TWO-LANE HIGHWAY SIMULATION AND ANALYSIS

By

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This work is dedicated to my parents and friends.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Official (represents the book *A policy on Geometric Design of Highways and Streets* by AASHTO in this document)
APL	Average platoon length
ATS	Average travel speed
DT	Driver type (defined by CORSIM)
DNTCP	Distance needed to complete the pass
DTC	Distance to collision (with the oncoming vehicle)
DTP	Desire to pass
EB	Eastbound
EPF	Entering percent following
FFS	Free-flow speed
HCM	Highway Capacity Manual
HCS	Highway Capacity Software
LOS	Level of service
MUTCD	Manual on Uniform Traffic Control Devices for Streets and Highways
PD	Percent delay
PFFS	Percent of free-flow speed
PHF	Peak hour factor
PI	Percent impeded
PSD	Passing sight distance
PTSF	Percent time-spent-following
RT	Record type (utilized in CORSIM)
TNTCP	Time needed to complete passing

TSIS	Traffic Software Integrated System (an integrated development environment that contains CORSIM)
TSWTP	Time spent wanting to pass
WB	Westbound
WTMO	Willingness to move over

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With the rapid growth of urban areas and the corresponding traffic demands, rural areas are experiencing significant growth as well. This growth is now resulting in congestion on facilities that did not have any before. One area that is becoming a concern, particularly in Florida, is rural areas transitioning into more developed areas. Access to these areas is usually by two-lane highways, and sometimes, there may be an occasional traffic signal within such facilities.

Appropriate analysis tools are very important for researchers to evaluate the operations on complex two-lane highways. However, the current state-of-the-art two-lane highway simulation tool, TWOPAS, is not capable of modeling signalized intersections within a two-lane highway facility. Meanwhile, the two-lane highway analysis procedure in the current Highway Capacity Manual (HCM) does not account for interruptions to the flow, such as from signals. To further investigate the traffic operation within these areas, a simulation tool and a facility-level analysis methodology are desired.

In this study, two-lane highway modeling capability is incorporated into CORSIM by making the necessary additions and modifications to the existing CORSIM software

code base. The primary issues addressed here are modeling passing maneuvers in an oncoming lane or a passing-lane section. Preliminary testing in this part showed that the results produced by the developed two-lane highway modeling logic are reasonably consistent with expected traffic flow theory and field observations discussed in the literature.

Based on the new modeling capability, an analytical methodology for analyzing two-lane highway facilities with various features was developed based upon the previous study done by Yu and Washburn (2009). Specifically, the vehicular data generated by CORSIM was used to distinguish the influence area of a signalized intersection from basic two-lane highway segments to accomplish facility segmentation. A two-lane highway facility combined with signalized intersections can then be evaluated based on the aggregate service measure(s) over different types of segments. The service measure percent delay was examined and verified for its efficacy in assessing the operations on complex two-lane highway facilities, especially the ones with signalized intersections.

CHAPTER 1
INTRODUCTION

1.1 Background

With the rapid growth in urban areas and the corresponding traffic demands, rural areas are experiencing significant growth as well. This growth is now resulting in congestion on facilities that did not have any before. One area that is becoming a concern, particularly in Florida, is rural areas transitioning into more developed areas. Access to these areas is usually by two-lane highways. In some cases, there may be an occasional traffic signal and possibly segments of multilane highway along such facilities.

Managing growing traffic demands is always a task for transportation planners and engineers, and appropriate analysis tools are very important for researchers to do such analysis. However, no analysis tool exists at present for analyzing two-lane highway facilities with occasional intersections. The signal spacing and other general characteristics of these roadways do not fit with the analysis criteria for signalized arterials in the current Highway Capacity Manual (TRB, 2010), and the two-lane highway analysis procedure does not account for interruptions to the flow, such as from signals.

1.2 Problem Statement

To further investigate the traffic operation within these areas, a simulation tool is required at first, and then a facility-level analysis methodology can be developed.

The current state-of-the-art tool in two-lane highway simulation is a software program named TWOPAS. This program performs a microscopic and stochastic simulation, and provides the ability to include considerable detail about two-lane

segments in the modeling process. However, TWOPAS is not capable of modeling signalized intersections within the modeled two-lane highway facility. Meanwhile, several other simulation programs capable of simulating two-lane roads with signalized intersections (e.g., CORSIM), cannot simulate passing by utilizing the lane in the opposing direction, which is the most distinctive and significant traffic operation feature of two-lane highways. Therefore, a simulation tool that is capable of modeling the combination of two-lane highway segments and signalized intersections is desired.

Afterwards, an analytical methodology for analyzing two-lane highway facilities with various features – particularly the combination of two-lane highway segments and signalized intersections can be developed based on the simulation program developed in the first stage. The new methodology uses the previous study done by Yu and Washburn (2009) as the starting point. As the previous methodology was developed based on the hybrid simulation approach, it is expected to be validated and updated based on the new simulation program with the capability of modeling both two-lane highway segments and signalized intersections.

1.3 Objective and Tasks

The objective of this study is to develop a simulation tool that is capable of modeling two-lane highway segments with various features, and then based on it, to develop an analytical methodology for analyzing complex two-lane highway facilities, particularly the combination of two-lane highway segments and signalized intersections. The following tasks are carried out to accomplish the objective of this study.

Task 1: Develop the logic models for simulating passing maneuvers on two-lane highways, including vehicle passing in opposing lane and vehicle passing in passing lane.

Task 2: Develop mechanisms for combining two-lane highway segments and various features, such as signalized intersections, passing lanes, etc.

Task 3: Add new inputs and outputs to CORSIM for allowing users to define inputs and obtain outputs as desired.

Task 4: Review the methodology developed by Yu and Washburn (2009), and set up experimental designs for analyzing the effect of a signalized intersection on a two-lane highway facility.

Task 5: Run the simulation experiments set up in Task 4, and prepare the simulation data for model development.

Task 6: Derive the models for facility segmentation that is part of the new methodology by using a statistical approach on the basis of the simulation data obtained in Task 5.

Task 7: Examine the efficacy of percent delay as an appropriate service measure for two-lane highway facilities.

Task 8: Develop the procedure of the analytical methodology and validate the methodology.

With this study done, a simulation tool that is capable of modeling the combination of two-lane highway segments and signalized intersections becomes available. And together with the new analytical methodology, traffic operations on complex two-lane highways (e.g., two-lane highway with occasional signalized intersections) can be analyzed. As the new capability is incorporated in CORSIM, it will allow users to build comprehensive traffic networks including a variety of facility types for different purposes of analysis.

1.4 Chapter Organization

This document consists of four chapters beyond the Introduction Chapter. Chapter 2 gives an overview of the previous studies on two-lane highway simulation tools, analytical methodology for evaluating two-lane highway operations, and traffic models of two-lane highway operations. Chapter 3 describes the development approach for the new two-lane highway simulation tool in detail, including the logic and models employed in CORSIM to facilitate two-lane highway modeling. Chapter 4 is devoted to the development of the analytical methodology for analyzing two-lane highway facilities including signalized intersections. Finally, the conclusions are summarized and the future study is proposed in Chapter 5.

CHAPTER 2
LITERATURE REVIEW

This chapter summarizes the current achievements in two-lane highway simulation development, analytical methodology for evaluating two-lane highway operations, traffic models of two-lane highway operations, and field data collection. The overview of the literature in each aspect is given in the following sections.

2.1 Two-Lane Highway Simulation Tools

This section summarizes the development of two-lane highway simulation tools since the 1960s. The material is organized chronologically in order to facilitate a better understanding of the development of the two-lane highway simulation tool in the past few decades.

2.1.1 Simulation of Two-way Traffic on Isolated Two-Lane Road (1967)

Paul Warnshuis (1967) constructed a computer simulation that modeled each individual vehicle's behavior directly. This simulation was intended to serve as a tool helping develop a theoretical description of the interaction between the two lanes and how that interaction influences the traffic flow in each lane. The simulation was coded in Fortran IV, and it assumed that the two-lane road extends infinitely in both directions by using a two-lane circular track and does not have any restrictions on speed and passing. The inputs, the flow rate in each lane, the distribution of the desired speed, the initial ordering of vehicles, and the initial spacing of vehicles could be specified by users. Each vehicle, based on other assumptions, travels at a fixed desired speed except for the following or passing condition.

Passing maneuvers in the simulation were governed by the rules specified in the paper. A car that intends to pass another car may do so only if its leader has a relatively

lower desired speed, the oncoming vehicle is far enough for the vehicle to complete pass and the gap in front of the passed vehicle is sufficient for the vehicle to return to the normal lane after passing.

Several other constraints were also made, such as the determination of the safe distance ahead of the passed vehicle and to the first oncoming vehicle when the passing is completed, that a vehicle may pass only one vehicle at a time, and that a vehicle may be passed by only one vehicle at a time.

The findings from the simulation were that: 1) changes in the distribution of the desired speed seemed to not affect the relationship between average speed and desired speed, 2) the average speed decreased towards the minimum desired speed when the density increased, and 3) the number of platoons did not increase significantly as the density increased.

However, the logic of the passing maneuvers described by Warnshuis seems too simple to replicate reality because it only considers three factors when deciding whether a vehicle initiates passing or not. It also relied on several other unrealistic assumptions, such as there being the same density for both directions, that the vehicles have a uniform length, that vehicles can only accelerate and decelerate at 4 mi/h/s, etc.. These assumptions also narrowed the scope of application.

2.1.2 TRARR (1980s)

TRARR (TRAffic on Rural Roads) was developed in the 1970s and 1980s by the Australian Road Research Board and designed for two-lane rural highways. TRARR is also a microscopic simulation model tracking each generated vehicle as it travels through the whole segment. The interactions between a vehicle and road conditions or other traffic are determined by that vehicle’s driver behavior and performance. TRARR

provides more user-definable variables in order to allow one to create scenarios with different road and traffic conditions. Koorey (2002) stated that TRARR uses four main input files to describe the situation to be simulated:

- ROAD is used to define a two-lane highway section with desired road geometry, auxiliary (passing) lanes, and no-passing lines;
- TRAF includes the information on traffic volume and composition, simulation time and vehicle speeds;
- VEHS contains operating characteristics of vehicle fleets, such as engine power, mass and fuel consumption;
- OBS functions as a detector input file, which specifies the locations along the highway where data is to be recorded.

The outputs produced by TRARR are summarized in the output file OUT. The major outputs reported are mean speed, travel time, frustration (via time spent following), and VOC (Vehicle Operating Costs) benefits resulting from passing. Koorey also pointed out that TRARR can also be used to evaluate the benefits of adding passing lanes depending on the various spacing between passing lanes.

As the simulation with more details and flexibility than its predecessors, TRARR lacks the ability to handle varying traffic flows down the highway yet, particularly due to major side roads or signalized intersections.

2.1.3 TWOPAS (1998)

The latest version of TWOPAS (TWO-lane PASsing) was revised by Leiman et al. in 1998, as the basis for the analysis procedure modifications in the Highway Capacity Manual 2000 (TRB, 2000). Currently, the TWOPAS simulation model has been built in to the Traffic Analysis Module (TAM) of the Interactive Highway Safety Design Model (IHSDM) (ITT Corporation, 2007). As a microscopic simulation model for two-lane, two-way highways, the position of each vehicle simulated in TWOPAS is reviewed every

second. The manner of each vehicle's travel is determined by the models replicating the real world.

Unlike TRARR, TWOPAS can allow users to specify up to 13 vehicle types, percent of traffic platooned, and distribution of desired speeds for unimpeded driver speed choices. Field data were used to model 1) variations in driver performance and preferences; 2) decisions of initiating, continuing or aborting passing maneuvers, and decision of returning to normal lanes; 3) lane choice, lane changing and passing in a passing-lane section. It should be noted that the IHSDM documentation states that by experience a directional flow rate of greater than 1700 veh/h usually causes congestion that stops the program (ITT Corporation, 2007).

As for output, a very detailed output file will be generated when simulation runs complete. It includes simulated flow rate, percent time-spent-following, average travel speed, trip time, traffic delay, geometric delay, total delay, number of passes, vehicle-kilometers traveled, and total travel time.

Similar to TRARR, TWOPAS does not currently simulate traffic turning on or off the highway at intersections and driveways. And since TWOPAS was developed with U.S. data, it is considered to be generally more applicable for modeling conditions in the U.S. than TRARR.

2.1.4 SLX-Based Two-Lane Road Simulation Model (2001)

Lemessi (2001) developed a car-following and lane-changing microscopic simulation model of a two-lane road section in SLX (Simulation Language with eXtensibility) to qualify and quantify the environmental impact on traffic.

Based on the car-following model in this paper, a driver tries to maintain his desired speed if the distance to its leader is greater than the driver-specific critical

value. If this distance headway becomes less than the critical value, the driver decides whether to pass the slower leader or to decelerate and follow the slower leader at the lower speed. Lemessi indicated that these decisions are made based on the driver's aggressiveness, which is his desired speed. Desired speed is link-dependent and assigned to each vehicle individually, following a user-definable normal distribution over the link average free-flow speed. Users can also define the parameters related to vehicle type, and lane and flow regime that determine the uniform distribution used to generate entry vehicles.

Another model developed in this study is the lane-changing model, which considers both deterministic and stochastic psycho-physical aspects. It is stated that a vehicle can be in three lane-changing scenarios: 1) the driver perceives a slower vehicle ahead in its free-flow condition and changes lane without decelerating; 2) the driver stops the conscious braking and changes lane at the speed less than his desired speed; 3) the driver changes lane at an approximately same speed as its leader when in the following mode. An overtaking factor is assigned to each vehicle for comparing with a given threshold value to prevent the over-frequent lane changes. The threshold values can be specified for different vehicle types. If the overtaking factor is greater than the threshold value, several checks will be made to determine whether the driver can improve the present situation by changing lane or not. The simulation model does not allow the same vehicle to execute two consecutive passing maneuvers less than 15 seconds apart.

Two output files are provided by this simulation model. One contains records of each vehicle's information during each simulation run, and the other one contains

information on random variables, which include running time, lost time, macroscopic flow, macroscopic density, and macroscopic speed.

Lemessi's work reliably creates traffic operations on two-lane highways in the simulation world, but it is not comprehensive enough to simulate the various features that are typical to two-lane highways, such as passing lanes and isolated intersections.

2.1.5 TWOSIM (2006)

Kim (2006) indicated that it is difficult to determine capacity for a two-lane highway from field data because finding two-lane highway sites operating at or near capacity is not easy. Therefore, he developed a new microscopic simulator with MATLAB 6.5, called TWOSIM. TWOSIM is intended to be used for estimating the capacity for two-lane, two-way highways under a variety of prevailing geometric and traffic conditions. Field data, such as vehicle count, speed, vehicle length, headways, directional distribution, lane width, shoulder width, and terrain type were collected for the calibration of input parameters in the newly developed simulation model.

TWOSIM was developed in three stages: 1) TWOSIM I with a straight tangent level segment, no opposing traffic, no additional traffic entering or exiting within a given segment, and no passing maneuvers; 2) TWOSIM II with an additional algorithm for overtaking behavior; 3) TWOSIM III with more advanced options that consider the presence of a driveway, horizontal curves, grades along a given segment, and truck traffic. The logic for the primary operations on two-lane highways (i.e., car-following, passing) is supported by Gipp's car-following model, AASHTO (A policy on Geometric Design of Highways and Streets) criteria of minimum passing sight distance, and McLean's modified-time hypothesis as to the stimulus perceived by drivers in making stream-crossing gap-acceptance decisions.

The results of Kim's experiments indicate that: 1) the capacity of two-lane, two-way highways is found to be a function of average free-flow speed; 2) the presence of a passing zone does not affect capacity; 3) the directional capacity is independent of the opposing flow rate; and 4) the capacity varies due to the variations in the number of trucks and driveways, and changes in horizontal curvature and grade. Therefore, he suggested that the HCM should present varying directional capacities as a function of road and traffic conditions. He also suggested considering the influence of large trucks in Gipp's car-following model.

2.1.6 Simulation of Mixed Traffic Flow on Two-Lane Road (2008)

Based upon the field data collected at different locations of two-lane roads in India, Dey et al. (2008) developed a simulation program in the Visual Basic language. The authors indicated that this simulation can be used to determine the capacity of a two-lane road and to study the effect of traffic mix on capacity and speed. Also, the effect of directional split on capacity and the effect of slow moving vehicles on capacity can be estimated by using this simulation.

For the inputs, Poisson distribution was used to generate the arrival pattern according to the data collected and the normal distribution for the basic desired speed (the maximum speed the specific vehicle can reach) assignment to each vehicle. Also, the average rate of acceleration during a passing was found to be exponential function of passing time from the field data. Additionally, the simulation allows users to modify the percent compositions of the six categories of vehicles.

The passing logic in this simulation program is similar to others, but it was presented in the sequential states that describe a passing maneuver executed by a passing vehicle from beginning to end. For the practical reasons, the maximum number

of vehicles that can be passed by a vehicle at a time is 3. At the same time, an initiated passing can be rejected by the driver as well in the simulation. However, the critical point beyond which a passing vehicle cannot reject passing was not presented in the paper. For the vehicle traveling at its free-flow speed, the simulation will evaluate the possibility of flying passing when it is catching up with a slow vehicle. Although the critical position where the vehicle has to decide whether to execute flying passing or to decelerate and follow the leader was discussed in the paper, the details of evaluating the possibility of flying passing was not provided.

2.2 Analytical Methodologies for Analyzing Two-Lane Highway Operations

This section provides an overview of the existing methodologies for analyzing traffic operations on two-lane highways, as well as other performance measures proposed in the previous studies for assessing level of service on two-lane highways.

2.2.1 HCM Methodology and Related Studies

2.2.1.1 HCM methodology

Chapter 15 of the Highway Capacity Manual (HCM) (TRB, 2010) provides an operations-analysis methodology, which allows for a single direction of traffic flow for two-lane highway segments. The analysis procedures can also be applied to segments with the general terrain classification of level or rolling. It is stated that the directional methodology must be applied to segments in mountainous terrain or with grades of 3% or more with a length of 0.6 miles or more, and/or segments with a passing lane. The performance measures of Average Travel Speed (*ATS*) and Percent Time-Spent-Following (*PTSF*) are used to determine the level of service (LOS) for Class I two-lane highways, on which drivers care about mobility the most. Only *PTSF* is employed to determine the LOS for Class II two-lane highways with drivers' less attention to efficient

mobility. For the newly added Class III two-lane highways, which represent roadways in rural developed areas or scenic roadways, percent of free-flow speed (*PFFS*) is the primary service measure. The LOS criteria for two-lane highways are presented in Table 2-1. LOS F indicates the situation when the demand exceeds the capacity.

The calculations for *ATS* and *PTSF* for directional two-lane highway segments without a passing lane are given by Equations 2-1 to 2-4.

$$ATS_d = FFS_d - 0.00776(v_d + v_o) - f_{np} \tag{2-1}$$

$$BPTSF_d = 100\left(1 - \exp\left(a v_d^b\right)\right) \tag{2-2}$$

$$PTSF_d = BPTSF_d + f_{np} \tag{2-3}$$

$$PFFS_d = \frac{ATS_d}{FFS_d} \times 100 \tag{2-4}$$

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-minute period in the analysis direction (pc/h)

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

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

$BPTSF_d$ = base percent time-spent-following in the direction analyzed

$PTSF_d$ = percent time-spent-following in the direction analyzed

$PFFS_d$ = percent of free-flow speed in the direction analyzed

For segments that contain passing lanes, the *ATS* calculated and *PTSF* values are further adjusted by Equations 2-5 and 2-6.

