Next i
                    Print #6, Trim(CStr(k1)) + "," + Trim(Print_String) + "," + CStr((Sum_Speed / 10))
            Next k

            Close #5
            Close #6

          Next E
        Next D
Next C
```

```
        Next B
Next A

End Sub
```

D.3 Determine Upstream Effective Length

Sub Getting_Expective_Length(Common_Directory)

```
Dim OriginalFile_1, OriginalFile_2, OutputFile, Print_Head As String
Dim Header_OriginalFile_1, Header_OriginalFile_2 As String
Dim A, B, C, D, E, F, file_number As Integer
Dim lineoftext As String
Dim i, j, k, point, length_string As Integer
Dim String_WithoutSignal(1 To 120, 1 To 2), String_WithSignal(1 To 120, 1 To 13) As String
Dim Diff_1, Diff_2 As Double

For A = 400 To 1000 Step 300
        For B = 50 To 60 Step 5
                For C = 60 To 90 Step 15
                        For D = 55 To 75 Step 10
                                For E = 5 To 11 Step 3
                                        Header_OriginalFile_1 = CStr(A) + "_0" + CStr(B) + "_" + CStr(C) + "_0" + CStr(D) +
"_" +
                                                                CStr(E) + "%.csv"
                                         F = A / B * 100 - A
                                         Header_OriginalFile_2 = "WithoutSignal_" + CStr(A) + "_" + CStr(F) + ".csv"

                                        OriginalFile_1 = Common_Directory + Header_OriginalFile_1
                                        OriginalFile_2 = Common_Directory + Header_OriginalFile_2
                                        OutputFile = Common_Directory + Header_OriginalFile_1+".csv"

                                        Open OriginalFile_1 For Input As #1
```

```
Open OriginalFile_2 For Input As #2
Open OutputFile For Append As #3

j = 1
Do Until EOF(1)
        Line Input #1, lineoftext
        lineoftext = Trim(lineoftext)

        If Len(lineoftext) < 20 Then
                GoTo LabA1
        End If
        point = InStr(lineoftext, ",")

         k = 1
        Do Until point = 0
                String_WithSignal(j, k) = Left(lineoftext, point - 1)
                k = k + 1
                length_string = Len(lineoftext)
                lineoftext = Right(lineoftext, length_string - point)
                point = InStr(lineoftext, ",")
        Loop

        j = j + 1
Loop
LabA1:

j = 1
Do Until EOF(2)
        Line Input #2, lineoftext
        lineoftext = Trim(lineoftext)

        If Len(lineoftext) < 20 Then
                GoTo LabA2
```

```
                End If
                point = InStr(lineoftext, ",")

                k = 1
                Do Until point = 0
                        String_WithoutSignal(j, k) = Left(lineoftext, point - 1)
                        k = k + 1
                        length_string = Len(lineoftext)
                        lineoftext = Right(lineoftext, length_string - point)
                        point = InStr(lineoftext, ",")
                Loop
                j = j + 1
        Loop
LabA2:

Print_Head = CStr(A) + "," + "0." + CStr(B) + "," + CStr(C) + "," + "0." + CStr(D) +  _
                "," + CStr(E) + "%" + ","
 For j = 3 To 12
         For i = 40 To 1 Step -1
        Diff_1 = CDbl(String_WithoutSignal(i + 1, 2)) - CDbl(String_WithSignal(i + 1, j))
        Diff_2 = CDbl(String_WithoutSignal(i, 2) - CDbl(String_WithSignal(i, j)))

        If Diff_2 = 0 Then
                Print_Head = Print_Head + CStr(1 - CDbl(String_WithoutSignal(i + 1, 1))) + ","
                        GoTo LabA3
                End If

        If Diff_1 > 0 And Diff_2 < 0 Then
                Print_Head = Print_Head + CStr(1 - CDbl(String_WithoutSignal(i + 1, 1))) + ","
                        GoTo Lab1A3
                End If

                If Diff_1 > 0 And Diff_2 > 0 And Diff_1 < 5 And Diff_2 < 5 And  _
```

```
                                        Abs(Diff_1 - Diff_2) < 0.5 Then
                                      Print_Head = Print_Head + CStr(1 - CDbl(String_WithoutSignal(i, 1))) + ","
                                            GoTo LabA3
                                      End If
                                Next i
LabA3:
                        Next j

                        Print #3, Print_Head

                        Close #1
                        Close #2
                        Close #3
            Next E
          Next D
        Next C
      Next B
    Next A