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

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + L_d + f_{pl} L_{pl} + \left(\frac{1+f_{pl}}{2} \right) L_{de} \right]}{L_t} \tag{2-6}$$

where

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

$PTSF_{pl}$ = percent time-spent-following for the entire segment including the passing lane

L_t = total length of analysis segment (mi)

L_u = length of two-lane highway upstream of the passing lane (mi)

L_d = length of two-lane highway downstream of the passing lane and beyond its effective length (mi)

L_{pl} = length of the passing lane including tapers (mi)

L_{de} = length of the passing lane

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

However, the analysis methodologies in Chapter 15 of the HCM 2010 are not suitable for analyzing situations where two-lane highways are combined with occasional signalized intersections. It is only suggested that isolated signalized intersections on two-lane highways can be evaluated with the methodology in Chapter 18 of the HCM

2010, “Signalized Intersections”. If two-lane highways are located in urban and suburban areas with multiple signalized intersections at spacing of 2.0 mi or less, the methodologies of Chapter 16 of the HCM 2010, “Urban Street Facilities”, and Chapter 17 of the HCM 2010, “Urban Street Segments” will be applied.

2.2.1.2 NCHRP Project 20-7 (160)

Harwood, et al. (2003) noticed that the users encountered two issues regarding the two-lane highway operational analysis procedure in the HCM 2000. This resulted in a study done by Harwood and his associates to address these issues. The first issue is that users found lower service volumes for two-lane highway segments in the HCM 2000 than in the HCM 1997. Harwood, et al. determined that overestimating percent time-spent-following (*PTSF*) by the HCM 2000 methodologies resulted in this issue, which was proved by the mathematical comparisons. Based on the cause of the issue, they suggested revising the adjustment to directional *PTSF* for percentage of no-passing zones, as follows:

$$f_{np} = f_{adj}\left(\frac{v_d}{v_d + v_o}\right) \tag{2-7}$$

where f_{adj} is the adjustment factor for directional split and no-passing zones.

As a result, the equation for estimating directional *PTSF* should be revised as:

$$PTSF_d = BPTSF_d + f_{adj}\left(\frac{v_d}{v_d + v_o}\right) \tag{2-8}$$

With this solution, the undesirable difference between the two-way and directional segment procedures can be eliminated.

The second issue identified by Harwood, et al. is that users found that the classes of two-lane highways defined in the HCM procedures are not applicable to two-lane

highways with various features, such as a reduced speed limit when approaching urban or suburban areas, and scattered signals on the urban extension of a two-lane highway. The researchers believed that such areas of interest (termed as “developed areas” in the report) are difficult to refer to any class of two-lane highways due to complex driver expectations. The field observations collected on the two-lane highways in Florida verified their opinions. To improve the HCM procedures, the researchers suggested that two-lane highways in developed areas can be defined as a new type for the purpose of analysis, either appearing as a new class in Chapter 20 of the HCM 2000 or in a new chapter. Another proposed solution is to expand Chapter 15, “Urban Streets” of the HCM 2000 so that it contains an analysis procedure for these “developed arterials”. The details of each approach discussed in the report are briefly summarized as follows:

- Incorporate the concepts from Chapter 15 of the HCM 2000 into Chapter 20 to accommodate two-lane highways in developed areas. The concepts from Chapter 16, “Signalized Intersections” and Chapter 17, “Un-Signalized Intersections” of HCM 2000 may also be utilized in this incorporation for analyzing situations with intersections.
- Develop a speed prediction model that is capable of estimating urban arterial *ATS* more precisely in order to analyze urban streets and two-lane highways in developed areas.
- Create a new chapter that contains an assessing methodology for two-lane highways and multilane highways in developed areas.

2.2.1.3 New approaches to estimating percent time-spent-following

Regarding the percent time-spent-following overestimation problem in the HCM 2000 methodologies, Durbin (2006) proposed two new approaches to estimating the percent time-spent-following (*PTSF*). It is suggested that only vehicles that are impeded by slower leading vehicles and not able to travel at their own desired speeds or pass

slower vehicles contribute to *PTSF*. Based on this idea, the author provided two methods for estimating *PTSF*.

Weighted-average approach. Durbin indicated that the proportion of passenger cars traveling involuntarily in platoons is virtually identical in meaning to *PTSF*. For estimating this proportion, two assumptions were made. First, it was assumed that heavy vehicles never follow passenger cars, but the converse is not true. Second, it was assumed that passenger cars traveling involuntarily in platoons travel at the average speed equal to about the average speed of the leader. Thus, the formula was developed as follows:

$$P_{pc2} = \frac{S_{atot} - S_{dpc} \times P_{pc} - S_{ahv} \times P_{hv}}{S_{ahv} - S_{dpc}} \tag{2-9}$$

where

P_{pc2} = proportion of passenger cars involuntarily traveling in platoons

S_{atot} = average actual travel speed of all vehicles

S_{dpc} = average desired travel speed of passenger cars

P_{pc} = proportion of passenger cars in the traffic mix

S_{ahv} = average actual travel speed of heavy vehicles

P_{hv} = proportion of heavy vehicles in the traffic mix

Probabilistic approach. This approach attempted to estimate *PTSF* by using probabilities. To implement this approach, slow-moving vehicles and fast-moving vehicles were defined for the vehicle classification. Any vehicle, regardless of its size and performance, can be placed in one of the two categories. Slow-moving vehicles usually lead or potentially lead platoons, while fast-moving vehicles are usually impeded by slow-moving vehicles and join platoons for lack of passing opportunities. The

relationship between the two vehicle categories was then used to estimate the value of *PTSF*. Two probabilities were included in this estimation, one of which represents the probability that a vehicle travels in a platoon and the other represents the probability that a platooned vehicle involuntarily travels at a speed lower than the desired speed. Therefore, *PTSF* can be estimated by

$$PTSF = P_p \times P_t \tag{2-10}$$

where

P_p = probability of a vehicle being part of a vehicular platoon

P_t = probability of a vehicle traveling involuntarily in a platoon at a speed lower than the desired speed

The details of the way to measure these two probabilities were also discussed in the paper. The probability regarding vehicles traveling in a platoon was suggested being measured by using vehicle time headway data. Vehicles with time headways less than a pre-defined threshold value are considered inside platoons, either involuntarily or voluntarily. The procedure for estimating the probability that a vehicle involuntarily travels in a platoon in the field was developed based upon the assumptions that the distribution of desired speeds is the same for both vehicles in platoons and vehicles in free-flow conditions, which is usually a normal distribution. This procedure first measures the average travel speed of slow vehicles, and then the distribution of desired speeds for all vehicles can be established, based on which vehicles with speeds higher than the average speed of slow-moving vehicles can be identified.

The author stated that both methods could provide reasonable *PTSF* values, which were lower than the ones with the HCM 2000 procedures. However, although it is

rational to distinguish involuntary following and voluntary following when estimating *PTSF* by the probabilistic approach, the definitions of these two following types need to be further investigated. Because from driver's aspect, traveling in a platoon at the speed less than its desired speed but greater than or equal to its tolerable speed may not be considered involuntary. Thus, the distribution of tolerable speed needs to be investigated as well as the distribution of desired speed.

2.2.2 Percent Delay Based Methodology

Yu and Washburn (2009) developed a facility-based evaluation methodology for two-lane highways. This methodology allows the various features (e.g., isolated intersections, passing lanes) that are typical to an extended length of two-lane highway to be analyzed as a single facility. Yu and Washburn's method divides the facility into appropriate segments, and properly accounts for the operational effects of traffic flow transitioning from one segment to another. A two-lane highway with an isolated signalized intersection was used as a model in this study. For a two-lane highway facility with signalized intersections, the entire length of the facility is divided into three types of segments: the basic two-lane highway, the signal influence area, and the affected downstream segment. Percent delay (*PD*) was selected as the common service measure for the interrupted-flow facility of a two-lane highway with signalized intersections. According to the paper, percent delay is calculated by dividing delay by free-flow travel time. An excess of average travel time over free-flow travel time for drivers on a facility is defined as delay. *PD* is calculated by

$$PD = \frac{\sum_{H,S}(D_H + D_S)}{\sum_{H,S}\left(\frac{L_H}{FFS_H} + \frac{L_S}{FFS_S}\right)} \times 100 \qquad (2\text{-}11)$$

where

PD = average percent delay per vehicle for the entire facility (%)

D_H = average delay time per vehicle for the two-lane highway segment (s/veh)

D_S = average 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_H = length of the two-lane highway segment (ft)

L_S = length of the signalized intersection influence area (ft)

Percent delay mainly represents a driver's perception of freedom during driving. A driver's freedom may be affected by the presence of signal controls, restrictive road conditions (e.g., no-passing zones), or opposing traffic, so these effects can also be reflected by percent delay. The LOS criteria for two-lane highway facilities, based on percent delay, are shown in Table 2-2.

The authors used a simulation-based approach to develop the computational methodology, due to the difficulties of measuring the field data needed. A hybrid simulation approach combining CORSIM and TWOPAS was applied. The CORSIM simulation program was used to model a two-lane roadway segment that contains a signalized intersection. The downstream two-lane highway segment relative to the signalized intersection was then simulated by TWOPAS. The regression models for estimating the length of the effective upstream/downstream segment were developed

based on the simulation results. Given the regression models, the process of determining the LOS of a two-lane highway facility could be outlined as follows:

- **Step 1.** Compute the lengths of the effective upstream and downstream segments, and then divide the facility into segments accordingly.
- **Step 2.** Determine the free-flow speed of the facility.
- **Step 3.** Compute the average travel speed on the basic two-lane highway segments.
- **Step 4.** Compute control delay at the signalized or unsignalized intersection influence area.
- **Step 5.** Compute the average travel speed on the affected downstream segment.
- **Step 6.** Determine the delay for each segment by calculating the difference between average and free-flow travel time, except for the intersection influence area, using the control delay as the segment delay.
- **Step 7.** Determine the percent delay by dividing the total delay by the total free-flow travel time along the whole facility, leading to the determination of the facility-level LOS.

However, Yu and Washburn's methodology was built upon the results of the hybrid simulation approach, therefore, it is still necessary to use field data or an integrated simulation model to verify and validate this methodology.

2.2.3 South Africa Methodology

The South African National Roads Agency Limited (SANRAL) (2006) developed an analytical methodology for determining the level of service of two-lane highways in rural areas. A macroscopic simulation model was developed and extensively calibrated by the field observations in this study. Although called a macroscopic simulation model, the simulation model actually used the microscopic method to simulate free-flow conditions and the macroscopic method to simulate queue formation/platooning, respectively. The simulated platooning can be further used to estimate traffic

performance measures, such as average travel speed, density, percentage followers, and follower density.

The researchers also investigated several alternative performance measures other than average travel speed and percent time-spent-following, which were employed in the HCM 2000 two-lane highway methodology. They identified follower density as the most suitable measure, for it contains the combined meanings of percentage followers, traffic flow, and average travel speed. None of these three measures can fully reflect a driver's perception of LOS when traveling on a two-lane highway. To provide criteria for determining LOS based on follower density, the field observations were analyzed in two aspects. The researchers first developed a set of threshold values for determining the follower-density-based LOS, and then estimated the minimum acceptable LOS below which the capacity expansion of a facility needs to be considered. The details of the criteria are presented in Tables 2-3 and 2-4.

It should be noted that the calibration of the simulation model and the LOS criteria were based on the field data collected in South Africa. Therefore, the characteristics of driver behaviors there, such as passing on wide shoulders or in no-passing zones, should be attended to when applying the methodology developed in this study to other countries.

2.2.4 Threshold Speed Based LOS Criteria

Romana and and Pérez (2006) pointed out that for two-lane highway travelers, their desire to pass and frustration over being delayed should not be determined only by looking at headways of the difference between actual speed and desired speed. After reviewing the existing measures of effectiveness and other possible measures, the authors proposed a measure named threshold speed to better determine the LOS of a

two-lane highway facility. As stated in the paper, threshold speed is defined as the minimum acceptable speed for drivers that are traveling under high flow rate condition on a road with the consistent characteristics. They proposed that threshold speed should be applied along with existing performance measures (i.e., average travel speed and percent time-spent-following) to determine the LOS of two-lane highways in the HCM 2000. The new LOS assessing approach developed in the paper is summarized as follows:

- If the average travel speed on a two-lane highway is higher than the threshold value, the percent time-spent-following will be the only necessary measure to check to ascertain the level of service.
- If the average travel speed is lower than the threshold value, percentage time-spent-following becomes less important than speed when determining the LOS of two-lane highway facility.

With this new approach, it is expected that various features of different highways can be considered, rather than classifying highways into Class I and II in the first place. At the same time, a driver's perception of the level of service can also be reflected by this approach.

However, the authors did not provide details of how to determine the threshold speed, and instead only suggested several ways to select threshold speed. A field study using the new approach mentioned in the paper used 80 km/h as the threshold speed, which approximated the users' expectations obtained from surveys in Spain. The example LOS criteria with the threshold value of 80 km/h compared with the HCM 2000 criteria in the paper are presented in Table 2-5.

2.2.5 Queuing Relationship Based Methodology

Two problems concerning the current HCM (TRB, 2010) two-lane highway procedures were pointed out by Polus and Cohen (2009) based on their field

observations in Israel. One is that LOS is insensitive to average travel speed, and the other is overestimation of percent time-spent-following. At this point, the authors developed queuing models for estimating the LOS of two-lane highways on the basis of Queuing Theories. Next, they estimated the parameters of the queuing models using field data collected in northern Israel. Five major parameters were included in this study: flow, average platoon length, traffic intensity, percent time-spent-following, and freedom of flow. As the authors stated, all of the parameters can be used alone or in combination with others to determine LOS. Some primary findings are summarized as follows:

Flow. A linear relationship was found between one-way flow and speed according to the field data. Although only one-way flow was involved, they found that the speed data had included the influence of the opposing flow.

Average platoon length. The average number of headways per platoon obtained from the field data were used to calculate the average platoon length by simply adding one to account for the lead vehicle.

$$APL = Q_0 + 1 \tag{2-12}$$

where

APL = average platoon length (vehicles)

Q_0 = average number of headways inside platoons (dimensionless)

The average number of headways inside platoons Q_0 was found to be related to one-way flow as follows:

$$Q_0 = \begin{cases} 1 + 0.00343V - 1.255 \times 10^{-6} V^2 & \text{for } V \le 700 \text{ veh/h} \\ 1.6152 + 0.00168V & \text{for } V > 700 \text{ veh/h} \end{cases} \tag{2-13}$$

where V is the one-way volume.

The field data shows a positive relationship between Q_0 and one-way flow, and Q_0 is significantly sensitive to flow. Thus, *APL* was believed by the authors to be the best parameter for measuring LOS.

Traffic intensity. The M/M/1 queuing model was used to describe the formation of platoons on two-lane highways. The ratio between the average wait time for the first vehicle behind a lead vehicle to pass, and the average inter-arrival time vehicles join the back of the platoon, which was defined as traffic intensity ρ, can be derived as

$$\rho = 1 - \frac{1}{Q_0} \tag{2-14}$$

The relationship between ρ and one-way flow V was also estimated by the field data (ρ was measured by Equation 2-15) as follows:

$$\rho = 0.68\left(1 - \exp(-0.004V)\right) \tag{2-15}$$

The field data show that traffic intensity levels off after the one-way volume reaches 600 veh/h.

Percent time-spent-following (*PTSF*). The equation for calculating *PTSF* was derived on the basis of Queuing Theories, as follows:

$$PTSF = 100 \times \frac{(Q_0 - 1)}{(Q_0 + N_0 - 1)} \tag{2-16}$$

where

$PTSF$ = percent time-spent-following (%)

N_0 = average number of headways between platoons (dimensionless)

Again, the relationship between *PTSF* and two-way flow was derived based on the field data, thus

$$PTSF = 1 - \exp\left(-0.000504V_p\right) \tag{2-17}$$

where V_p is the two-way flow in passenger cars equivalent.

Freedom of Flow. This parameter represents the freedom a driver perceives when traveling in the traffic stream, which thereby becomes the LOS evaluation measure. It can be calculated by

$$\eta = \frac{N_0}{\rho} \tag{2-18}$$

where η is the freedom of flow.

A regression model expressing the relationship between η and two-way flow V_p was shown as

$$\eta = \frac{4950}{V_p} \tag{2-19}$$

As the authors concluded, although all five parameters can be used to estimate LOS of two-lane highways, as they are highly related to the flow characteristics, *PTSF* was considered the most suitable measure because it is capable of evaluating the cost of delay in economic terms. It was also suggested that freedom of flow η should be an additional measure for estimating LOS, as it might contain more information on driver's perception. Based on the proposed flow measures, the new thresholds for LOS on two-lane highways were developed (Table 2-6).

The *PTSF* values calculated by equations in this paper were much lower than the HCM *PTSF* values, which is consistent with the findings in other studies. However, since the models developed in this paper were based on the field data in Israel, the local characteristics should be taken into account when applying the methodology outside the country.

2.2.6 Empirical Study Based Performance Measures

2.2.6.1 The Montana studies

It is well known that *PTSF* is difficult to measure in the field; therefore, some researchers tried to find alternative performance measures that are easier to collect on two-lane highways. Al-Kaisy and Sarah Karjala (2008) investigated six selected performance indicators: average travel speed, average travel speed of passenger cars, average travel speed as a percent of free-flow speed, average travel speed of passenger cars as a percent of free-flow speed of passenger cars, percent followers, and follower density. This empirical study was based on field data collected in the state of Montana at four different study sites. The relationships between performance measures and primary platooning variables such as traffic flow, heavy vehicle percentage, and no-passing zone percentage were examined. As a result, follower density was identified as the best measure for describing the level of service of two-lane highway operations, as it can reflect the traffic level that was indicated as a dominant factor that determines the performance of most highways. The authors stated that percentage of followers has no strong relationship with traffic level, as it depends mainly on time headway distribution. Ways to measure follower density in the field were also provided in the paper. However, the threshold values of follower density for determining the LOS of two-lane highway operations were not discussed in the paper.

In 2010, Al-Kaisy and Freedman did another empirical investigation into a new measure, Percent Impeded (*PI*), which was also intended to evaluate performance of traffic operations on two-lane highways. *PI* was defined in the paper as a point measure indicating the percentage of vehicles that follow slower vehicles due to lack of passing opportunities on two-lane highways. The methodology for deriving *PI* is similar to the

probabilistic approach for estimating *PTSF* in Durbin's (2006) thesis. In order to capture driver's perception of impedance experienced during driving, locations in the immediate upstream and downstream area of a passing lane where platoons break up and form were suggested as ideal for collecting data. Besides *PI*, the performance measures of percent followers, follower density, and ratio of average travel speed to free-flow speed were also examined in this study as well for reference.

PI was eventually validated in this study. The empirical analysis showed that *PI* is more sensitive to platooning than other performance measures. In comparison, *PI* contains more information, due to relatively high correlations with other measures and platooning variables, except for traffic flow. Furthermore, *PI* is the only measure that shows the variation trend, corresponding to the one of general performance measures for upstream and downstream of the passing lane, presented in the HCM 2000. Four multivariate regression models were developed to estimate the four performance measures using the data from one of the two study sites. The one for *PI* is given as

$$PI = 11.4 + 0.0012X_1 + 0.0015X_2 + 0.016X_3 + 0.879X_4 - 0.794X_5 + 0.081X_6 \qquad (2\text{-}20)$$

where

X_1 = volume in the same direction of travel

X_2 = volume in the opposing direction of travel

X_3 = percent no-passing

X_4 = distance downstream of the passing lane

X_5 = presence of merge effect (limited to the first station downstream of passing lane)

X_6 = percentage of trucks

Variables X_1 and X_2 were found to be insignificant at the 95% confidence level.

The LOS criteria based on *PI* were not discussed in the paper, and as the authors expected, further investigation into the performance measure *PI* for a complete analytical methodology is needed.

2.2.6.2 The Oregon study

The Facility Analysis and Simulation Team at the Oregon Department of Transportation (2010) established several models for predicting alternative two-lane highway performance measures, due to the difficulty in collecting field data of *PTSF*. Several performance indicators and platooning variables that could describe two-lane highway operations were selected. After that, field data were collected from 13 sites in Oregon State for model development, and another 4 sites in Oregon for model validation. Based on the field data, the predicting models were formulated and finally validated.

The performance indicators included in this study were average travel speed, average travel speed of passenger cars, average travel speed as a percent of free-flow speed, average travel speed of passenger cars as a percent of free-flow speed of passenger cars, percent followers, and follower density. All the indicators were directly or indirectly obtained from the field data. It should be noted that follower density in this study was measured by

$$\text{Follower density} = \frac{\text{Number of followers}}{\text{Average follower travel speed}} \tag{2-21}$$

To establish the models, platooning variables were collected as well and served as the independent variables in the models, including traffic flow, percent heavy vehicles, standard deviation of free flow speeds, percent no-passing zones, and terrain.

As a result of statistical analysis, follower density was finally selected as the best performance indicator. Further, the researchers found that follower density has a wide spectrum, which makes the level of service categories easy to determine. The final model is given in Table 2-7.

This promising estimate model is of great importance, as only a few studies have been done on follower density, especially on the basis of field data. The conclusions of this study may be used as references when developing two-lane highway simulation models and analytical methodologies.

2.2.7 Driver Comfort Service Measures

The HCM 2000 uses density as the key indicator to determining the level of service of traffic operations on both rural and urban freeways. However, Kim, Courage et al. (2003) pointed out that although density is ideal for assessing urban freeways which usually need to accommodate high demands, it is uncertain whether density is appropriate for assessing rural highways, where “driver comfort” is more important to drivers. Therefore, the researchers proposed three measures which are intuitively related to driver comfort: acceleration noise, number and duration of cruise control applications, and percent time-spent-following. In their initial study, CORSIM was used to generate the required data instead of collecting them in the field. The step-by-step data were processed to derive the proposed measures, as summarized below:

- **Acceleration noise.** This is calculated as the standard deviation of the acceleration for each vehicle produced by CORSIM.
- **Number and duration of cruise control applications.** A cruise control emulator was developed and incorporated into the previous CORSIM logic. Next, the proportion of time the cruise control was used and the number of cruise control applications was recorded and reported.

- **Percent time-spent-following.** This following scheme, developed by Halati et al. (1997), was used to determine when a vehicle is identified as a follower. The percent time-spent-following was then determined.

The relationships between traffic volume and the proposed measures were determined by plotting the processed data. According to these relationships, the authors concluded that acceleration noise can be directly used to establish LOS criteria for rural highways. The other two measures considered were found to require further investigations.

Although the proposed measures are promising, field data are needed to confirm the conclusions in the paper, as all the findings were based on the simulation data. Also, because the measures in the paper apply to all rural highways rather than only two-lane rural highways, it is necessary to consider the characteristics of two-lane highways when utilizing Kim et al.'s work in two-lane highway analysis.

2.3 Traffic Models of Two-Lane Highway Operations

Given a logical framework for operations on two-lane highways, it is very important that the appropriate traffic models are added to each step to make the framework functional. This section reviews the previous work on passing sight distance model(s), desire to pass model(s), and gap acceptance model(s). A study on desired speed distribution is introduced at the end of this section.

2.3.1 Passing Sight Distance Model

According to NCHRP Report 605 (Harwood, et al. 2008), the current passing sight distance (PSD) criteria for two-lane highways in A Policy on Geometric Design of Highways and Streets (AASHTO, 2004) is for highway design, while the criteria for marking passing or no-passing zones on two-lane highways are included in the Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) (FHWA, 2003).

AASHTO defines the minimum PSD required as the sum of four distances traveled by a passing vehicle: 1) distance traveled during perception and reaction time, and during initial acceleration before passing vehicle moves into the left lane; 2) distance traveled in the left lane during passing; 3) distance between passing vehicle and opposing vehicle when passing is completed; and 4) distance traveled by an opposing vehicle for two-thirds of the time the passing vehicle travels in the left lane. The criteria are essentially the same as those in the 1954 AASHTO policy. However, the PSD value at different design speed levels proposed by AASHTO based on this definition is substantially larger than the one derived by the MUTCD models. This was explained in the report as that AASHTO defines passing sight distance in a too conservative way, which commits a passing driver to complete the pass; while the MUTCD criteria are appropriate for a passing driver wishing to abort the pass.

Glennon (1988) derived a mathematical model for estimating the critical position for a passing driver during the passing maneuver to complete or abort the pass, when he found that the AASHTO (1984) PSD model did not consider the possibility of an aborted maneuver and then derived long distances. Glennon's model was developed based on several assumptions as follows:

- The opposing vehicle travels at the design speed;
- The passing vehicle accelerates to the design speed at or before the critical position;
- The impeding vehicle travels at a constant speed design speed; and
- The minimum acceptable headway is one second for both clearance between passing and impeding vehicles and the clearance between passing and opposing vehicles at the end of the pass.

As a result, the critical position model and the critical passing sight distance model were obtained as follows:

$$\Delta_c = L_p + m\left[\frac{(2m + L_I + L_P)}{2v - m} - \sqrt{\frac{4v(2m + L_I + L_P)}{d(2v - m)}}\right] \tag{2-22}$$

$$S_C = 2v + \frac{2v(L_P + m - \Delta_c)}{m} \tag{2-23}$$

where

Δ_C = critical separation between passing and impeding vehicles (ft)

L_I = length of impeding vehicle (ft)

L_P = length of passing vehicle (ft)

m = difference in speed between passing and impeding vehicles (ft/s)

v = design speed, speed of passing and opposing vehicles (ft/s)

d = deceleration rate of passing vehicle abortion (ft/s)

S_C = critical sight distance (ft)

To apply these models to the design and the marking of a passing zone, the author made some additional assumptions. In this case, the PSD requirements obtained become considerably less than the AASHTO PSD requirements, but very close to the MUTCD requirements (even though the MUTCD requirements were derived with a completely different set of models and criteria). By doing another analysis of the sensitivity of PSD values to vehicle length, the author concluded that the impact of truck length is not as significant as reported in previous studies.

Hassan et al. (1996) also developed a revised model for determining the minimum required passing sight distance, and it is similar to the Glennon's model. It is based on the concept of critical sight distance and developed by means of kinematic equations

describing the interactions between the vehicles involved in a passing maneuver. Three main modifications are identified in the paper:

- Hassan's model uses a more practical approach than the Glennon's model by assuming that a passing driver whose front bumper is abreast with the front bumper of the passed vehicle will complete the pass even if the critical position has not yet been reached. Thus, two models are derived, for both $\Delta_C>0$ and $\Delta_C\leq 0$.
- Hassan's model uses a more conservative assumption that the passing driver will experience an additional perception-reaction time when deciding whether to abort the passing maneuver or not.
- The clearance distance between the passing vehicle and opposing vehicle at the end of an aborted pass is different from that in a completed pass. The authors pointed out that it should be related to the speeds of the passing and opposing vehicle at that moment. And the clearance distance between the passing and passed vehicle, either at the end of an aborted pass or a completed pass, is related to the speed of the trailing vehicle, rather than a constant speed differential.

The revised model was finally validated using a set of field data, and the results showed that the PSD requirements are longer when a passing vehicle is trying to pass a truck for speeds up to 110 km/h. Therefore, it is suggested that the MUTCD standards consider the effect of vehicle length to ensure safe passes.

The current PSD requirements for design and marking vary for different design speeds. However, many random variables are ignored in the calculation process. Facing this fact, Khoury and Hobeika (2007) derived a PSD distribution that considers the contributing random variables by devising a Monte-Carlo simulation model and a closed form analytical estimation model. The base model for this development is Glennon's PSD model, because it provides the best description of the mechanics of a passing maneuver. The distribution of each contributing parameter was assigned according to the literature and the field data sources, as shown in Table 2-8.

The authors used a software package named Cristal Ball to perform the random sampling from the corresponding probability density functions for each parameter mentioned in Table 2-8. The best-fit distribution of the resulting PSD obtained is a Gamma distribution. The main descriptive statistics of the critical point (the position beyond which a passing vehicle is committed to complete the pass) and the PSD values for a design speed of 80 km/h (50 mi/h) are presented in Table 2-9.

Then the PSD distribution results were verified by a parallel solution method, which was used to obtain the first and second moments of the PSD distribution. The differences of means and standard deviations between the Monte-Carlo simulation and the analytical model are less than 2%. For the conclusions, the authors indicated that the MUTCD and Glennon's design values are close to the mean of the PSD distribution in this paper, but the AASHTO PSD values are overestimated. As the authors expected, the PSD distribution obtained in this study can be used to develop a unique microscopic simulation, which makes it feasible to quantify the trade-off between performance and safety finally.

2.3.2 Desire to Pass Model and Gap Acceptance Model

A passing maneuver is always motivated by a driver's desire to pass. Therefore, a reasonable model for estimating driver's desire to pass has a key role in a simulation. However, only a few studies have been done on this topic, for it is difficult to capture someone's desire to pass in the real world. The latest study done by Farah and Toledo (2010) provides a model that captures both drivers' desire to pass and their passing gap acceptance decisions to complete a desired pass. It should be noted that the passing gap in this paper is defined as the time gap between the opposing vehicle and the subject vehicle when the subject vehicle is in the following mode. This term is

essentially the same as passing sight distance but by time instead of distance. The development of the model is based on the data collected with a driving simulator regarding the subject driver's decision whether or not to pass the impeding vehicle. The model contains two levels of choices. The first level choice is the decision whether or not to consider passing according to the desire to pass, and the second level choice is the decision whether or not to accept an available passing gap. The desire to pass is formulated as a binary choice problem:

$$DP_{nt} = \begin{cases} 1 & \text{if } U_{nt}^{DP} \geq 0 \\ 0 & \text{if } U_{nt}^{DP} < 0 \end{cases} \tag{2-24}$$

where

DP_{nt} = the choice indicator variable with value 1 if the driver desires to pass and zero otherwise

U_{nt}^{DP} = the utility to the driver from desiring to pass (a function of explanatory variables speed differential, following distance and cumulative distance)

And if a driver desires to pass, then he/she will evaluate the available passing gaps against his/her minimum acceptable gap. The driver passes the impeding vehicle only if the available passing gap is acceptable.

$$A_{nt} = \begin{cases} 1 & \text{if } G_{nt} \geq G_{nt}^{cr} \\ 0 & \text{if } G_{nt} < G_{nt}^{cr} \end{cases} \tag{2-25}$$

where

A_{nt} = choice indicator variable with value 1 if the gap is accepted and zero otherwise

G_{nt} = the available passing gap

G_{nt}^{cr} = the critical passing gap (a function of explanatory variables current speed, lead speed, opposing vehicle speed, road curvature, type of front vehicle and age)

Then the probability to complete a passing maneuver is determined by the product of the probabilities of a positive decision on both choices.

The results show that all the variables in the model are statistically significant at the 95% confidence level. Although the results reported are promising, the authors indicated that the collection of driving simulator data may result in biases in behavior. In some cases, simulator drivers may modify their behavior if they feel indifferent or tired in the course of the experiment. And the passing behavior model proposed in this paper is not able to handle the situations such as a passing driver aborts the pass or passes multiple vehicles at a time.

2.3.3 Desired Speed

User dissatisfaction is usually due to the situation that a driver cannot travel at his/her desired speed when following a slower vehicle. Therefore, Catbagan and Nakamura (2008) believed that a “follower” can be defined more logically as a vehicle traveling below the desired speed because it is impeded by the slower lead vehicle. Then they found it necessary to develop desired speed distributions to calculate a vehicle’s following probability given its current speed, or identify followers. Thus, it

would be possible to better replicate following behavior on a passing-restricted two-lane highway in a simulation. The study took the data collected from an ordinary two-lane highway (with passing restrictions) in Japan as the basis. The data were gathered for six consecutive months. A video survey was also conducted at the study segment to verify the detected vehicle lengths based on vehicle classifications. Two types of vehicles were identified for the study purpose, which are passenger cars (PC, 0 to 4.9 m) and heavy vehicles (HV, 4.9 m and longer).

The authors then used the modified Kaplan–Meier Approach (the original method from Hoogendoorn, 2005) to estimate free speed distributions by using the field data. The primary findings are summarized as follows:

- Holiday travelers usually prefer higher desired speeds than weekday travelers. The desired speeds of passenger-car drivers are not sensitive to period of time during a day. And heavy-vehicle drivers generally prefer higher speeds during daytime. Figure 2-1 presents the analysis results.
- Truck driver's behavior is greatly affected when following another truck on a rainy day.
- Gradient has some effect on the free speed distributions. The author mentioned that drivers may travel at speeds lower than the actual desired speeds because of the slope constraints.

Although the estimated desired speed distributions for various conditions were proposed in this paper, no specific distribution function was presented for estimating parameters and adjustment factors in different situations.

Table 2-1. HCM LOS criteria for two-lane highways (TRB, 2010)

LOS	Class I Percent Time-Spent-Following	Class I Average Travel Speed (mi/h)	Class II Percent Time-Spent-Following	Class III Percent of Free-Flow Speed
A	≤ 35	> 55	≤ 40	> 91.7
B	> 35–50	> 50–55	> 40–55	> 83.3–91.7
C	> 50–65	> 45–50	> 55–70	> 75.0–83.3
D	> 65–80	> 40–45	> 70–85	> 66.7–75.0
E	> 80	≤ 40	> 85	≤ 66.7

Table 2-2. LOS criteria for two-lane highway analysis

Level of service	Percent Delay (%)
A	≤ 7.5
B	> 7.5 – 15
C	> 15 – 25
D	> 25 – 35
E	> 35 – 45
F	> 45

Table 2-3. LOS thresholds of follower density

LOS	"Typical" follower density	Range of follower densities
A	1.0	0.3–1.4
B	2.0	1.3–3.3
C	4.0	3.0–6.7
D	8.0	6.3–9.5

Table 2-4. Minimum acceptable level of service for two-lane highways

Two-lane highway class	Acceptable level of service	
	Normal day peak hour	30th Highest hour
Class I Highways	LOS C	LOS D
Class II Highways	LOS D	LOS D

Table 2-5. Comparison of LOS boundaries in the HCM 2000 and the proposed approach

LOS	HCM 2000 (Class I, speed in km/h)	HCM 2000 (*PTSF* in %)	Speeds Greater than Threshold, 80 km/h	Speeds Lower than Threshold, 80 km/h
A	> 90	$PTSF \leq 35$	$\%DV \leq 30$	-
B	$80 < s \leq 90$	$35 < PTSF \leq 50$	$30 < \%DV \leq 55$	-
C	$70 < s \leq 80$	$50 < PTSF \leq 65$	$55 < \%DV \leq 75$	-
D	$60 < s \leq 70$	$80 < PTSF \leq 90$	$75 < \%DV$	$60 < s \leq 80$
E	$40 < s \leq 60$	$80 < PTSF$	-	$40 < s \leq 60$
F	-	-	-	$s \leq 40$

%*DV* = percentage of delayed vehicles

Table 2-6. Proposed new thresholds for level of service on two-lane highways

Level-of-service	*PTSF* thresholds (%)	Two-way flow thresholds(pc/h)	Freedom-of-flow thresholds (Eta)
A	0–15	0–300	≥16.5
B	15–30	300–700	7.1–16.5
C	30–45	700–1,200	4.1–7.1
D	45–60	1,200–1,800	2.8–4.1
E	60–75	1,800–2,700	1.8–2.8
F	75–100	≥2,700	≤1.8

Table 2-7. Fitted model for follower density

Variable	Coefficient
Dependent variable	
Follower density	
Independent variable	
Intercept	-0.4823332
Traffic volume	0.0067640
Opposing volume	0.0006175
% Heavy vehicles	0.0008791
% No Passing	0.0008097
Terrain (1 for Level, 2 for Rolling)	0.2482458

Table 2-8. Probability distributions of contributing parameters

Parameter	Name	Distribution type	Source
V	Speed	Truncated normal	Field data
M	Speed differential	Log-normal	AASHTO, MUTCD, Polus, Glennon
D	Deceleration rate	Truncated normal	Fambro
L_P,L_I	Vehicle lengths	Discrete	US Census Bureau (vehicle invt.)
R	Perception reaction	Log-normal	McGee, Neuman, Lerner
G_C	Completed pass gap	Uniform	AASHTO, MUTCD, Polus, Glennon
G_A	Aborted pass gap	Uniform	AASHTO, MUTCD, Polus, Glennon

Table 2-9. Main descriptive statistics of the PSD design values

Variable	Critical point	PSD	
Confidence interval (%)	95	95	99
Mean (m)	3.75	213.65	215.32
Median (m)	3.57	209.25	210.23
Standard deviation (m)	2.08	9.99	9.45

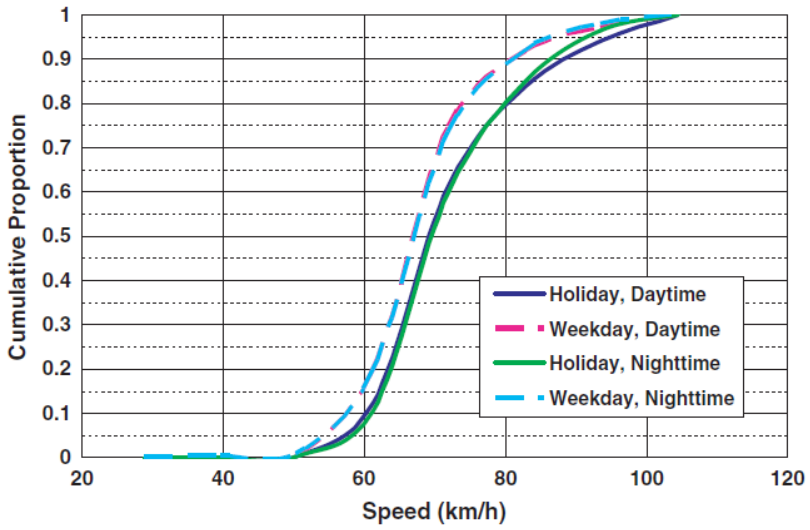


Figure 2-1. Desired speed distribution during holidays and weekdays (PC_PC, southbound, no rain). Source: Catbagan, Nakamura. Desired Speed Distributions on Two-Lane Highways Under Various Conditions, Transportation Research Record, Washington D.C., 2004

CHAPTER 3
DEVELOMENT OF TWO-LANE HIGHWAY SIMULATION TOOL

This chapter describes the development approach for the new two-lane highway simulation tool, including the logic and models employed in CORSIM to facilitate two-lane highway modeling. All of the following tasks were accomplished.

- Develop traffic flow modeling routines for two-lane highway operations.
- Develop a mechanism for integrating signalized intersections into a two-lane highway facility.
- Modify the input and output mechanisms of CORSIM as necessary for the two-lane highway-modeling capability.

3.1 Develop Traffic Flow Modeling Routines for Two-Lane Highway Operations

The logic of the various components involved in modeling the two-lane highway passing maneuvers (either oncoming lane or passing lane) are described in the following subsections.

3.1.1 Car Following

The car-following model currently employed in CORSIM is the Pitt car following model (Halati et al., 1997). This car-following model was also retained for use in the two-lane highway modeling. This model incorporates the distance headway and speed differential between the lead and follower vehicle as two independent variables, and the basic assumption is that the follower vehicle will try to maintain a safe space headway.

3.1.2 Passing in an Oncoming Lane

The most distinguishing feature of traffic operations on two-lane highways is passing in the oncoming lane (when passing lanes are not present). Therefore, this passing maneuver is constrained by not only the amount of opposing-lane distance used in the execution of a passing maneuver, but also the sight distance and clear-

distance (or gap size) a follower requires before attempting a passing maneuver. The former issue depends on road design and markings of no-passing zone, while the latter issue depends on traffic demands. The following subsections describe the various components of logic employed to determine when and how a vehicle will perform a passing maneuver in the oncoming lane.

3.1.2.1 When will a vehicle attempt to pass a vehicle in front of it?

Step 1. Determine if the subject vehicle is in a following mode. Currently, the program defines a vehicle as being in a following mode when the time headway between it and the vehicle immediately in front of it is equal to or less than 3 seconds – this is currently the logic the HCM 2010 uses to approximate *PTSF*[1]. The value of 3 seconds, however, can be changed by the analyst in the input file. Additionally, the trailing vehicle must be traveling at a speed at least equal to the speed of the leading vehicle. If the subject vehicle is determined to be in a following mode, then the following steps are carried out to determine if the following vehicle will attempt a passing maneuver.