End Sub
```

APPENDIX E
PLOTS OF DIFFERENCE IN AVERAGE TRAVEL SPEED FROM TWOPAS

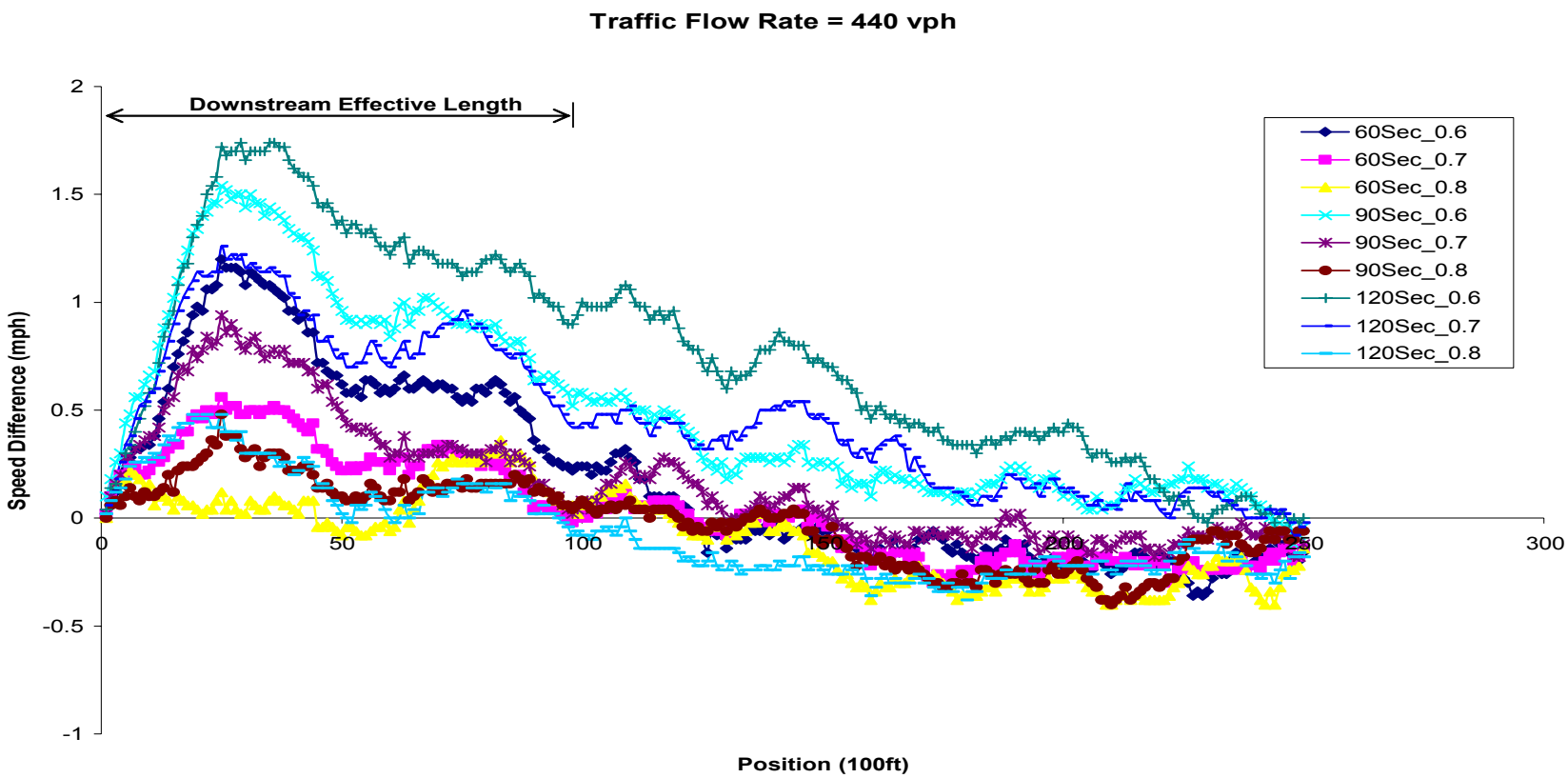


Figure E-1. Difference in the average travel speed with traffic volume = 440 veh

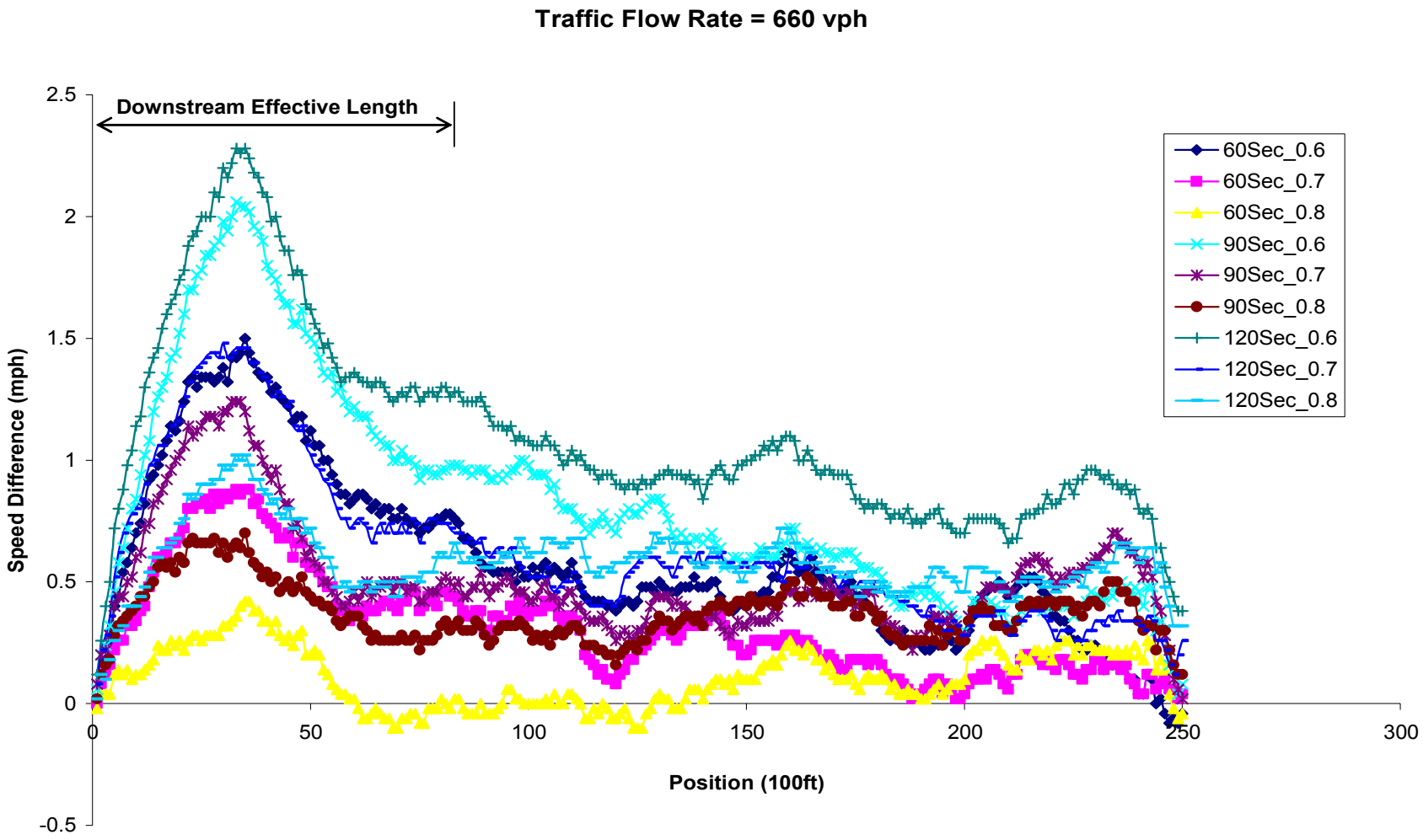


Figure E-2. Difference in the average travel speed with traffic volume = 660 veh/h

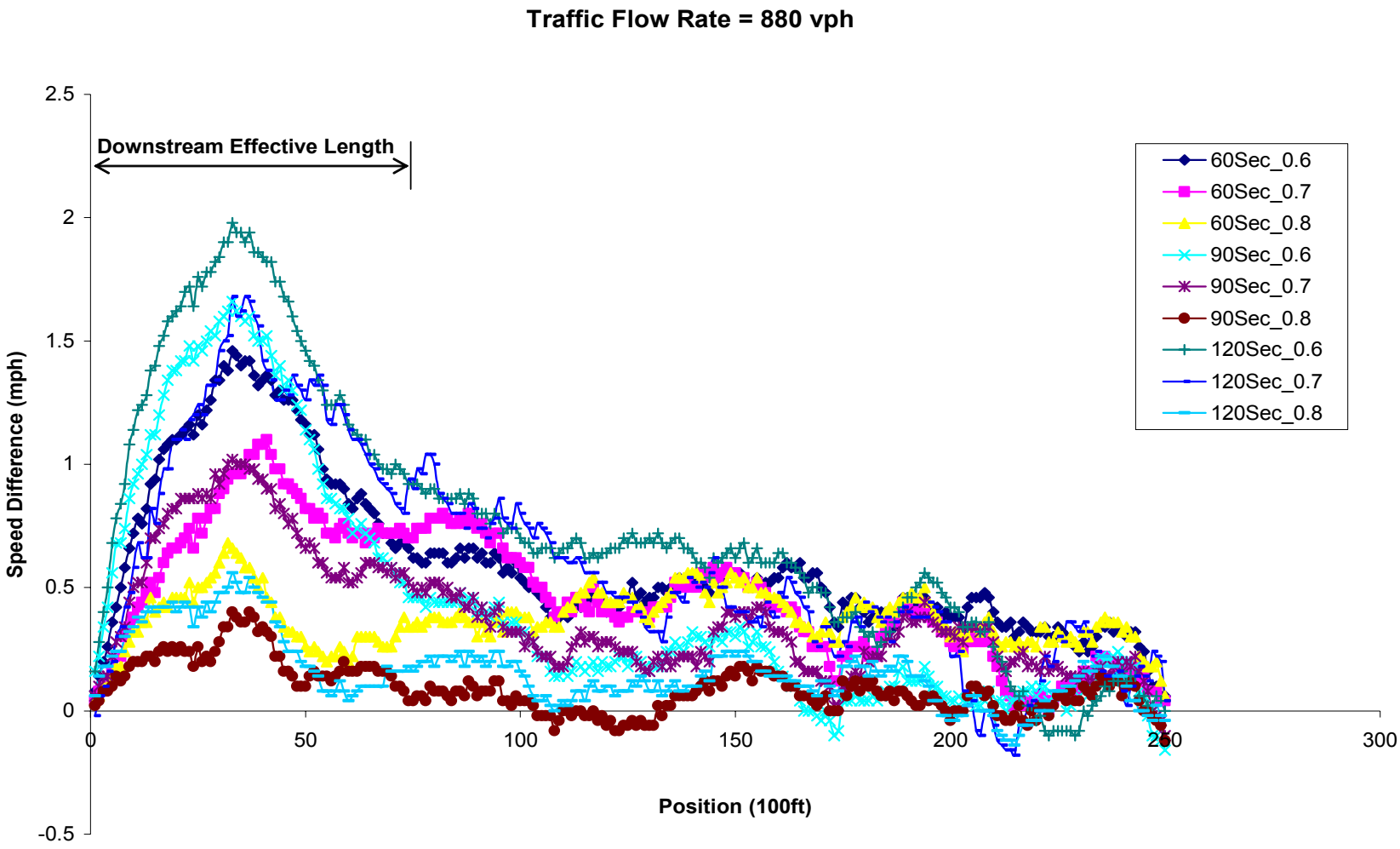


Figure E-3. Difference in the average travel speed with traffic volume = 880 veh/h

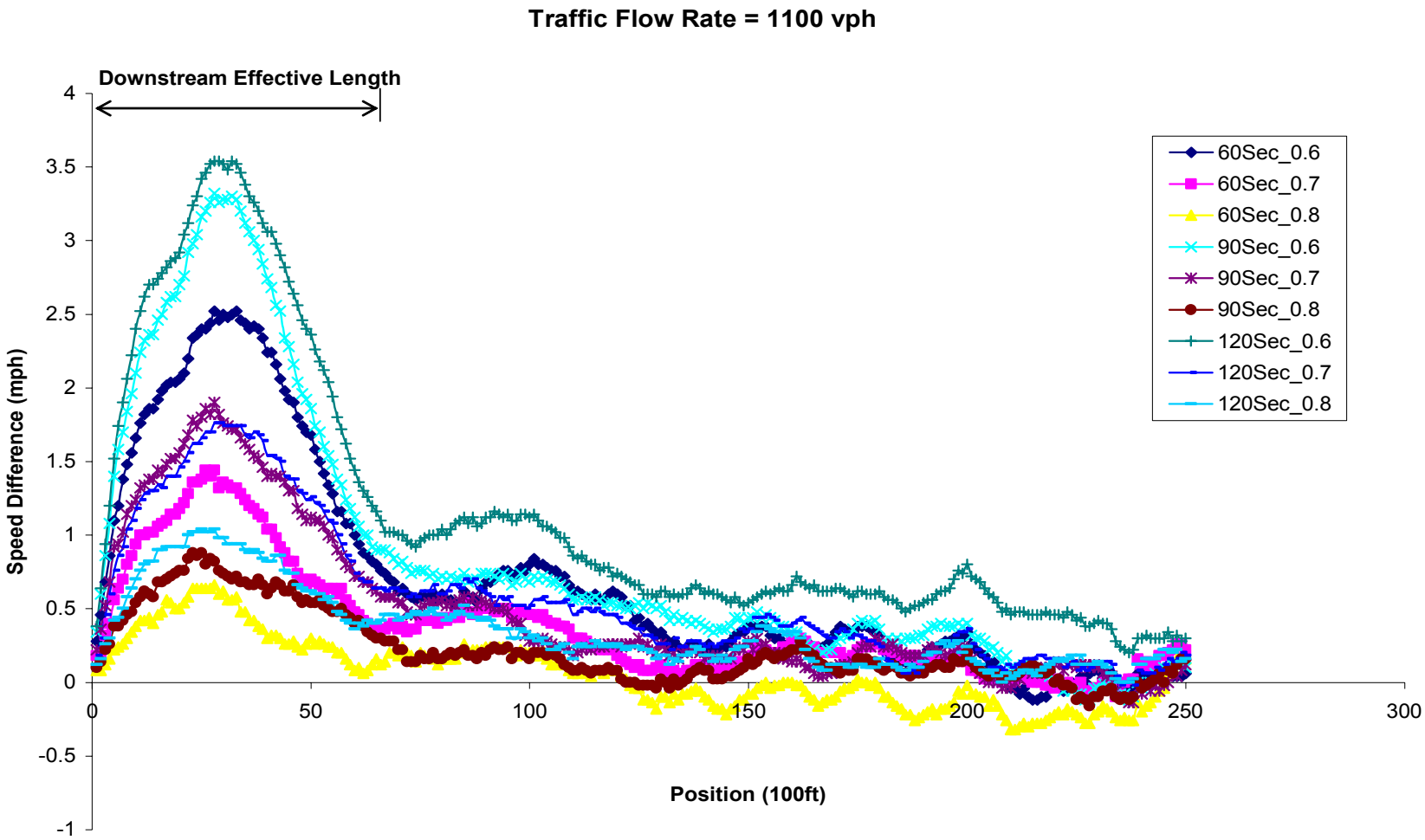


Figure E-4: Difference in the average travel speed with traffic volume = 1110 veh/h

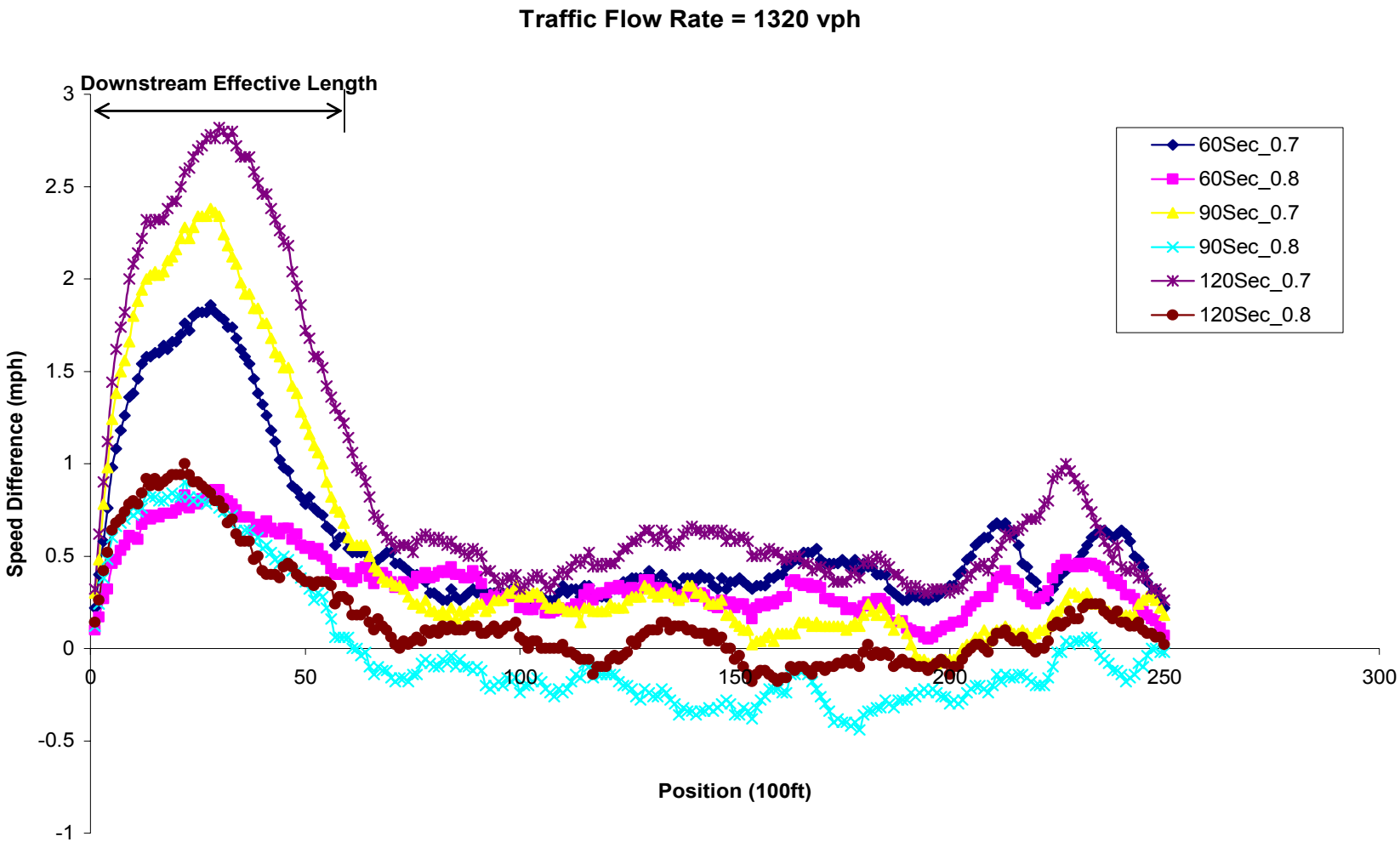


Figure E-5. Difference in the average travel speed with traffic volume = 1320 veh/h

APPENDIX F
PLOTS OF DIFFERENCE IN AVERAGE TRAVEL SPEED FROM CORSIM

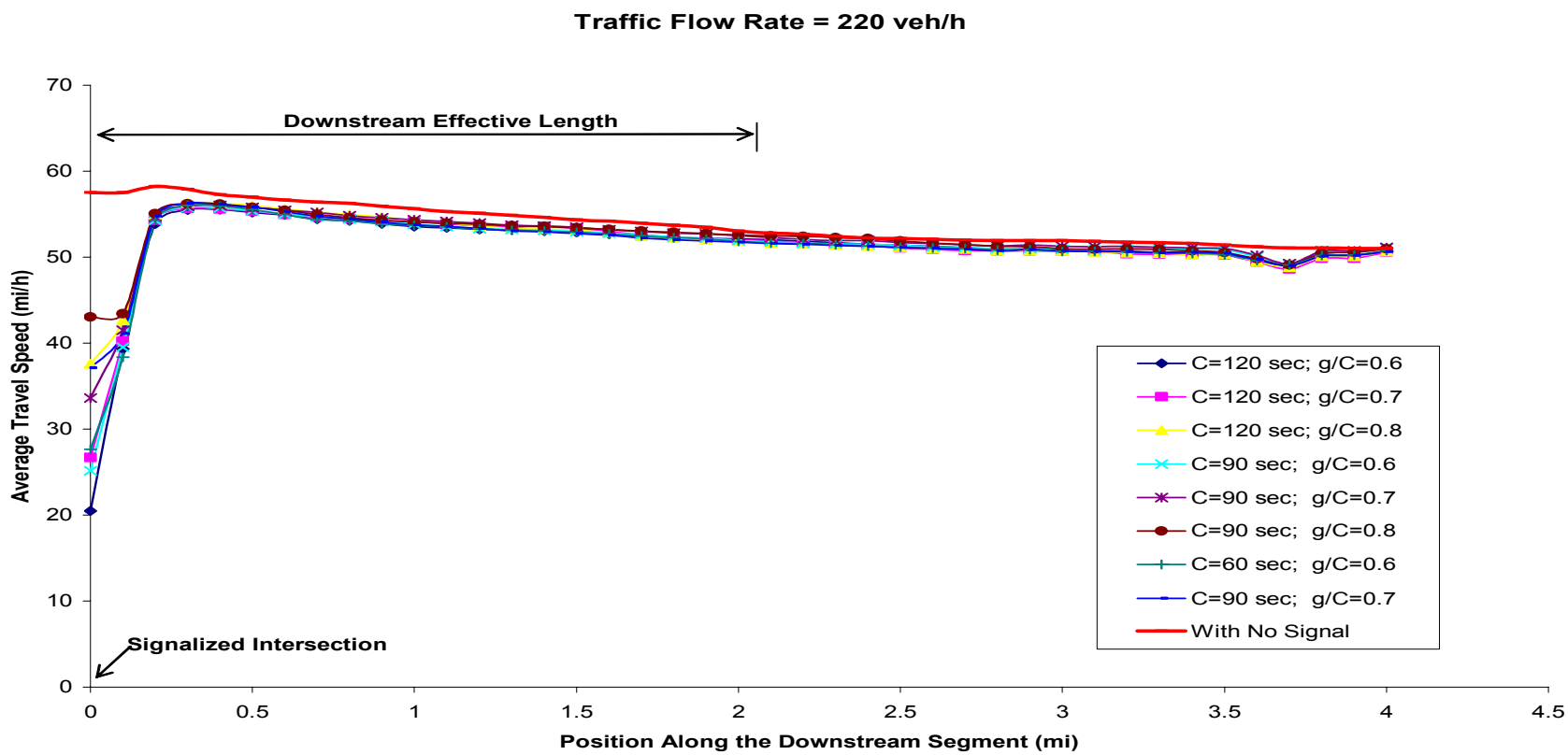


Figure F-1. Difference in the average travel speed with traffic volume = 220 veh/h

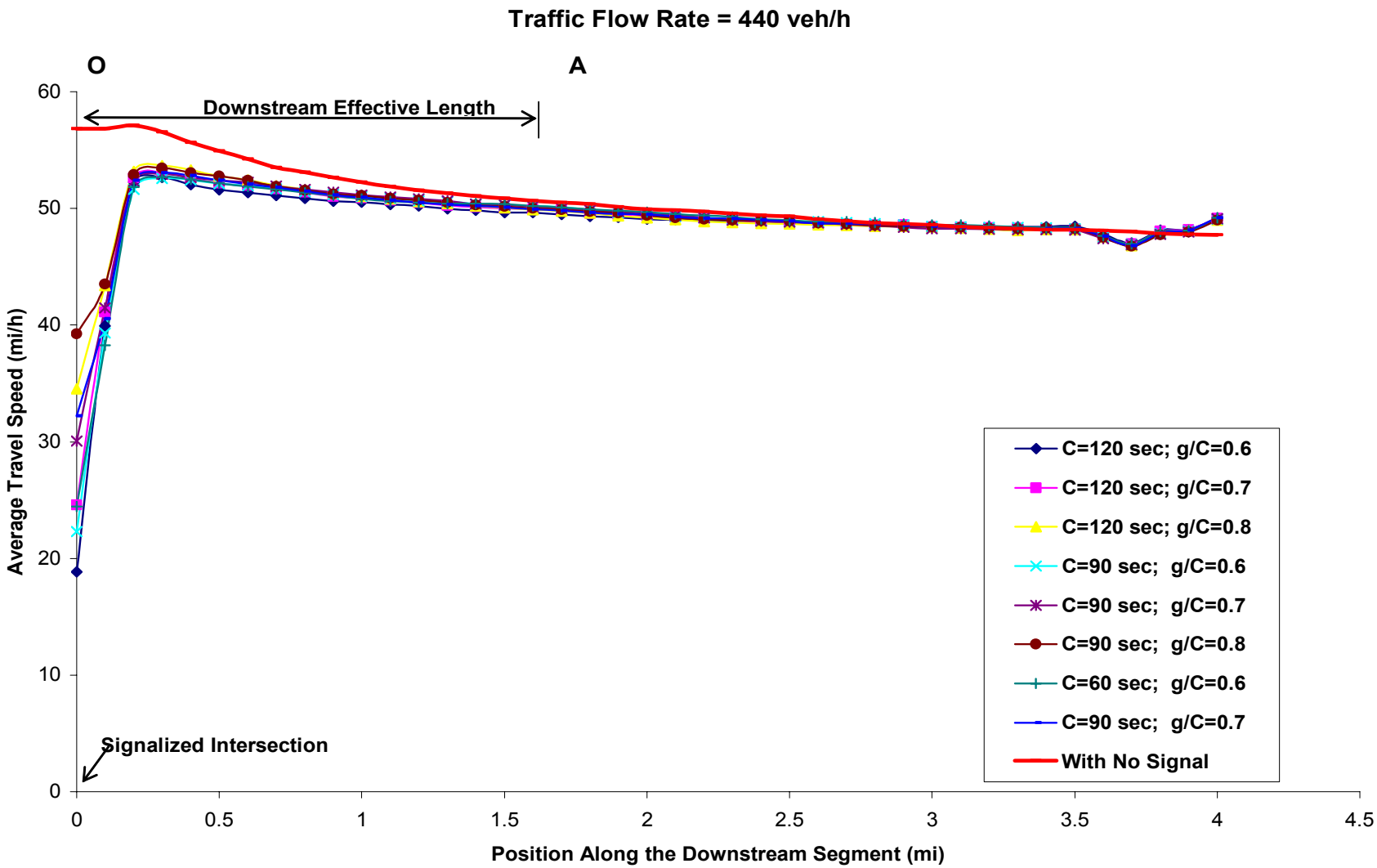


Figure F-2. Difference in the average travel speed with traffic volume = 440 veh/h

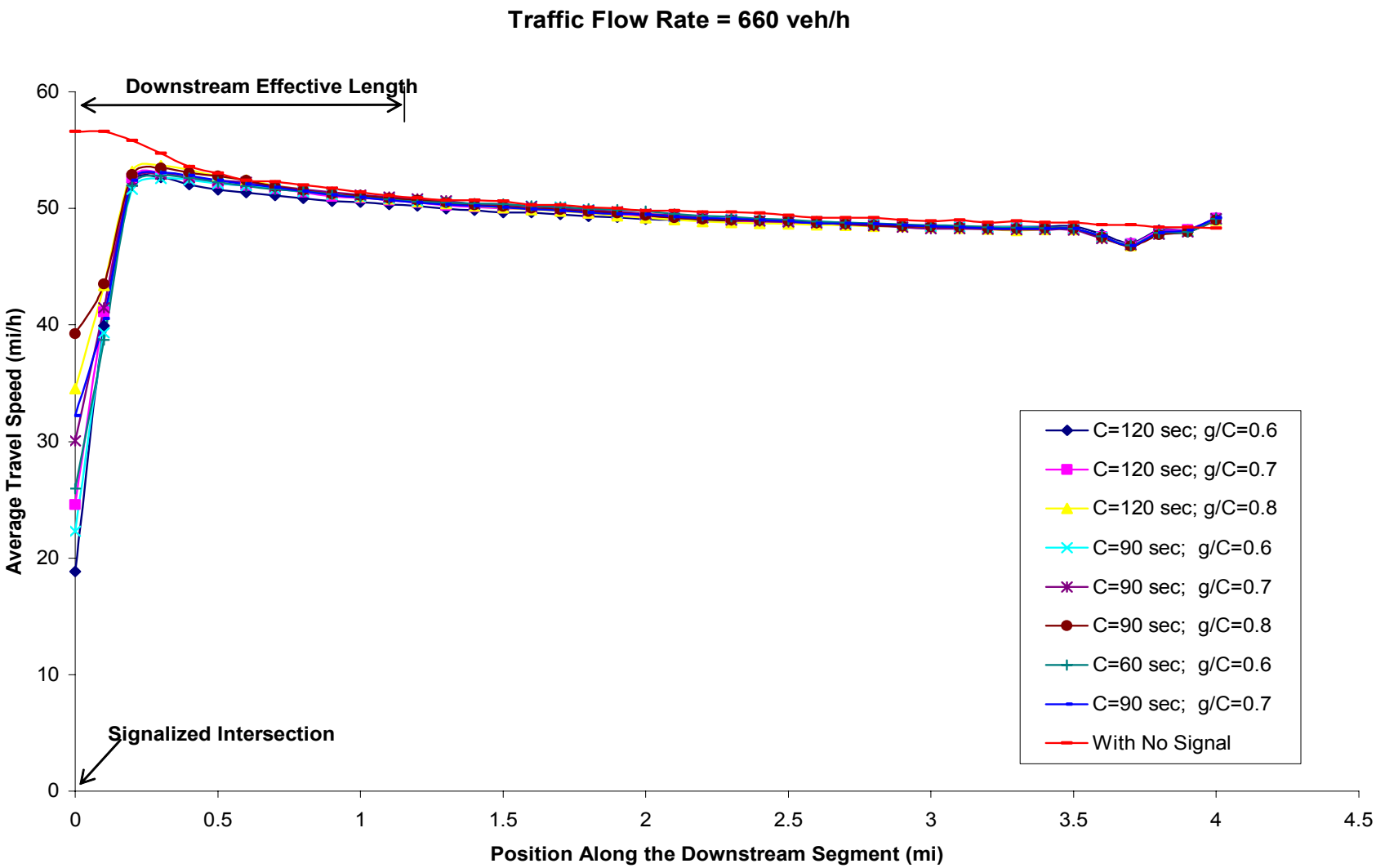


Figure F-3. Difference in the average travel speed with traffic volume = 660 veh/h

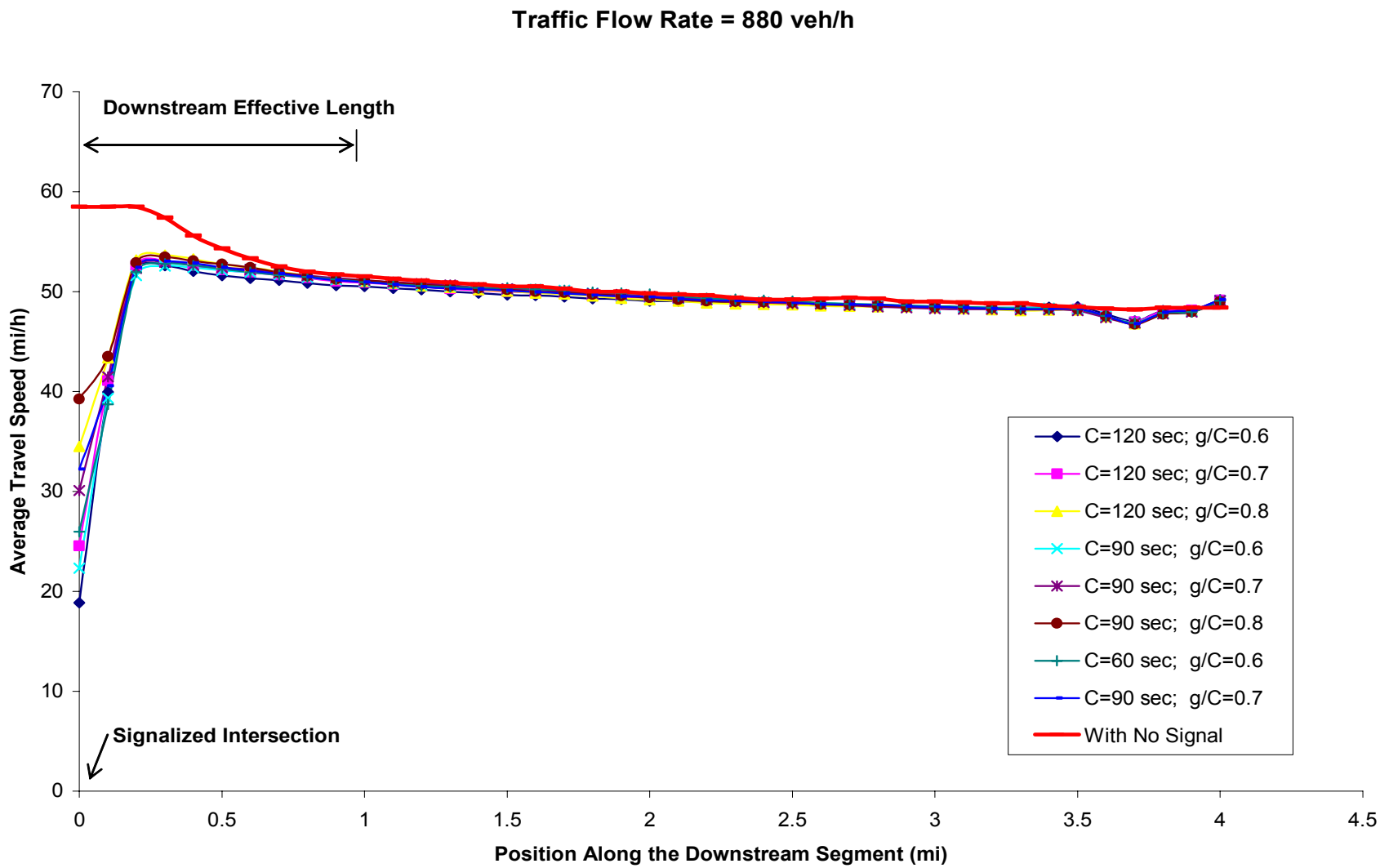


Figure F-4. Difference in the average travel speed with traffic volume = 880 veh/h

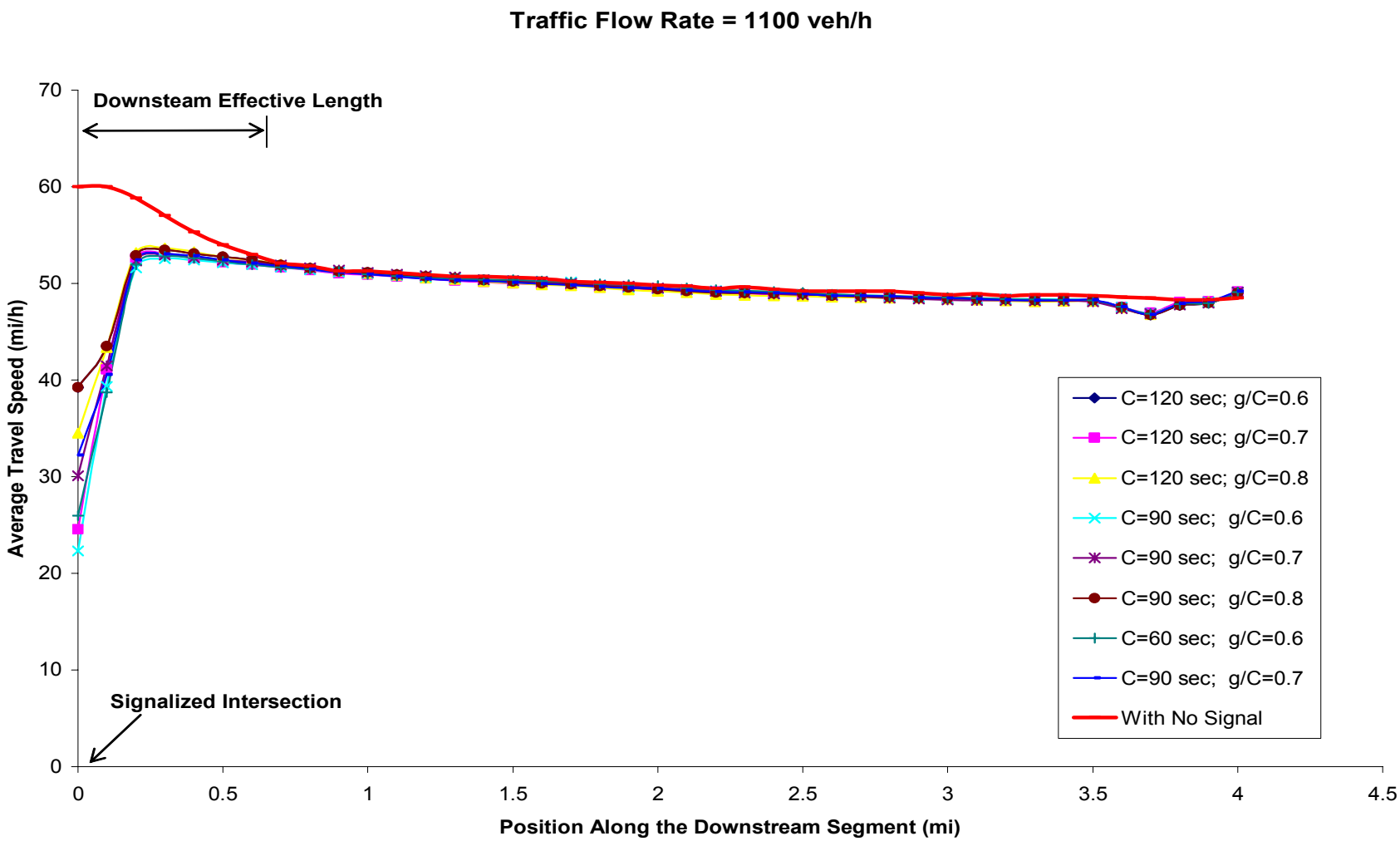


Figure F-5. Difference in the average travel speed with traffic volume = 1100 veh/h

APPENDIX G
NOMENCLATURE TABLE

a : deceleration rate