Step 2. Determine tolerable speed. If it is determined that a vehicle is in a following mode, then the tolerable speed for that vehicle is calculated. Tolerable speed is defined as the maximum speed at which the desire to pass for a following driver will be 100%. Tolerable speed varies for different driver types (Equation 3-1 for the computation), due to the different degree of aggressiveness for each driver type. Note that a driver's desired speed is a function of free-flow speed and driver type. For

[1]It is anticipated that alternative approaches to the determination of whether a vehicle is in a following mode will eventually be considered for incorporation into the program, such as the probabilistic approach by Hideki, et al. (2008).

example, a driver of type 1 will have a desired speed of 88% of the link free-flow speed, while a driver of type 10 will have a desired speed of 112% of the link free-flow speed (these percentages of free-flow speed can be modified by users on RT (Record Type) 147 of the TRF input file in CORSIM).

$$TolerableSpeed_i = DesiredSpeed_i \times \frac{80+i}{100} \tag{3-1}$$

where

$TolerableSpeed_i$ = tolerable speed for driver type *i* (mi/h)

$DesiredSpeed_i$ = desired speed for driver type *i* (mi/h)

Step 3. Determine desire to pass. The main factor influencing a driver's desire to pass is the difference between their actual travel speed and their desired speed. The degree of the following driver's desire to pass is quantified as a number between 0 and 1 based on a non-linear function of the current speed of the subject vehicle. The initial desire to pass (*DTP*) value is given by

$$DTP = \begin{cases} 1, \text{if } CurrentSpeed < TolerableSpeed \\ 0, \text{if } CurrentSpeed = DesiredSpeed \\ \left(\dfrac{DesiredSpeed - CurrentSpeed}{DesiredSpeed - TolerableSpeed}\right)^4, \text{otherwise} \end{cases} \tag{3-2}$$

The *DTP* is illustrated graphically in Figure 3-1 for one example driver. The *DTP* value is firstly adjusted by an impatience factor. For vehicles that have a positive *DTP*, but are in a following mode and have not yet initiated a passing maneuver, the impatience factor will incrementally increase the *DTP* value with each simulation time step; thus increasing the probability of a passing maneuver being initiated. The impatience factor is also a function of the driver type, with larger increments being

associated with more aggressive drivers. The *TSWTP* counter will initiate when a vehicle has a *DTP* value greater than zero and will reset when the vehicle completes a pass or its *DTP* value goes to zero. The impatience value can be adjusted on record type 155. The impatience factor value is given by

$$ImpatienceFactor = TSWTP \times ImpatienceValue \times \sqrt{DT} \tag{3-3}$$

where

ImpatienceFactor = the increment to the desire to pass value for each second of time spent wanting to pass

TSWTP = time spent wanting to pass (s)

DT = driver type (defined by CORSIM)

ImpatienceValue = degree of impatience in waiting to pass (default = 0.001)

Then the *DTP* is adjusted based on the length of the trailing vehicle and the length of the leading vehicle. A longer leading vehicle, such as a large truck, will increase the *DTP* value for the trailing vehicle. Conversely, the *DTP* value will be decreased for a longer trailing vehicle. The final adjustment to the *DTP* value (*AdjDTP*) is given by

$$AdjDTP = (DTP + ImpatienceFactor) \times \left[1 - \left(\frac{1}{14} - \frac{1}{FolLeng}\right)\right] \times \ln\left[\exp(1) - \left(\frac{1}{14} - \frac{1}{LeadLeng}\right)\right] \times \left(\frac{LeadLeng}{FolLeng}\right)^{0.5} \tag{3-4}$$

where

FolLeng = length of the trailing vehicle (ft)

LeadLeng = length of the leading vehicle (ft)

It should be noted that the value of 14 used in Equation 3-4 is the shortest length of vehicle defined in CORSIM.

Finally, if the adjusted *DTP* value is not less than 0.25, it will be compared to a generated uniform random number between 0 and 1. If the adjusted *DTP* value is less than the random number, the subject vehicle will continue to follow the leading vehicle. If the adjusted *DTP* value is greater than or equal to the random number, the subject vehicle will initiate a passing maneuver, subject to other constraints as described in the following section.

3.1.2.2 Constraints governing whether a pass will be initiated

If it has been determined that a vehicle wants to initiate a passing maneuver, the following issues are considered.

Step 1. Is the vehicle in a passing-allowed section? The program logic currently dictates that all passing maneuvers must be initiated in a passing-allowed section of the roadway (i.e., skip striping in the applicable direction)[2]. However, it is possible for a passing maneuver to be completed in a no-passing-allowed section, consistent with the field observations from Harwood et al. (2008) study. This is described in more detail under step 8.

Step 2. Check whether a vehicle upstream of the subject vehicle is performing a passing maneuver. If the subject vehicle is currently in the process of being passed by another vehicle, the subject vehicle will not initiate its passing maneuver.

Step 3. Check whether the maximum number of allowed passing maneuvers is currently in progress. The number of vehicles that can be simultaneously executing a passing maneuver in the oncoming lane is limited to three per each platoon of

[2]For computational efficiency reasons, this check is actually performed before the desire to pass calculations.

vehicles. Thus, the maximum number of vehicles that can be executing a passing maneuver along the defined length of highway is three times the number of platoons within that defined length of highway. A platoon is defined by a leading vehicle that is not in a following mode and trailing vehicles that are all considered to be in a following mode.

Step 4. Check the number of vehicles that must be passed to complete the passing maneuver. A vehicle is prevented from starting a passing maneuver when, due to insufficient gaps for merging between the vehicles ahead, there are more than five vehicles that would need to be passed.

Step 5. Check whether any queued vehicle exists in the 1-mi downstream segment. A vehicle whose speed is equal to or less than 30 ft/s in the 1-mi downstream segment from the subject vehicle is considered to be queued. If any queued vehicle is found, the subject vehicle will not initiate its passing maneuver.

Step 6. Determine the required passing sight distance. If the subject vehicle is allowed to initiate a passing maneuver per the above constraints, then the passing sight distance (PSD) will be calculated. PSD is the minimum distance necessary between the potential passing vehicle and an oncoming vehicle that will still allow the potential passing vehicle to safely initiate and complete a passing maneuver of a leading slower vehicle in the oncoming lane. If the horizontal and/or vertical alignment aspects of a highway do not provide unobstructed sight distance at least equal in length to the PSD, then the highway is typically striped with solid yellow center lines (i.e., no-passing

allowed)[3]. Both A policy on Geometric Design of Highways and Streets (AASHTO, 2004) and the Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) (FHWA, 2003) provide minimum PSD values for design and marking. However, their respective recommended PSD values vary significantly, with the AASHTO-recommended values being considerably larger than the MUTCD-recommended values.

Harwood et al. (2008) proposed recommendations on the adequacy of current procedures and guidelines used to estimate minimum PSD requirements for highway design and pavement marking. Based on the literature review and the field study, Harwood et al. (2008) indicated that the Glennon model (1988) and the Hassan et al. model (1996) are the most reasonable models, which result in considerably shorter PSD values than the AASHTO-recommended values, much closer to the MUTCD-recommended values. Furthermore, the Harwood et al. study argued that crash statistics for two-lane highways do not provide much support for the notion that the current practice for marking passing zones (based on the MUTCD criteria) is in need of revising.

Based on the field data results of the Harwood et al. (2008) study, the authors recommend the following assumptions be used when applying the Glennon and Hassan et al. models:

- The speeds of the passing and opposing vehicles are equal and represent the design speed of the highway.

[3]CORSIM does not compute available passing sight distance based on the specified roadway geometry; thus, it is necessary for the user to explicitly identify the allowable passing zones along the length of the highway.

- The vehicle being passed travels at uniform speed and the speed difference between the passing vehicle and vehicle being passed is 12 mi/h.
- The passing vehicle has sufficient acceleration capability to reach the specified speed difference relative to the passed vehicle by the time it reaches the critical position (same as the assumption that each vehicle will travel at the constant speed after the critical position).
- The lengths of the passing and passed vehicles are 19 ft.
- The passing driver's perception-reaction time in deciding to abort a passing vehicle is 1 second.
- The deceleration rate used in aborting a pass is 11.1 ft/s^2.
- For a completed or aborted pass, the headway between the passing and passed vehicles is 1 second.
- The minimum clearance headway between the passing and opposing vehicles at the point at which the passing vehicle returns to its normal lane is 1 second.

By using these assumptions, the Glennon and Hassan et al. models result in PSD values similar to the MUTCD-recommended PSD values.

The default PSD model used in CORSIM is the AASHTO model (Table 3-1 and Figure 3-2 for the details). However, it is also possible to use the MUTCD PSD values instead. Users can modify several CORSIM inputs to implement the MUTCD PSD values (Table 3-2 for the revised values of the variables).

If the available sight distance for the subject vehicle is equal to or greater than the minimum required PSD value, the following check is made.

Step 7. Check length of passing zone and compare to the minimum passing zone length. The length of the passing zone (as indicated by roadway markings) is compared to the minimum passing zone length (which is equal to distance traveled by the passer before it returns to the normal lane during the passing maneuver, as suggested by AASHTO). A vehicle will not initiate a passing maneuver unless the

available passing zone length is equal to or greater than the minimum passing zone length. If the marked passing zone length is greater than the minimum passing zone length, then the following check is made.

Step 8. Determine effective passing zone length and compare to the distance needed to complete the pass. As mentioned earlier, it is possible for a vehicle to complete its passing maneuver in a section of roadway marked as no-passing allowed. The length of the available passing zone (from the current position of the passing vehicle) is initially determined from just the roadway markings. This value is then adjusted based on the permissible amount of distance beyond the marked passing zone allowed for the passing vehicle's driver type. This results in an effective passing zone length, calculated as follows:

$$IllegalPassDistPct = \begin{cases} MinPct\,, \text{ if Driver type} = 1 \\ MaxPct\,, \text{ if Driver type} = 10 \\ \text{Linear interpolation, otherwise} \end{cases} \tag{3-5}$$

where

IllegalPassDistPct = allowable percentage of the total passing distance beyond the passing zone (as indicated by roadway markings)

MinPct = allowable percentage of the total passing distance corresponding to vehicle of driver type 1 (default = 0)

MaxPct = allowable percentage of the total passing distance corresponding to vehicle of driver type 10 (default =25)

$$DistAvailForPass = AvailPassZoneDist + AvailPassZoneDist \times \frac{IllegalPassDistPct}{100} \quad (3\text{-}6)$$

where

DistAvailForPass = the total amount of distance available for passing, based on roadway markings and allowable distance downstream of marked passing zone

AvailPassZoneDist = length of marked passing zone available at beginning of passing maneuver

The distance needed to complete the pass (*DNTCP*), which before the passing maneuver is initiated is equal to the estimated distance traveled by the subject vehicle during its passing maneuver (in the PSD calculations in step 6), is compared to the distance available for the pass (i.e., effective passing zone length). If the *DNTCP* is less than the effective passing zone length, then the passing maneuver can be initiated.

3.1.2.3 How a potential passer executes its passing maneuver?

If all the requirements discussed in the previous section are satisfied for a potential passing vehicle, it will initiate the passing maneuver. The general logic of executing a passing maneuver is divided into three stages, as illustrated in Figure 3-3.

Stage 1: Initiate passing in the normal lane. At the beginning of a passing maneuver, the potential passer starts to accelerate at the acceleration rate based on its own speed according to the AASHTO criteria or revised AASHTO criteria for approximating the MUTCD criteria. At the same time, the potential passer moves over to the opposing lane.

Stage2: Passing in the opposing lane. After the potential passer moves into the opposing lane, it will keep on accelerating until it reaches a speed 12 mi/h (this value

can be changed on RT 155) greater than the speed of the vehicle being passed[4]. Meanwhile, the variable *DNTCP* (distance needed to complete pass) is compared to the variable *DTC*[5] (distance to collision with the oncoming vehicle) every time step (Figure 3-4 for an illustration of these variables). For the subject vehicle who finds that more than one vehicle need to pass before it initiates the passing maneuver, the *DNTCP* is based on the position of the most downstream vehicle to be passed.

As mentioned previously, before the passing maneuver is initiated, the *DNTCP* is equal to the estimated distance traveled by the subject vehicle during its passing maneuver (in the PSD calculations in step 6). However, once the passing maneuver is in progress, the *DNTCP* is continually changing (generally decreasing). For calculating *DNTCP*, the time needed to complete passing (*TNTCP*) is first calculated. Since the potential passer will stop accelerating when its speed is 12 mi/h (i.e., 17.6 ft/s) greater than the vehicle being passed, the value of *TNTCP* is computed

$$TNTCP = \begin{cases} t_{accel} - (t - t_0) + t_{const}, & 0 \le t - t_0 \le t_{accel} \\ t_{const} - (t - t_0 - t_{accel}), & t - t_0 > t_{accel} \end{cases} \tag{3-7}$$

$$t_{accel} = \frac{v_2(0) + m - v_1(0)}{a_1} \tag{3-8}$$

$$t_{const} = \frac{ClearGap - (VehPos_1 - VehPos_2)}{m} \tag{3-9}$$

where

$TNTCP$ = time needed to complete passing (s)

t_{accel} = time needed to reach the maximum passing speed (s)

[4]Based on NCHRP Report 605 (Harwood et al, 2008).

[5]The *DTC* will be infinite when no oncoming vehicle is present.

t_{const} = time needed to complete passing after the accelerating period (s)

t_0 = time step when the passing maneuver initiates (s)

t = current time step (s)

$v_1(0)$ = initial speed of the passing vehicle (ft/s)

$v_2(0)$ = initial speed of the vehicle being passed (ft/s)

m = speed differential between the passing vehicle and the vehicle being passed when the passing vehicle reaches the maximum passing speed

a_1 = acceleration of the passing vehicle (ft/s^2)

$VehPos_1$ = position of the passing vehicle relative to the upstream end of the segment when it stops accelerating (ft)

$VehPos_2$ = position of the vehicle being passed relative to the upstream end of the segment when the passing vehicle stops accelerating (ft)

Once the *TNTCP* is obtained, the *DNTCP* can be calculated by

$$DNTCP = \begin{cases} \{v_1(t) + a_1[t_{accel} - (t - t_0)]\} \times TNTCP - \frac{1}{2} a_1 [t_{accel} - (t - t_0)]^2, & 0 \le t - t_0 \le t_{accel} \\ v_1(t) \times TNTCP, & t - t_0 > t_{accel} \end{cases} \tag{3-10}$$

where

v_1(t) = speed of the passing vehicle at time step *t* (ft/s)

v_2(t) = speed of the vehicle being passed at time step *t* (ft/s)

$$DTC = (VehPos_3 - VehPos_1) - v_3(t) \times \frac{(VehPos_3 - VehPos_1)}{v_1(t) + v_3(t)} \quad (3\text{-}11)$$

where

$VehPos_3$ = position of the oncoming vehicle relative to the upstream end of the segment at time step *t* (ft)

v_3(t) = speed of the oncoming vehicle at time step *t* (ft/s)

After the subject vehicle moves onto the opposing lane to perform its passing maneuver, the value of *DNTCP* is compared to the value of *DTC* at every time step until the passing maneuver is completed. The result of this comparison leads to the following different situations that must be considered.

If the *DNTCP* is less than the *DTC*, the passer will continue its passing maneuver as planned. Another issue related to the completion of passing is the gap size in front of the vehicle being passed, which is included in the calculation of *DNTCP*. The passer requires a certain gap size to be able to return to the normal lane in order to complete passing. The default value for this gap size (i.e., ClearGap in Figure 3-4) is 75 ft, but can be revised on Record Type 155. Normally, the passing vehicle only accepts the gap in front of the vehicle being passed if it is greater than or equal to the minimum gap. In some cases, the passing vehicle will consider to pass the next vehicle if the gap in front of the vehicle being passed is not sufficient. However, if the passing vehicle has already passed 5 vehicles, it will be forced to squeeze back to the normal lane.

If the *DNTCP* is greater than or equal to the *DTC*, the decision on whether the passing vehicle will continue or abort its passing maneuver is dependent on the relative position of the passing vehicle to the passed vehicle. A study by Harwood and Glennon (1976) defined the critical position as the point at which the sight distances required to

abort the pass and to complete the pass are equal. Two studies, one by Vanvalkenburg et al. (1971) and one by Weaver et al. (1972) independently recognized a key position of a passing maneuver occurs at the point where the passing driver can no longer safely abort the pass and is, therefore, committed to complete it. However, no unanimously accepted definition of the critical position currently exists. Harwood et al. (2008) suggests that the critical position can be considered to be the point when the passing vehicle and vehicle being passed are directly abreast of one another. This definition is applied in the CORSIM logic. Two situations were identified as follows:

- The passer has not reached the critical position. If the passing vehicle has not yet reached the critical position when *DNTCP* is greater than *DTC*, it will abort the passing maneuver. The process of returning to the normal lane is similar to that of completing a passing maneuver. The gap behind the vehicle being passed will be checked for the return. The minimum gap here is also set as three times the length of the passing vehicle, and the passing vehicle will only accept a gap greater than or equal to the minimum gap.

- The passer has reached the critical position. If the passer has reached the critical position when *DNTCP* is greater than *DTC*, the passer will continue with completing the passing maneuver. In order to accommodate this without collision, the passing vehicle will speed up and/or the oncoming vehicle will decelerate. The specific amount of acceleration by the passing vehicle and/or deceleration by the oncoming vehicle is a function of the current acceleration rate of each vehicle, the passing vehicle's acceleration capabilities, the oncoming vehicle's deceleration capabilities, and the current *DTC* value. In the simulation, the vehicle being passed is set to be aware of this expedited passing, and it usually slows down to assist the passing maneuver. Occasionally, the vehicle being passed may not slow down even though it is aware of the situation, in such cases, the passing vehicle will be forced to abort its passing maneuver.

Stage3. Return to the normal lane. For a passing maneuver being completed, the passer will return to the normal lane in front of the vehicle being passed when the gap is sufficient. For an aborted passing maneuver, the passer will return to the normal lane behind the vehicle being passed if there is a sufficient gap. The existing mandatory lane changing logic in CORSIM is utilized for this situation.

Under certain conditions, the passing vehicle will consider passing more than one vehicle. Specifically, if the passing vehicle's speed is greater than the speed of the vehicle in front of the current vehicle being passed, and the gap in front of the current vehicle being passed is insufficient, the passing vehicle will attempt to pass the vehicle in front of the current vehicle being passed, subject to the logic and constraints as previously discussed.

3.1.3 Passing in a Passing-lane Section

A passing lane is defined as a lane added to improve passing opportunities in one direction of travel on a conventional two-lane highway. Although it may vary by jurisdiction, the logic implemented in this simulation program assumes that slower vehicles will move to the right lane in a passing-lane section and the passing vehicles will pass on the left (usually this is indicated by a sign such as 'Keep Right Unless Passing'). Ideally, each driver will drive following the guidance. However, it is recognized that this does not always happen; thus, the developed logic allows for the possibility that an impeding vehicle will not move over.

For each vehicle in a passing-lane section (hereafter referred to as the subject vehicle), the logic first checks the headway between this subject vehicle and the vehicle immediately behind, and then checks the headway between this subject vehicle and the vehicle immediately ahead. If the subject vehicle has a vehicle behind it in following mode (i.e., headway ≤ follower headway threshold) and is not in following mode itself, the willingness to move over (*WTMO*) to the right lane of the passing-lane section for the subject vehicle will be considered. The value of *WTMO* is determined as follows:

$$WTMO = \begin{cases} 0, & \text{if } SVS > (FFS + 5) \\ (FFS + 5 - SVS)/15, & \text{if } (FFS - 10) < SVS < (FFS + 5) \\ 1, & \text{if } SVS < (FFS - 10) \end{cases} \quad (3\text{-}12)$$

where

$WTMO$ = willingness to move over

SVS = subject vehicle speed (mi/h)

FFS = free-flow speed (mi/h)

This parameter will be adjusted by dividing by the square root of driver type if the length of the subject vehicle is less than 40 ft (the length of a single-unit truck), otherwise it remains as the original value (not dividing by the square root of driver type results in a higher probability to move over for trucks). The adjusted willingness to move over is compared to a generated uniform random number between 0 and 1, and if it is greater than the random number the subject vehicle will move over to the right lane; otherwise, it will stay in the left lane.