ATS_1: average travel speed without the effect of a signalized intersection, mi/h
ATS_2: average travel speed within the downstream effective length of the upstream signalized intersection
ATS_d: average traffic speed in the analysis direction
ATS_{pl}: average travel speed for the entire segment including the passing lane (mi/h)
BFFS: Base flow-flow speed for both directions of travel combined
BPTSF: Base percent time-spent-following for both directions of travel combined
c: capacity of lane group
c_L: lane group capacity per lane, veh/lane
$c_{m,x}$: capacity of movement x, veh/h
C: cycle length
d: control delay per vehicle
d_1: uniform control delay assuming uniform arrivals
d_2: incremental delay to account for effects of random arrivals
d_3: initial queue delay
d_A: delay for approach A
d_I: delay per vehicle for the intersection
d_l: computed control delay for the left-turn movements, s/veh
$d_{M,LT}$: delay to major street left-turning vehicles, s/veh
d_r: computed control delay for the right-turn movements, s/veh
d_{Rank1}: delay to Rank 1 vehicles, s/veh
d_t: computed control delay for the through movements, s/veh
D_D: delay at the two-lane highway downstream of the signalized intersection
D_H: delay time per vehicle for the two-lane highway segment, s/veh
D_S: delay time per vehicle for the signalized intersection influence area, s/veh
D_U: delay at the two-lane highway upstream of the signalized intersection
E_T: passenger-car equivalent for trucks
E_R: passenger-car equivalent for RVs