The existing CORSIM logic is utilized for determining when a vehicle will move from the right lane (which drops at the end of the passing-lane section) back to the normal lane. Generally, the discretionary lane change logic will apply for vehicles in the right lane (either slow vehicles that moved out of the way, or faster vehicles that are trying pass slower vehicles that did not move over) until such a vehicle gets near to the end of the right lane, in which case the mandatory lane change logic will be applied.

3.2 Integrating Signalized Intersections into a Two-Lane Highway Facility

CORSIM consists of two microscopic stochastic simulation models: 1) NETSIM, which models traffic on urban streets, including intersections, and 2) FRESIM, which

models traffic on freeways. The previous two-lane highway modeling features were built into the FRESIM model.

Since the capability to connect FRESIM links to NETSIM links already existed, it was not necessary to make any special modifications to the underlying CORSIM models to accommodate the modeling of signalized intersections within a two-lane highway. Thus, the rest of this section simply provides guidelines for defining two-lane highway networks that contain signalized intersections, particularly the interface (i.e., connection) between the two-lane highway links and the signalized intersection links.

The signalized intersection and two-lane highway features should be defined as usual in the TRF file. Then, according to the CORSIM node numbering scheme[6], two nodes numbered between 7000 and 7999 should be added along the two-lane highway direction as the exit nodes for the NETSIM links, and also the entry nodes for the FRESIM links. The length of the link between a “7” node and a NETSIM node should be no less than 100 feet (Figure 3-5), since some vehicles with high speeds could possibly “jump over” a short link in a single time step. It is recommended that no-passing zone markings extend at least 200 feet upstream and downstream of the intersection (DelDOT, 2009). Passing in the opposing lane is not allowed on NETSIM links; thus, if the NETSIM links are less than 200 feet in length, no-passing zone markings should be specified for the adjoining FRESIM links.

[6]Node numbers 1 through 6999 can be used for internal nodes. Node numbers 7000 through 7999 can be used for interface nodes and node numbers 8000 through 8999 can be used for entry or exit nodes.

3.3 Inputs and Outputs of the Simulation Program

3.3.1 New Inputs

Several new inputs were added to CORSIM to accommodate user revision of various parameters for the two-lane highway modeling logic. The new inputs generally fall into two categories as follows:

- **Facility property inputs.** This type of inputs allows users to create a two-lane highway segment and make the appropriate marking configurations on it. Users can also define a two-lane highway by direction, by specifying the links that comprise the highway direction.
- **Passing maneuver inputs.** This type of inputs allows users to modify the values of the components in the passing models, such as follower headway threshold, speed differential between passing vehicle and passed vehicle, or the inputs used to calculate the minimum passing sight distance required.

The details of the new inputs are listed in Table 3-3.

3.3.2. New Outputs

Several new outputs were added to CORSIM to accommodate a variety of new performance measures and passing lane diagnostics.

3.3.2.1 Performance measure outputs

In the HCM 2010, the level of service on a two-lane highway is defined based on both *PTSF* and *ATS*. In the simulation, these two measures can be selected as outputs for users' purpose. And as the potential performance measure, follower density is added to the CORSIM outputs. According to the definition, follow density can be calculated by

$$\text{Follower density} = \text{Percentage of followers} \times \text{Traffic density} \tag{3-13}$$

All the three performance measures can be output in the simulation program either on link basis or on facility basis. Facility-wide values of *ATS*, *PTSF*, and follower density are included in the default output file (.out) of CORSIM.

In addition, another potential performance measure percent delay (*PD*) is also added as one of the outputs, for developing the analytical methodology in Chapter 4. The value of *PD* is determined by

$$PD = \frac{DelayTravelTotal}{MoveTimeTotal} \tag{3-14}$$

where

DelayTravelTotal = total time that all vehicles were delayed in the network (CORSIM definition)

MoveTimeTotal = total theoretical time for discharged vehicles to travel the length of the link if moving unimpeded at the free-flow speed (CORSIM definition)

3.3.2.2 Passing statistics outputs

As passing is the most significant feature of two-lane highway operations, the outputs of passing statistics can be employed as the supplementary information for analyzing two-lane highway operations. Users can get the number of attempted passes, the number of aborted numbers, and the number of completed passes in a specific simulation run. Other information such as the distance traveled by the passer in the opposing lane during the simulation time period can be obtained as well.

Most of these outputs are specified through the output processor tool in TSIS. The outputs specified in the output processor dialogs are generally written to ‘.csv’ files that can then be loaded into a spreadsheet program. The details of the new outputs are listed in Table 3-4.

3.4 Preliminary Tests on the New Simulation Program

The new simulation program is tested by running a set of scenarios, to verify whether the simulation results are consistent with commonly accepted traffic-flow theories.

A large number of experiments were conducted for the purpose of comparing the simulation results to the traffic-flow theories for two-lane highways. In these experiments, follower density, *PTSF*, and *ATS* were chosen as the primary performance measure outputs.

3.4.1 Experimental Design

It is expected that not only the traffic operations on basic two-lane highways are simulated and tested, but also the traffic operations on two-lane highways with various features such as passing lanes and signalized intersections. Besides the different facility geometries for different sets of experiments (Figure 3-6 for configuration of each facility), the appropriate options of two-way flow volume, directional volume split and percentage of heavy vehicles are applied to each experiment, as listed in Tables 3-5 and 3-6.

For all the experiments, 10 iterations of each experiment were run, and the average simulation results of the 10 iterations were obtained and analyzed. The findings from the data analysis will be discussed in the following sections, as well as the conclusions for the preliminary tests.

3.4.2 Findings of Data Analysis

For capturing the difference of performance measures resulted by different features on two-lane highways, the findings of the data will be summarized for the basic segment tests, passing lane tests and signalized intersection tests, respectively.

3.4.2.1 Basic segment tests

The tests in this part uses basic two-lane highway segment as the testing facility, without any special features. The only difference regarding the facility geometry is the center line markings, which determine the lengths of passing zones. Therefore, the performance measure results for different passing zone conditions will be compared with each other, and presented in Figure 3-7 to 3-9. Because only 0% heavy vehicles was used in the basic segment tests with 50% passing zone, for the convenience of comparing the results between different sets of basic segment tests, only the scenarios without trucks are included in this part.

Directional average speed vs. volume. Figure 3-7 illustrates the relationship between directional average speed and two-way flow rate. These results follow the expected trends and relationships: When the traffic flow rates are approximately equal on each lane of the two-lane highway, the average speed for each direction is similar; as the traffic directional split becomes more imbalanced, the average speeds in the peak direction are lower than in the off-peak direction, and the average speeds decline with increasing flow rates in each direction.

The average speeds are higher for the 100% passing allowed condition than the no-passing allowed condition, for each flow-rate scenario, until you get to high flow rates (approximately 2,500 veh/h two-way flow rate), at which point the average speeds are similar. High flow rates severely limit the passing opportunities; thus, essentially creating a no-passing-allowed condition. Also, the improvement to peak direction average speed for the passing-allowed condition relative to the no-passing-allowed condition increases with increasing traffic directional split imbalance, as expected. Also

as expected, whether for the passing-allowed or no-passing-allowed condition, the average speed levels off in the moderate to high two-way flow rate range.

As to the 50% passing-allowed condition, the performance for the EB direction is initially (i.e., at lower flow rates) in between the performance of the two 100% conditions until it converges to the 100% no-passing-allowed values, which it does at a lower flow rate than the 100% passing-allowed condition. This suggests that the effect of proportion of passing zone on the performance measures is not linear. For the WB direction, the average speeds for the 50% passing-allowed condition generally follow those for the 100% passing allowed condition. This may seem to be a somewhat unexpected result, but it is plausible. The passing logic allows a vehicle to complete its pass a certain distance downstream of the end of the passing zone (as mentioned in section 3.1.2.2). The passing logic also requires a passing maneuver to be completed before it exits the coded highway facility. For the facility configuration shown in Figure 3-6C, passing vehicles traveling in the EB direction must complete their pass before reaching the end of the 5-mi passing zone; whereas passing vehicles traveling in the WB direction can complete their pass partially into the no-passing allowed zone. Thus, the effective passing zone length is longer for the WB direction than it is for the EB direction, which explains why the 50% passing-allowed results are better in the WB direction than in the EB direction.

Directional *PTSF* vs. volume. Figure 3-8 illustrates the relationship between directional *PTSF* and two-way flow rate. The results of the relationship between the *PTSF* values and flow rate are also as expected. When the traffic flow rates are approximately equal on each lane of the two-lane highway, the *PTSF* value for each

direction is similar; as the traffic directional split becomes uneven, the *PTSF* in the peak direction is higher than in the off-peak direction, and the *PTSF* value increases with increasing flow rates in each direction.

The *PTSF* values are lower for the 100% passing-allowed condition than the no-passing-allowed condition, for each flow rate scenario, until the two-way flow rates reaches approximately 2,500 veh/h, where the *PTSF* values are similar. This is consistent with the results for average speed. And the improvement to peak direction *PTSF* for the passing allowed condition relative to the no-passing-allowed condition increases with increasing traffic directional split imbalance. Also as expected, whether for the passing-allowed or no-passing-allowed condition, the *PTSF* levels off in the moderate to high two-way flow rate range.

Similar to the average speed results, for the EB direction, the *PTSF* results of the 50% passing-allowed condition is initially in between the performance of the other two conditions until it converges to the 100% no-passing-allowed values. And for the WB direction, the *PTSF* values for the 50% passing-allowed condition generally follow those for the 100% passing allowed condition.

Directional follower density vs. volume. Figure 3-9 illustrates the relationship between directional follower density and two-way flow rate. The results of the relationship between the follower density values and flow rate are as expected: When the traffic-flow rates are equally split on each lane of the two-lane highway, the follower density for each direction is similar; as the traffic directional split becomes more imbalanced, the follower density in the peak direction is higher than in the off-peak direction, and increases with increasing flow rates in each direction.

The follower densities are lower for the 100% passing-allowed condition than the no-passing-allowed condition. For each flow rate scenario, until you get to high flow rates (approximately 2,500 veh/h two-way flow rate), at which point the follower densities become similar. Again, this is because high flow rates severely limit the passing opportunities, essentially creating a no-passing-allowed condition. Also, the improvement to peak direction follower density for the passing allowed condition relative to the no-passing-allowed condition increases with increasing traffic directional split imbalance, as expected.

As to the comparison between the 50% passing-allowed conditions and the other two conditions, the results for the follower density values follow the same trends and relationships as described for the average speed results.

3.4.2.2 Two-lane highway with a passing lane tests

As passing lanes are designed for improving passing opportunities by providing slow vehicles with an extra lane to move over, more slow vehicles are expected in passing-lane tests to better reflect the benefits of adding a passing lane. Therefore, 10% of traffic flow is set as trucks, and the results are compared with the ones for the basic segment tests with no-passing zones

Directional average speed vs. volume. Figure 3-10 illustrates the relationship between directional average speed and two-way flow rate.

The results for average speed generally follow the expected trends and relationships. In the EB direction (where the passing lane is present), the average speeds are generally higher than the speeds for the 100% no-passing-allowed and no-passing lane condition. The average speeds for the WB direction (with no passing lane) are similar to the speeds for the 100% no-passing-allowed condition. The results

indicate that the improvement to average speed will diminish around 2,500 veh/h. This is because at high flow rates, there is enough turbulence (particularly from trucks) created at the merge point at the end of the passing lane to offset the addition of the passing lane. The WB results show that the passing lane added in the EB direction does not offer any speed benefit to the WB direction, which is as expected. Other trends and relationships in the average speed results are as previously described for the other average speed results.

Directional *PTSF* vs. volume. Figure 3-11 illustrates the relationship between directional *PTSF* and two-way flow rate.

The presence of the passing lane generally provides similar improvements to the *PTSF* values in the EB direction as found for the average speed results. Likewise, for the WB direction (with no passing lane), the *PTSF* results are virtually identical.

Directional follower density vs. volume. Figure 3-12 illustrates the relationship between directional follower density and two-way flow rate.

The presence of the passing lane generally provides similar improvements to the follower density values in the EB direction as found for the average speed and *PTSF* results. Likewise, for the WB direction (with no passing lane), the follower density results are virtually identical.

3.4.2.3 Two-lane highway segment with a signalized intersection test

A test on a two-lane highway segment with a signalized intersection is also done to see the impact of adding a signalized intersection. As shown in Figure 3-6E, passing is not allowed in the NETSIM links used for modeling the signalized intersection operations. Additionally, passing is not allowed on the adjacent upstream and downstream 600-foot segments. Passing is allowed on all other segments. The flow rate

for the major highway is 500 veh/h (no trucks) for each direction, and the flow rate for the minor crossing street is 100 veh/h (no trucks) for each direction. The cycle length is 100 seconds. Only one experiment was run for this configuration, just to demonstrate that the effect of the signalized intersection is reflected in the facility performance measure results. The testing results produced by CORSIM were compared with the ones with the basic two-lane highway (Table 3-7).

The results confirm the expectation that the presence of a signalized intersection will worsen the operational performance on the whole facility. Additional investigation of the effects of signalized intersections along a two-lane highway will be presented in Chapter 4.

3.4.3 Conclusions of the Preliminary Tests

The results of the preliminary tests indicate that the new simulation capability in CORSIM can model traffic operations on complex two-lane highways (e.g., two-lane highway with occasional signalized intersections). It is also indicated that the two-lane modeling logic applied in the simulation program is reasonably consistent with theories and field observations discussed in the literature, and the modeling results are generally reasonable and consistent with expected traffic-flow theory for two-lane highways.

Table 3-1. Components and values of passing sight distance (AASHTO, 2004)

Component of passing maneuver	Speed range (mi/h)			
	30–40	40–50	50–60	60–70
	Average passing speed (mi/h)			
	34.9	43.8	52.6	62.0
Initial maneuver:				
^{+}a	1.40	1.43	1.47	1.50
$^{++}t_1$	3.6	4.0	4.3	4.5
$^{+++}d_1$	$1.467t_1\left(v - {}^{*}m + \frac{at_1}{2}\right)$			
Occupation of left lane:				
$^{**}t_2$	9.9[7]			
$^{***}d_2$	$1.467 \times {}^{\#}v \times t_2$			
Clearance distance:				
$^{\#\#}d_3$	100	180	250	300
Opposing vehicle:				
$^{\#\#\#}d_4$	$0.667 \times d_2$			
Minimum PSD	$d_1 + d_2 + d_3 + d_4$			

+acceleration of passing vehicle when initiating passing maneuver (mi/h/s)
++time required for initial maneuver (s)
+++initial maneuver — distance traveled from start of passing maneuver until passing vehicle encroaches upon oncoming lane (ft)
*difference in speed of passed vehicle and passing vehicle (mi/h)
**time spent traveling in the oncoming lane (s)
***the distance traveled by the passing vehicle from the point of encroachment in the oncoming lane to the point of return to the normal lane (ft)
#average speed of passing vehicle (mi/h)
##the shortest desirable distance between the front bumpers of the passing and opposing vehicle when the passing vehicle returns to the normal lane (ft)
###the distance traveled by the opposing vehicle during the time the passing vehicle travels from the position of being directly abreast of the vehicle being passed to the return to the normal lane (ft)

[7]The original AASHTO criteria included four separate values for t_2, based on the passing vehicle speed. The Harwood et al. (2008) study recommended a constant value of t_2, because no evidence has been found that the left lane travel time of a passing vehicle increases as the speed of the passed vehicle increases. The chosen value was the mean value of 9.9 seconds from their study.

Table 3-2. Revised values of the AASHTO PSD model variables for approximating the MUTCD PSD values

Component of passing maneuver	Speed range (mi/h)			
	30-40	40-50	50-60	60-70
	Average passing speed (mi/h)			
	34.9	43.8	52.6	62.0
	Initial maneuver:			
t_1	3.0	2.5	2.0	1.6
	Occupation of left lane:			
t_2	5.9	6.0	6.2	6.4
	Clearance distance:			
d_3	80	100	120	140

Table 3-3. New inputs for two-lane highway simulation

Parameter	Description	Range	Default	Field	Record Type
Passing Zone (See Appendix A for the corresponding marking configuration)	Specifies the center-line marking of a two-lane highway segment.	0 – Not Applicable 1 – Passing Not Allowed 2 – Passing Allowed 3 – Passing Allowed in Both Directions	0	74	20
Entry node of eastbound segment of a two-lane highway	Specifies the two-lane highway link's entry node number (for EB).	7000-8999	none	1-4	154
Entry node of westbound segment of a two-lane highway	Specifies the two-lane highway link's entry node number (for WB).	7000-8999	none	5-8	154
Follower Headway Threshold (s)	Specifies the follower headway threshold in seconds. It is used to set the time headway value that is used to determine whether a vehicle is in a following mode.	1.0 – 10.0	3	1-4	155
MinPct (%)	Specifies the illegal passing distance percentage acceptable for driver type 1 (most conservative driver).	0 – 100	0	5-8	155
MaxPct (%)	Specifies the illegal passing distance percentage acceptable for driver type 10 (most aggressive driver).	0 – 100	25	9-12	155
Minimum clearance distance between passing vehicle and passed vehicle (ft)	Specifies the shortest distance between the front bumper of the passed vehicle and the rear bumper of the passing vehicle, when the passing vehicle moves back to the normal lane, which allows the passing vehicle to safely complete its pass.	10-100	75	13-16	155
TimeToCollisionMin (s)	Specifies the time to collision that will cause a type 10 driver to abandon passing an additional vehicle during passing maneuver.	0-100	5	17-20	155

Table 3-3. Continued

Parameter	Description	Range	Default	Field	Record Type
TimeToCollisionMax (s)	Specifies the time to collision that will cause a type 1 driver to abandon passing an additional vehicle during passing maneuver.	0-100	10	21-24	155
Speed differential between passing vehicle and passed vehicle (mi/h)	Specifies the difference in speed between the passing vehicles and vehicle being passed when the passing vehicle has reached its top passing speed.	5-20	12	25-28	155
Impatience Value	Specifies the increment to the desire to pass value for each second of time spent wanting to pass.	0.000 – 0.001	0.001	61-64	155
Passing acceleration	Specifies the acceleration that will be applied to accelerate the passing vehicle to the desired passing speed when a passing maneuver is initiated.	a_1, if speed ≤ 40 mi/h a_2, if 40 < speed ≤ 50 mi/h a_3, if 50 < speed ≤ 60 mi/h a_4, if speed > 60 mi/h	a_1 = 1.40 mi/h/s a_2 = 1.43 mi/h/s a_3 = 1.47 mi/h/s a_4 = 1.50 mi/h/s	1-16	156
Time required for initial passing maneuver	Specifies the amount of time the passing vehicle spends moving from the normal lane to the point of encroachment in the oncoming lane	t_1, if speed ≤ 40 mi/h t_2, if 40 < speed ≤ 50 mi/h t_3, if 50 < speed ≤ 60 mi/h t_4, if speed > 60 mi/h	t_1 = 3.6 s t_2 = 4.0 s t_3 = 4.3 s t_4 = 4.5 s	17-32	156
Time spent traveling in the oncoming lane	Specifies the amount of time the passing vehicle spends from the point of encroachment in the oncoming lane until it returns to the normal lane.	t_1, if speed ≤ 40 mi/h t_2, if 40 < speed ≤ 50 mi/h t_3, if 50 < speed ≤ 60 mi/h t_4, if speed > 60 mi/h	t_1 = 9.9 s t_2 = 9.9 s t_3 = 9.9 s t_4 = 9.9 s	33-48	156
Minimum clearance distance	Specifies the minimum acceptable clearance distance between the passing vehicle and the first oncoming vehicle.	c_1, if speed ≤ 40 mi/h c_2, if 40 < speed ≤ 50 mi/h c_3, if 50 < speed ≤ 60 mi/h c_4, if speed > 60 mi/h	c_1 = 100 ft c_2 = 180 ft c_3 = 250 ft c_4 = 300 ft	49-64	156
Two-Lane Highways by Direction	Allows the user to define a two-lane highway by direction	N/A	N/A		190
Two-Lane Highway Direction Links	Used in conjunction with RT 190 to specify the links that comprise the highway direction	N/A	N/A		191

Table 3-4. New outputs for two-lane highway simulation (by direction)

Output	Description	Formula	Facility-based or link-based
Follower density	Indicates the average density of vehicles which are in the following mode.	Percentage of followers × Traffic density (Followers per mile)	Both
Percent time-spent-following	Indicates the average percentage of travel time spent by vehicles following in platoons behind slower vehicles.	$\frac{\sum_{\text{All vehicles}} \frac{\text{Travel time spent in platoons}}{\text{Total travel time}}}{\text{Total number of vehicles}}$ (%)	Both
Average travel speed	Indicates the average travel speed of vehicles.	$\frac{\sum_{\text{All vehicles}} \frac{\text{Total travel distance}}{\text{Total travel time}}}{\text{Total number of vehicles}}$ (mi/h)	Both
Delay Travel Per Move Time Ratio (*PD*)	Indicates the ratio of delay to free-flow travel time.	$\frac{\sum_{\text{All vehicles}} (\text{Actual travel time} - \text{Free flow travel time})}{\sum_{\text{All vehicles}} \text{Free flow travel time}}$ (%)	Both
Average speed of vehicles while performing a passing maneuver	Indicates the average travel speed of all the passers during their processes of passing maneuvers (i.e. from initiating passing to returning to the normal lane)	$\frac{\sum_{\text{All successful passers}} \frac{\text{Total distance traveled during passing maneuver}}{\text{Total travel time spent during passing maneuver}}}{\text{Total number of successful passers}}$ (mi/h)	Facility
Number of attempted passes	Indicates the total amount of the initiated passing maneuvers, including both completed and aborted passes, over the whole facility.	N/A	Facility
Number of aborted passes	Indicates the total number of passes that are initiated but not completed.	N/A	Facility
Clearance distance and/or time	Indicates the average clearance (in terms of distance or time) between passing vehicle and opposing vehicle when the passer completes the passing maneuver and returns to the normal lane.	$\frac{\sum_{\text{All successful passers}} \text{Clearance of the successful passers}}{\text{Total number of successful passers}}$ (ft or s)	Facility

Table 3-4. Continued

Output	Description	Formula	Facility-based or link-based
Average distance spent in opposing lane	Indicates the average distance the passer travels in opposing lane during the passing maneuver.	$\frac{\sum_{\text{All successful passers}} \text{Distance of all the successful passers in opposing lane}}{\text{Total number of successful passers}}$ (ft)	Facility
Average proportion of illegal passing distance	Indicates the average proportion of passing distance used beyond a passing zone, relative to the total passing distance.	$\frac{\sum_{\text{All successful passers}} \frac{\text{Passing distance beyond passing zone}}{\text{Total passing distance}}}{\text{Total number of successful passers}}$ (%)	Facility

Table 3-5. Preliminary experimental design (1) – facility geometry

	Facility length (mi)	% Passing zone	Terrain	Other features
Basic segment tests	10	100	Level	N/A
	10	0	Level	N/A
	10	50	Level	N/A
Passing lane tests	10	0	Level	A EB passing lane added from milepost 4 to 5
Signalized intersection test	3	0 on the adjacent upstream and downstream 600-ft segments, 100 on others	Level	A signalized intersection added in the middle

Table 3-6. Preliminary experimental design (2) – traffic inputs

Option	1	2	3	4	5	6	7	8	9
Two-way volume (veh/h)	100	500	1000	1500	2000	2500	3000	3500	4000
Directional splits (EB/WB)	50/50	*60/40	**70/30	-	-	-	-	-	-
% Heavy vehicle	***0	****10	-	-	-	-	-	-	-
Free-flow speed (mi/h)	65	-	-	-	-	-	-	-	-

*This split does not apply for two-way volume of 3500 and 4000 veh/h.
**This split does not apply for two-way volume of 3000, 3500 and 4000 veh/h.
***This percentage does not apply for the passing lane tests.
****This percentage does not apply for the basic segment tests with 50% passing zone.

Table 3-7. Two-lane highway with a signalized intersection testing results

Scenario		Avg. Speed (mi/h)	*PTSF* (%)	Follower Density (veh/mi)
Signalized intersection added	EB	54.70	46.38	1.53
	WB	54.67	46.87	1.56
No signalized intersection	EB	62.27	34.58	0.62
	WB	62.20	35.03	0.65

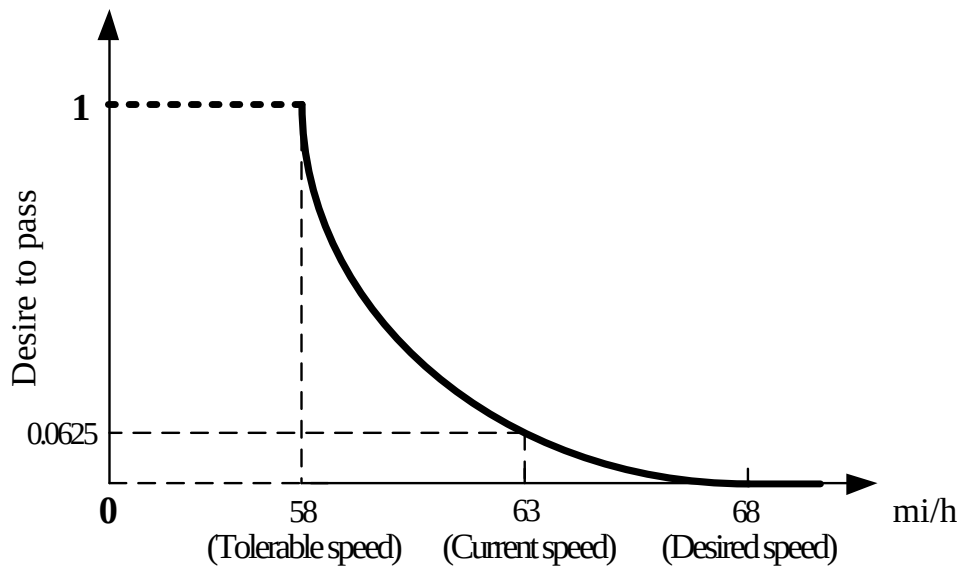


Figure 3-1. An example of Desire to Pass for a specific driver

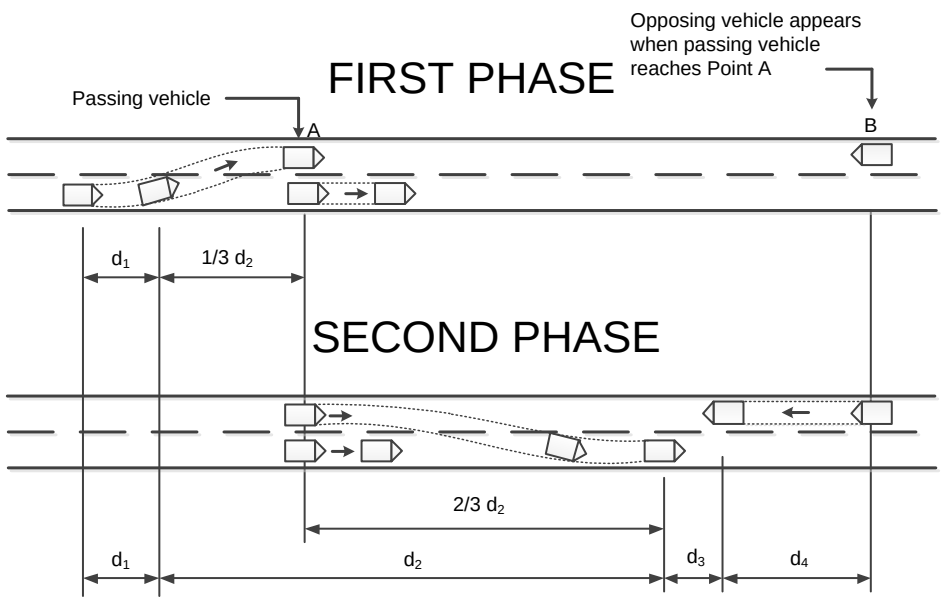


Figure 3-2. Elements of passing sight distance for two-lane highways (AASHTO, 2004)

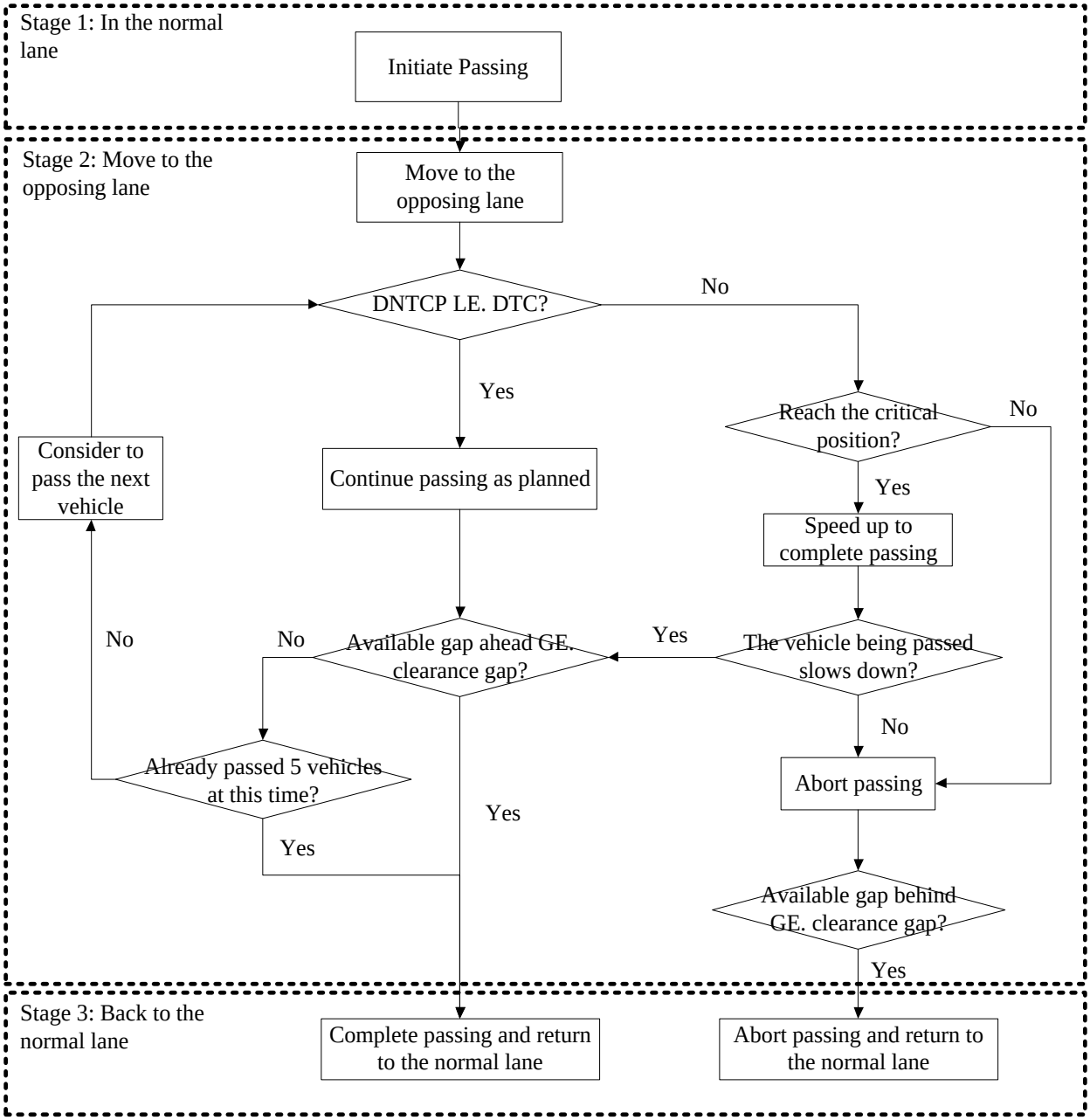


Figure 3-3. General passing maneuver process

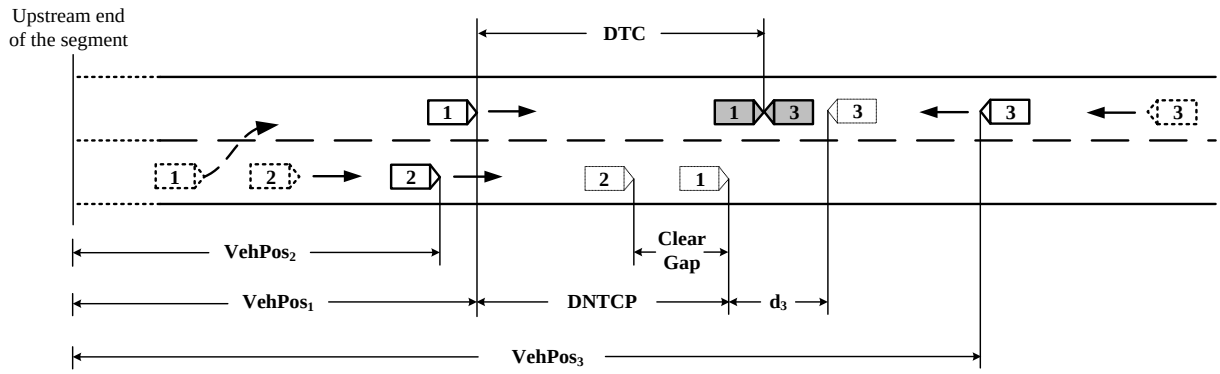


Figure 3-4. Illustration of *DNTCP* and *DTC*

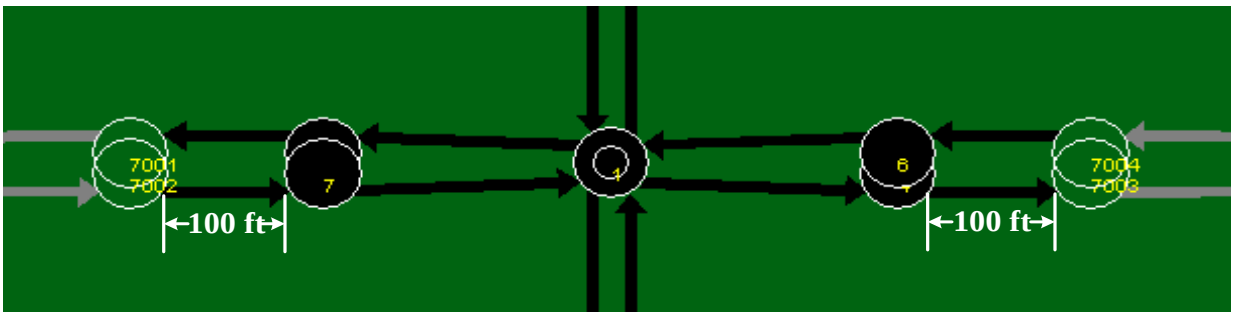


Figure 3-5. Example of the interface between FRESIM links and the NETSIM links

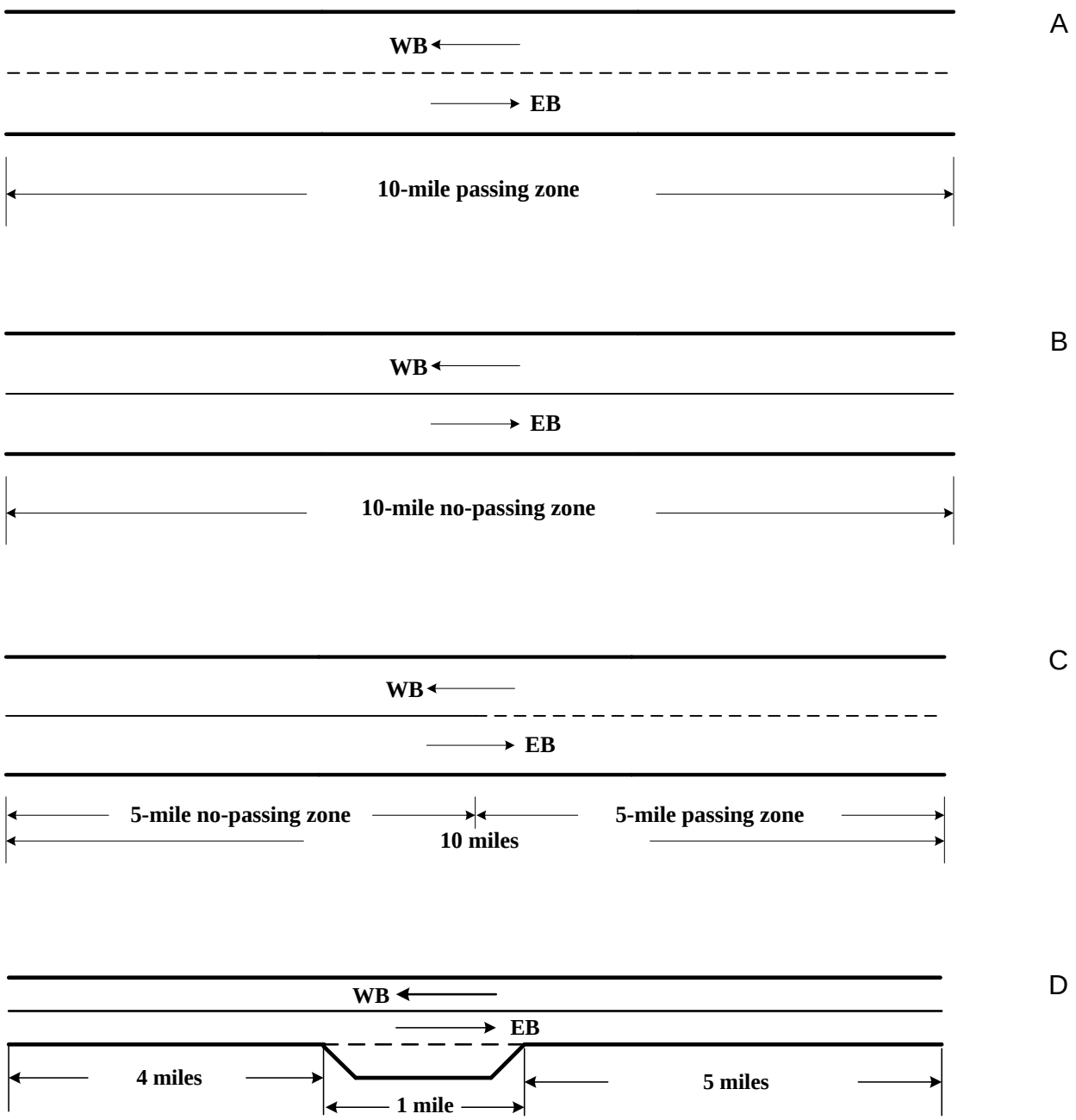


Figure 3-6. Configurations of testing facilities. A) Basic segment tests with100% passing zone B) Basic segment tests with no-passing zone C) Tests with 50% passing zone D) Tests with a passing lane E) Test with a signalized intersection