EPP_a: entering percent platooned at location a
f_A: adjustment factor for access-point density
f_{ATS}: adjustment factor for the effect of a signalized intersection on the average travel speed within the downstream effective length, mi/h.
f_G: adjustment factor for approach grade
f_{HV}: adjustment factor for heavy vehicles
f_{Lpb}: pedestrian adjustment factor for left-turn movements
f_{LS}: adjustment factor for lane width and shoulder width
f_{LT}: adjustment factor for left turns in lane group
f_{LU}: adjustment factor for lane utilization
f_{np}: adjustment factor for percentage of no-passing zones

f_p: adjustment factor for existence of parking activity adjacent to the lane group
f_{PA}: supplemental adjustment factor for platoon arrival during the green
f_{pl}: factor for the effect of a passing lane on average travel speed
f_{PTSF}: adjustment factor for percent time-spent-following
f_{Rpb}: pedestrian-bicycle adjustment factor for right-turn movements
f_{RT}: adjustment factor for right runs in lane group
f_W: adjustment factor for lane width
F: a smoothing factor
FFS: free flow speed for two-lane highways
FFS_H: free flow speed for the two-lane highway segment, ft/s
FFS_S: free flow speed for the signalized intersection influence area, ft/s
g: effective green time for lane group
g/C: effective green to cycle length ratio
G: roadway grade
I: upstream filtering factor for platoon arrivals
L: length of the entire facility, ft
L_i: length of segment i
L_A: acceleration length