E

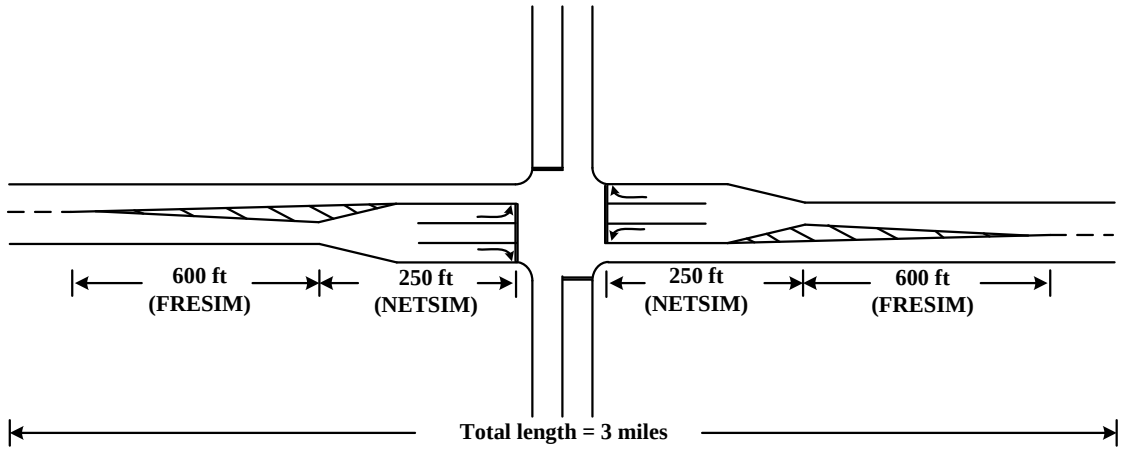


Figure 3-6. Continued

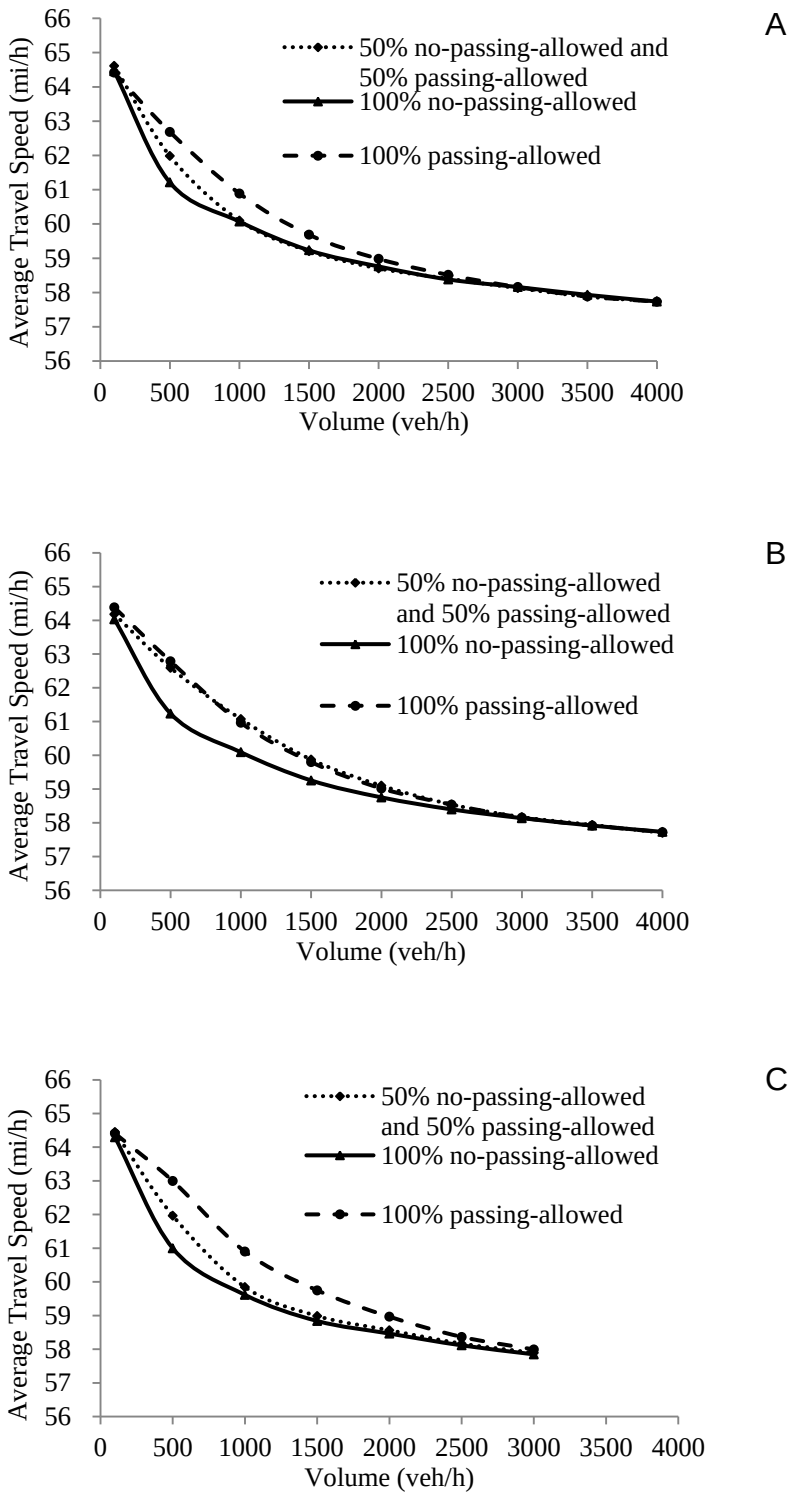
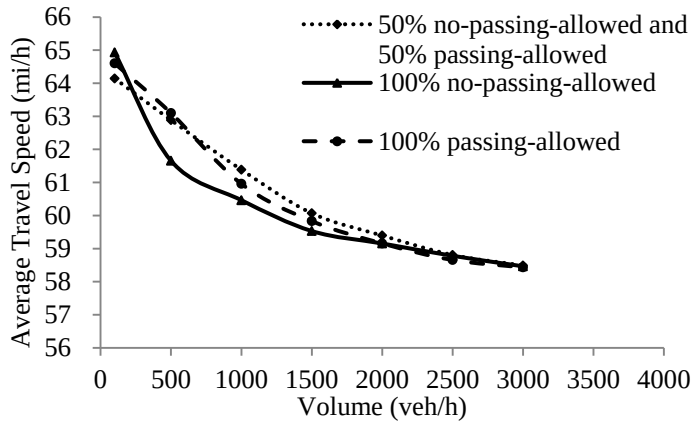


Figure 3-7. Directional average speed results for basic segment tests. A) EB Avg. Speed, 50/50 Volume Split B) WB Avg. Speed, 50/50 Volume Split C) EB Avg. Speed, 60/40 Volume Split D) WB Avg. Speed, 40/60 Volume Split E) EB Avg. Speed, 70/30 Volume Split F) WB Avg. Speed, 30/70 Volume Split

D



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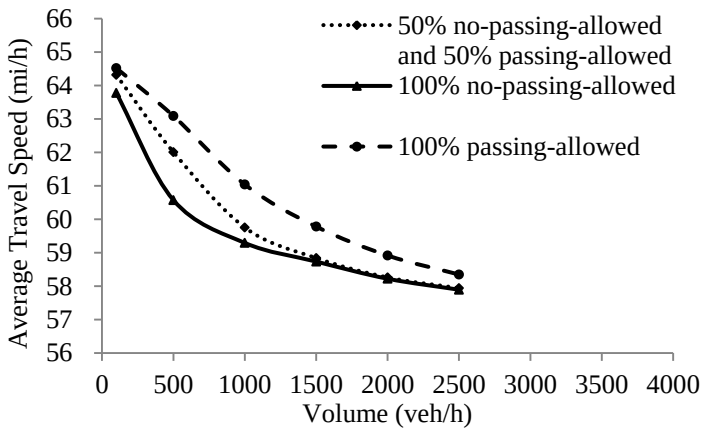

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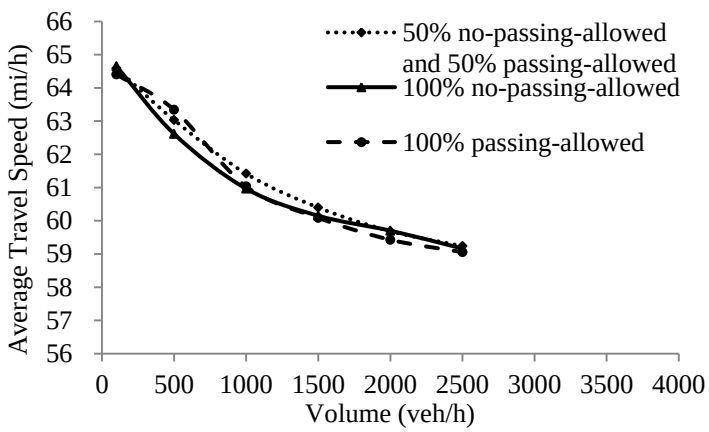


Figure 3-7. Continued

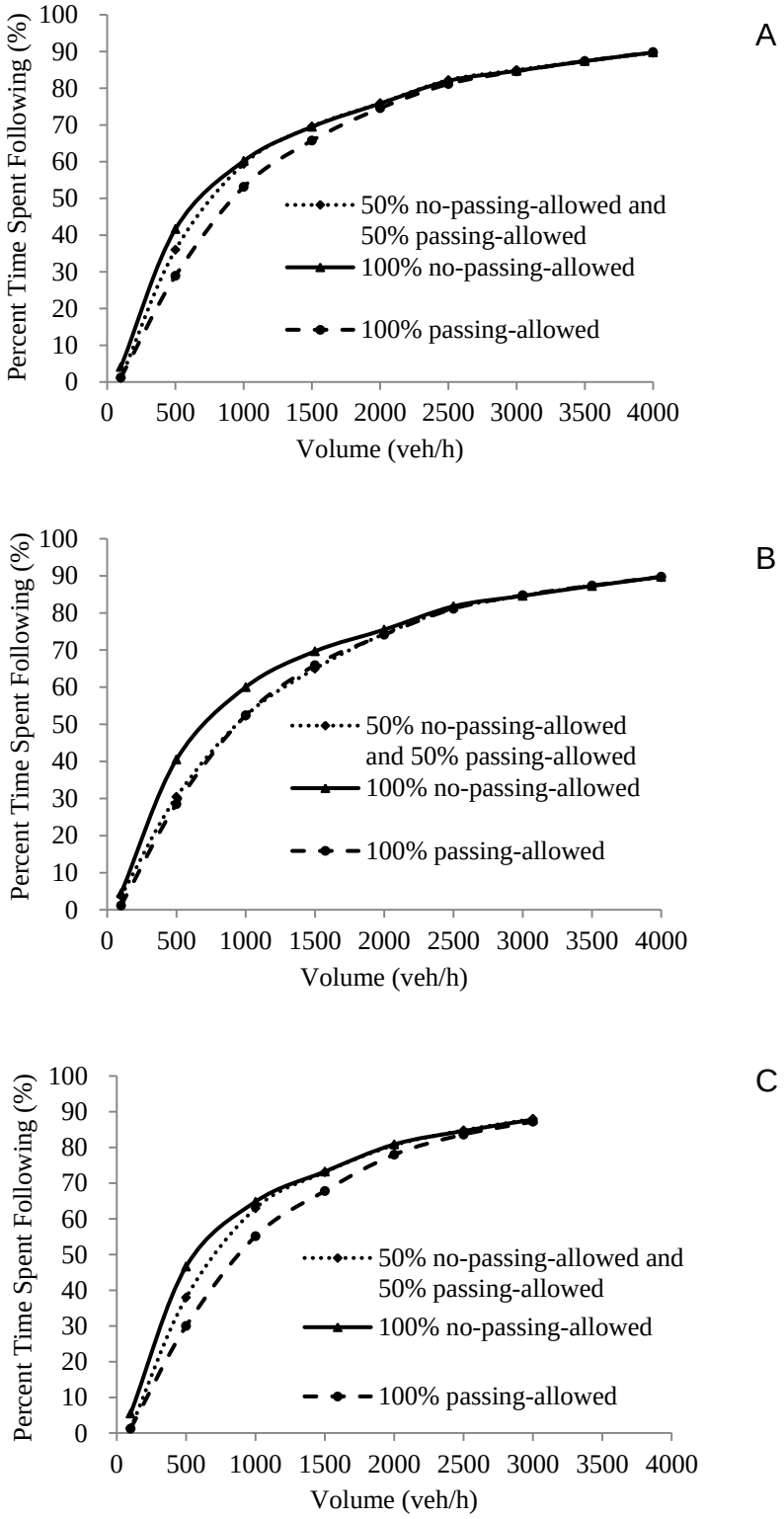
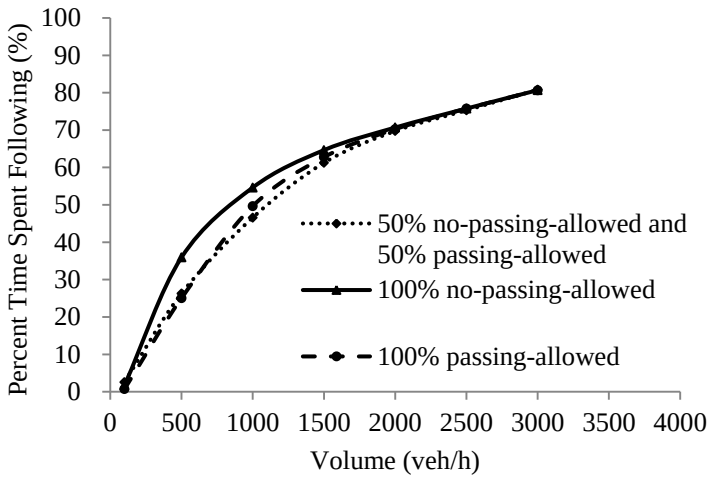
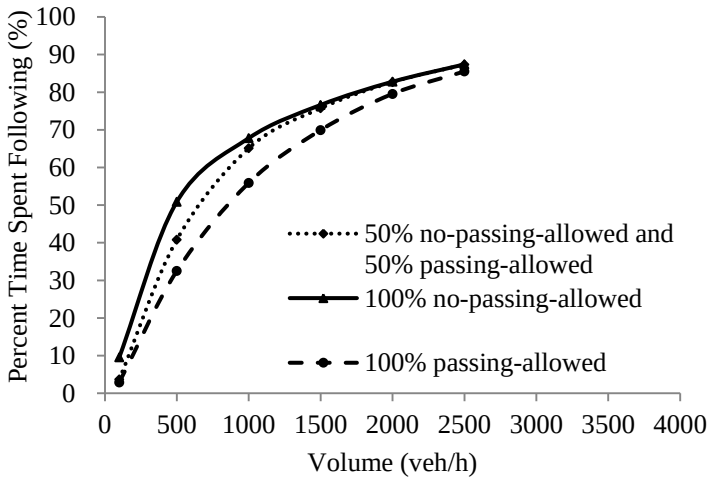


Figure 3-8. Directional *PTSF* results for basic segment tests. A) EB *PTSF*, 50/50 Volume Split B) WB *PTSF*, 50/50 Volume Split C) EB *PTSF*, 60/40 Volume Split D) WB *PTSF*, 40/60 Volume Split E) EB *PTSF*, 70/30 Volume Split F) WB *PTSF*, 30/70 Volume Split





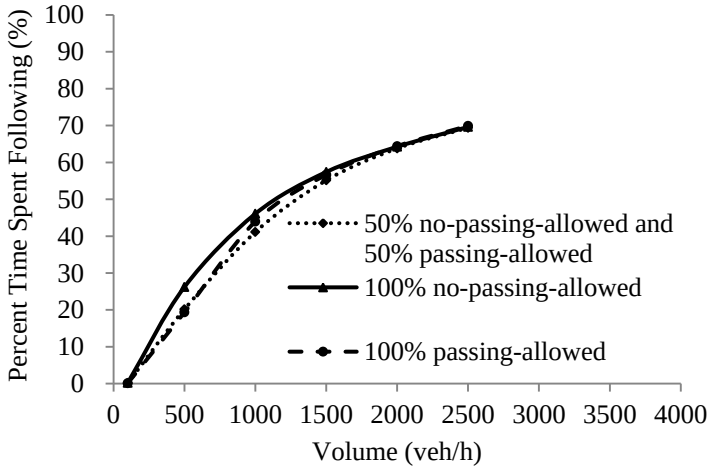


Figure 3-8. Continued

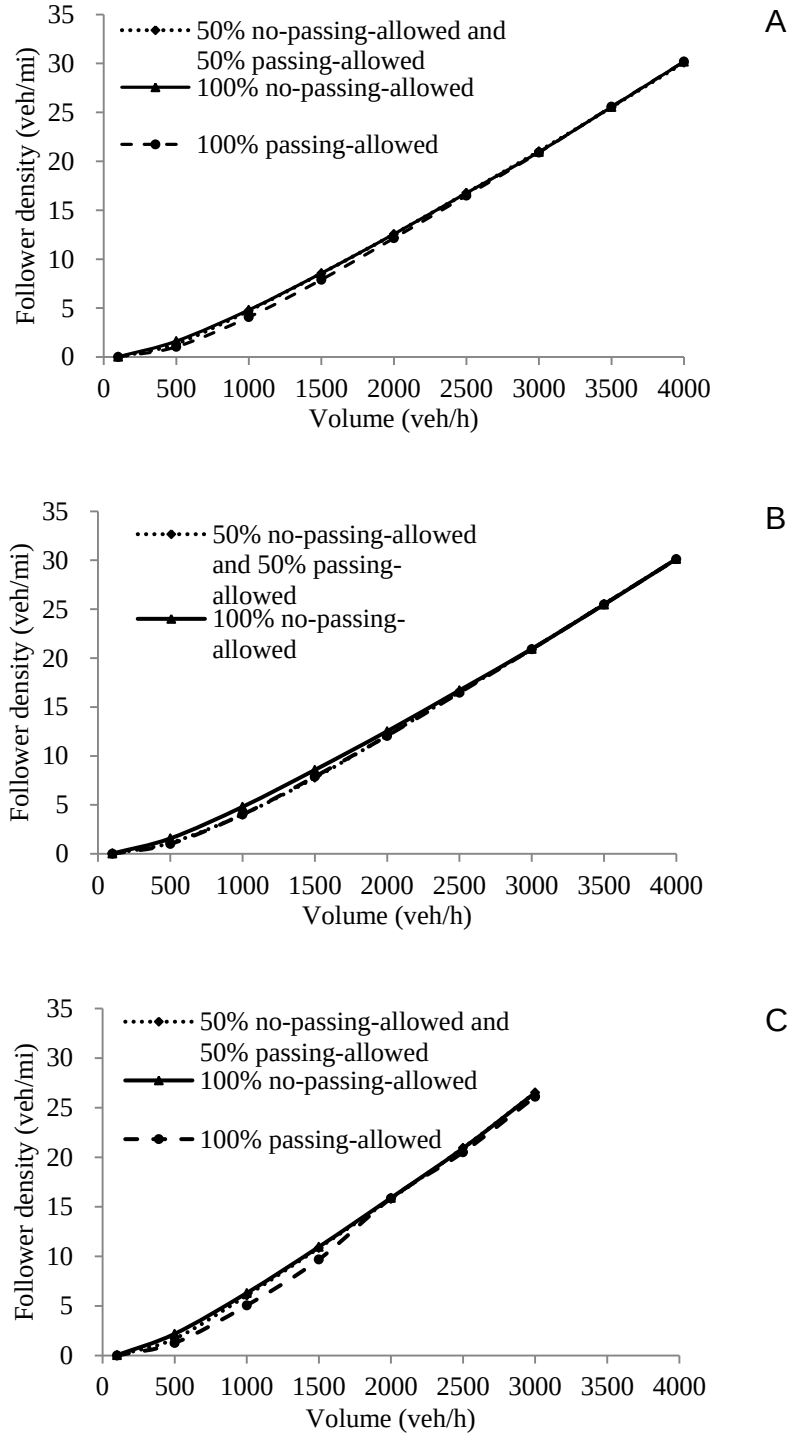
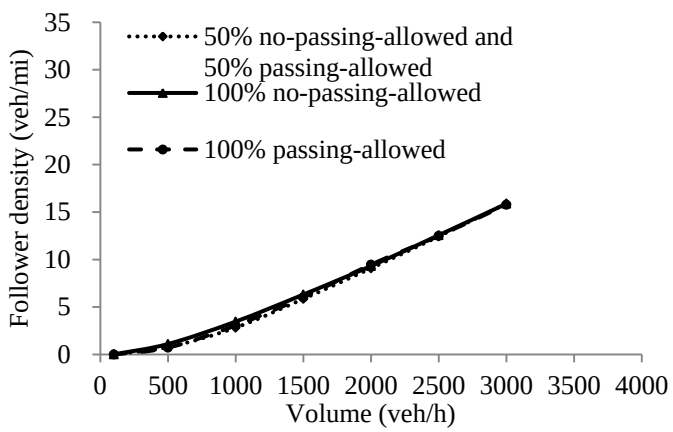
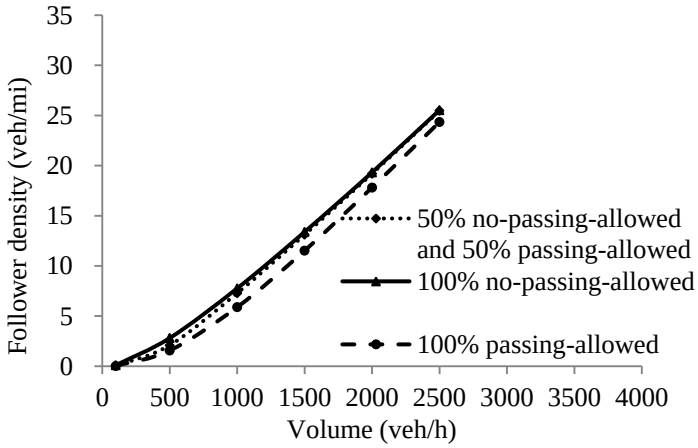