Len_{eff_down}: downstream effective length of a signalized intersection, mi
Len_{eff_up}: upstream effective length of a signalized intersection, ft
L_H: length of the two-lane highway segment, ft and
L_S: length of a signalized intersection influence area, ft
L_t: total length of analysis facility
L_U: length of two-lane highway upstream of the signalized intersection
L_V: average vehicle length
LOS: level of service of the facility
LOS_i: the level of service of segment i
k_B: second-term adjustment factor related to early arrivals
N: number of lanes in lane group
$p^*_{0,j}$: proportion of Rank 1 vehicles not blocked
P: proportion of all vehicles arriving during green
PTD: percent time-delayed per vehicle
P_{NP}: proportion of vehicles not in platoon
P_P: proportion of vehicles in platoon
P_R: proportion of RVs in the traffic stream
P_{RTOR}: proportion of right turn vehicles turning on red
P_T: proportion of trucks in the traffic stream
PHF: peak-hour factor
PF: progression adjustment factor
PF_2: adjustment factor for effects of progression, veh
q: hourly flow rate of traffic entering the two-lane highway, veh/h
Q: maximum distance in vehicles over which queue extends from stop line, veh
$\overline{Q}$: average queue length, ft
Q_1: first-term queued vehicles, veh,
Q_2: second-term queued vehicles, veh

Q_Q: average queue length
Q_{bL}: initial queue at start of analysis period, veh
r: effective red time
R_p: platoon ratio
s: saturation flow rate for subject lane group
s_L: lane group saturation flow rate per lane, veh/h
S: standard deviation of normal distribution
S_D: average travel speed for two-lane highway downstream of the signalized intersection
S_H: average travel speed for two-lane highway segment, ft/s
S_m: mean speed of the measure sample
S_U: average travel speed for two-lane highway upstream of the signalized intersection
SSD: stopping sight distance
t: headway criteria used to define when vehicles are following, sec
$\bar{t}_P$: mean highway of the vehicles in platoon
$\bar{t}_{NP}$: mean highway of the vehicles not in platoon
t_r: perception-reaction time
T: analysis time period, h (T=0.25 for a 15-min period)
T_t: total travel time
T_{tffs}: total travel time based on FFS
t_{Q1}: time for queue at movement one to dissipate
V: demand volume for the full peak hour in the analysis direction
v: speed at the end of the acceleration cycle, ft/s
v_0: speed at the beginning of the acceleration cycle, ft/s
v_1: movement 1 demand flow rate
v_2: movement 2 demand flow rate
v_3: movement 3 demand flow rate
v_d: passenger-car equivalent flow rate for the peak 15-min period in the analysis direction
$v_{i,1}$: major-street through vehicles in shared lane, veh/h
$v_{i,2}$: major-street right-turning vehicles in shared lane, veh/h
v_l: flow rate of left-turn traffic on the approach, veh/h
v_o: passenger-car equivalent flow rate for the peak 15-min period in the opposing direction
v_r: flow rate of right-turn traffic on the approach, veh/h
v_t: flow rate of through traffic on the approach, veh/h
v_x: flow rate for movement x, veh/h
$v^{a,3}$: flow rate at location A during flow state 3
V_a: total number of vehicle per cycle at location a
VF_1: total number of vehicle following per cycle from movement 1
VF_2: total number of vehicle following per cycle from movement 2
VF_2': number of vehicle following per cycle from movement 2 during state 3
VF_2'': number of vehicle following per cycle from movement 2 during state 2
VF_3: total number of vehicle following per cycle from movement 3
VF_{Q1}: number of vehicles following in movement 1 discharging from a queue
VF_{NQ1}: number of vehicles following in movement 1 discharging when no queue is present

X: v/c ratio
X_L: ratio of flow rate to capacity
Y_c: sum of critical lane group v/s ratios
α : the minimum time headway for vehicles not in platoon
α, β : acceleration model parameters, based on the design vehicle type
u : average service rate
λ : average arrival rate

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BIOGRAPHICAL SKETCH

Qingyong Yu was born on August 12, 1975, in the Liaoning Province of China. He graduated with a Bachelor of Science Degree from the Department of Mining Engineering of the China University of Mining and Technology in 1997. Upon the completion of the undergraduate studies, he enrolled the College of Traffic and Transportation of Southwest Jiaotong University. In 1998, he was chosen as the representative of the graduate school to participate in developing "Chinese Railway Passenger Tickets Distribution and Booking System," which is the largest and most complicated tickets distribution system in the world up to date. This project received first place in the 2000 National Science and Technology Progress Award, the most prestigious award in China; also it is part of the Smithsonian's Archive in the United States. He obtained a Master's Degree in Engineering from the College of Traffic and Transportation of the Southwest Jiaotong University in May 2000. After graduation, he worked for Asiainfo Technologies (China) Inc. as a system engineer and provided technical consulting services for 12 projects, which covered more than five major areas in China (a potential market of 300 million users). In 2000 he was the only new employee in this company to achieve the Excellent Employee Award.

In August 2001, he earned a graduate research assistantship from Mississippi State University and begun his graduate study in the Department of Civil Engineering. In January 2003, he transferred to the Department of Civil and Coastal Engineering of the

University of Florida and obtained a Master of Engineering Degree in Civil Engineering in December 2003.

After that, he continued his PhD program in traffic engineering. He was awarded Civil and Coastal Engineering (CCE) Alumni Fellowship to complete his studies at University of Florida. Meanwhile, he obtained the Outstanding Academic Achievement Award from University of Florida in 2003, 2004 and 2005.