Figure 3-9. Directional follower density results for basic segment tests. A) EB follower density, 50/50 Volume Split B) WB follower density, 50/50 Volume Split C) EB follower density, 60/40 Volume Split D) WB follower density, 40/60 Volume Split E) EB follower density, 70/30 Volume Split F) WB follower density, 30/70 Volume Split





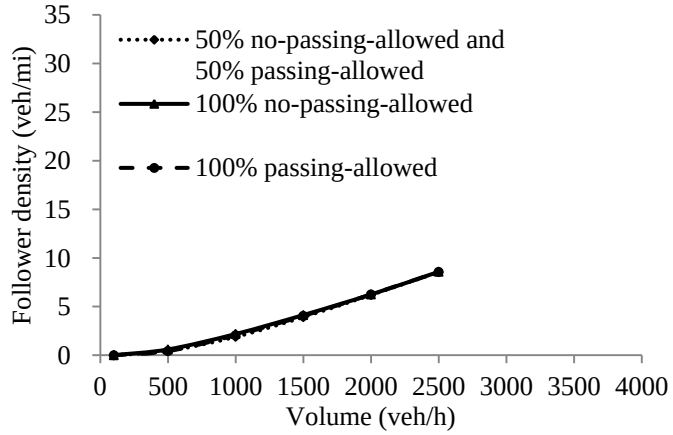
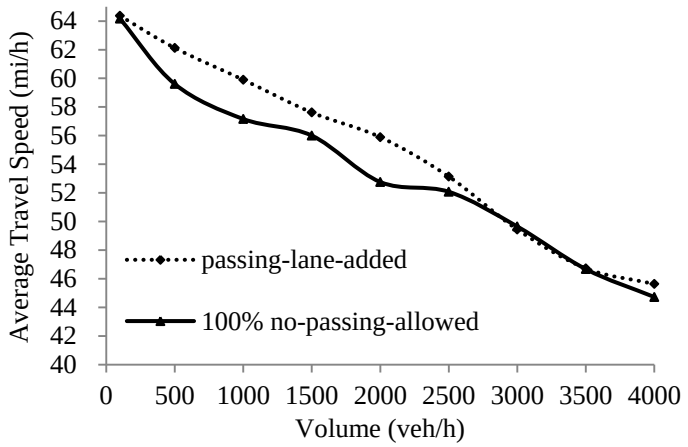
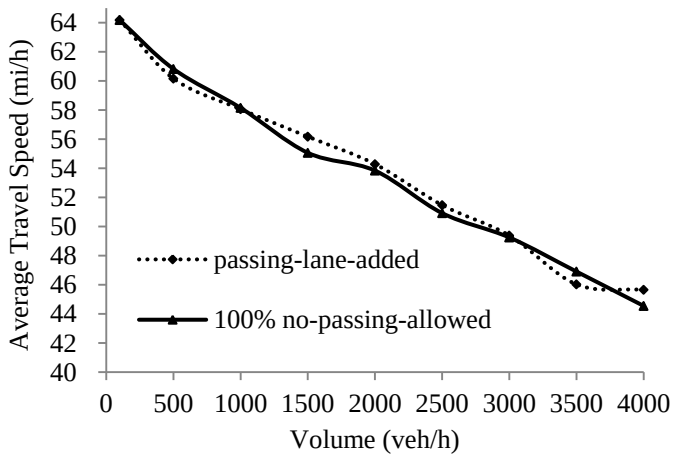


Figure 3-9. Continued





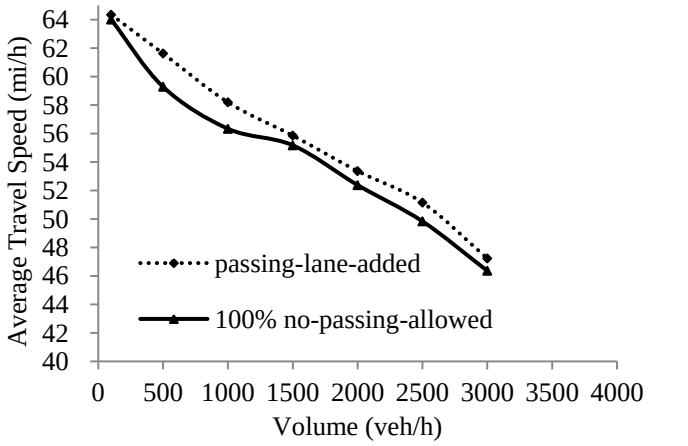
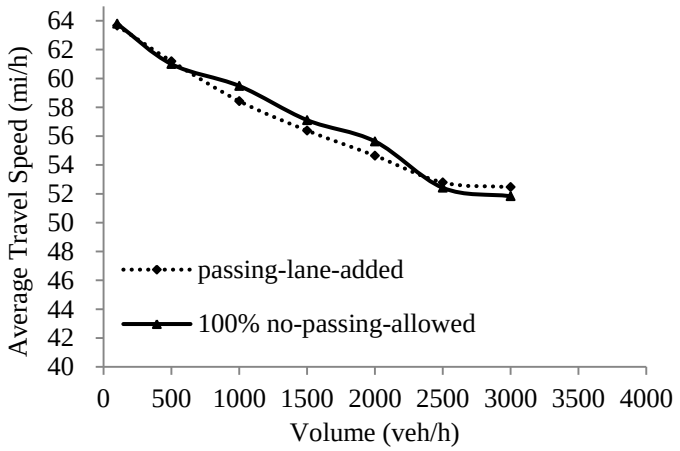


Figure 3-10. Directional average speed results for tests with a passing lane. A) EB Avg Speed, 50/50 Volume Split B) WB Avg Speed, 50/50 Volume Split C) EB Avg Speed, 60/40 Volume Split D) WB Avg Speed, 40/60 Volume Split E) EB Avg Speed, 70/30 Volume Split F) WB Avg Speed, 30/70 Volume Split



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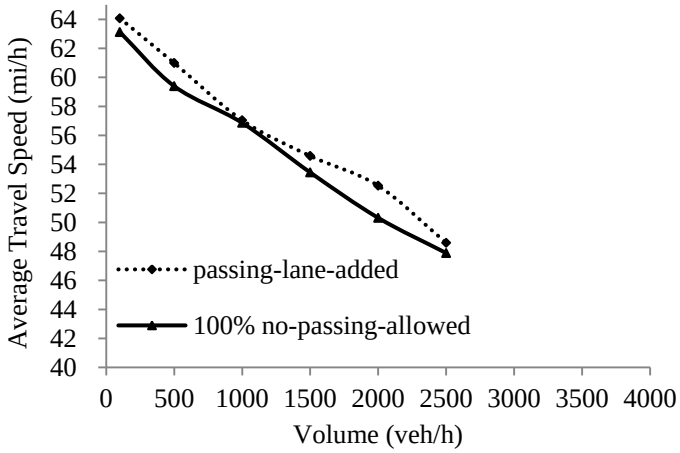

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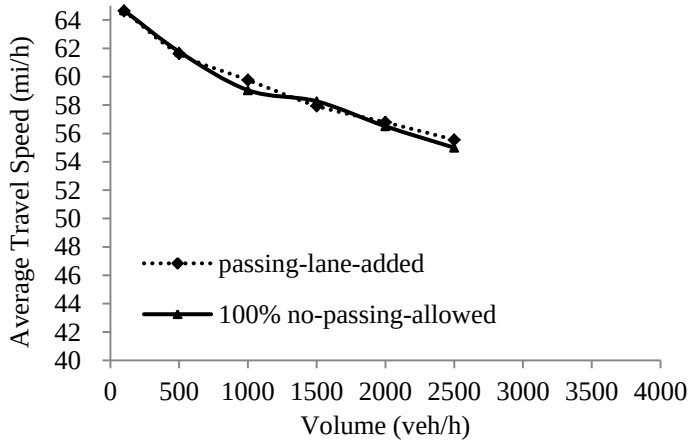


F

Figure 3-10. Continued

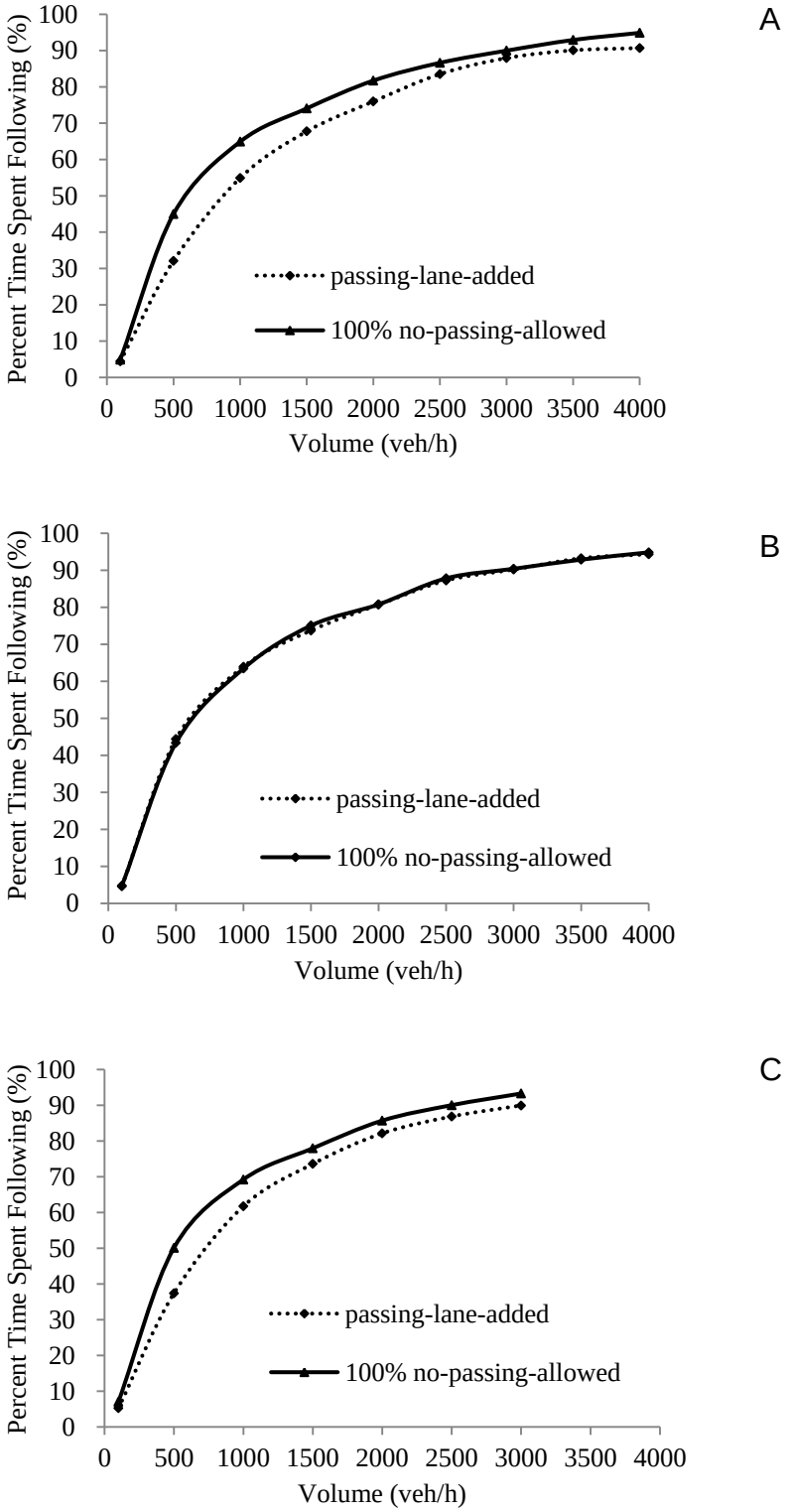
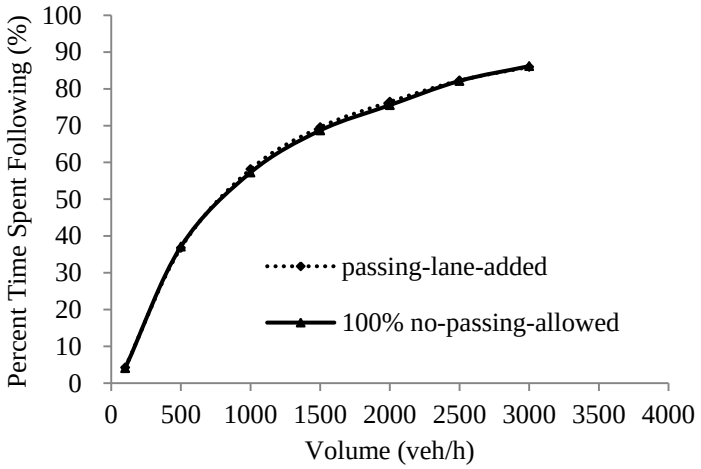
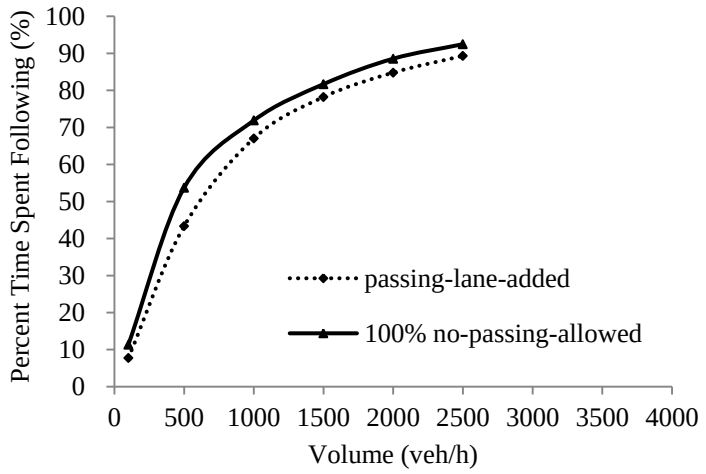


Figure 3-11. Directional *PTSF* results for tests with a passing lane. A) EB *PTSF*, 50/50 Volume Split B) WB *PTSF*, 50/50 Volume Split C) EB *PTSF*, 60/40 Volume Split D) WB *PTSF*, 40/60 Volume Split E) EB *PTSF*, 70/30 Volume Split F) WB *PTSF*, 30/70 Volume Split





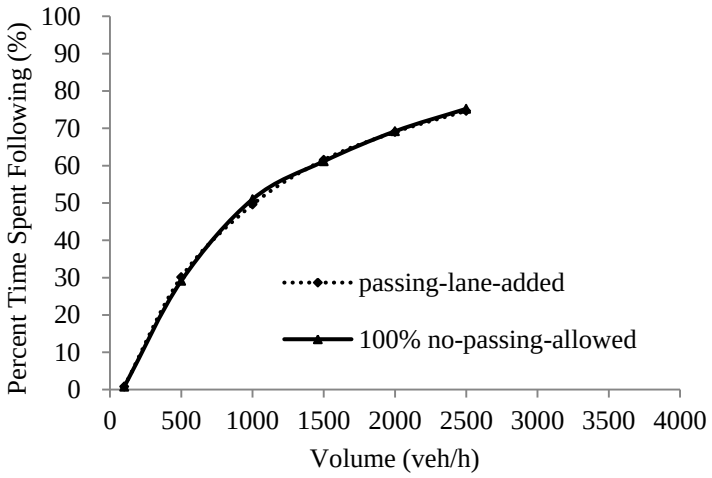


Figure 3-11. Continued

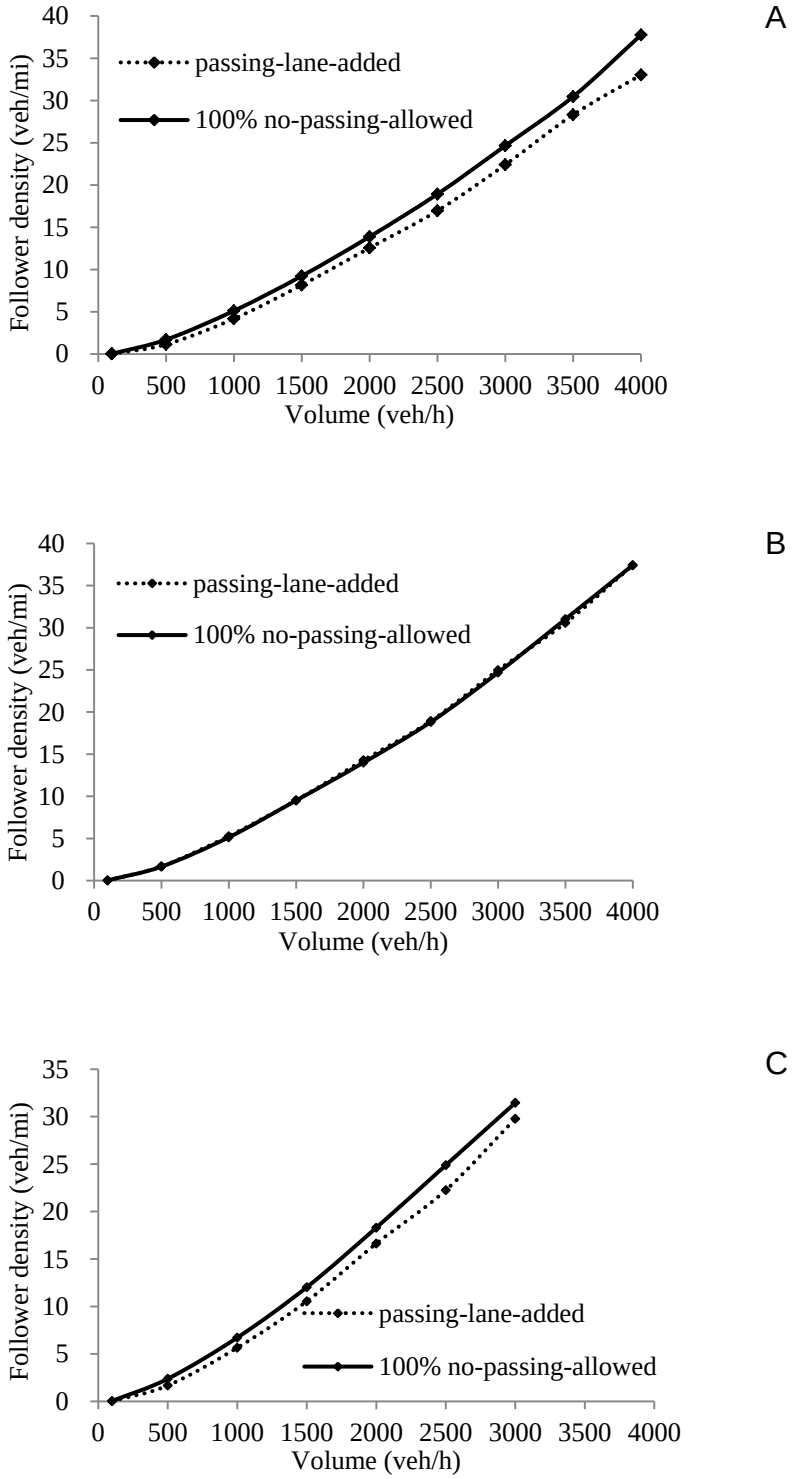
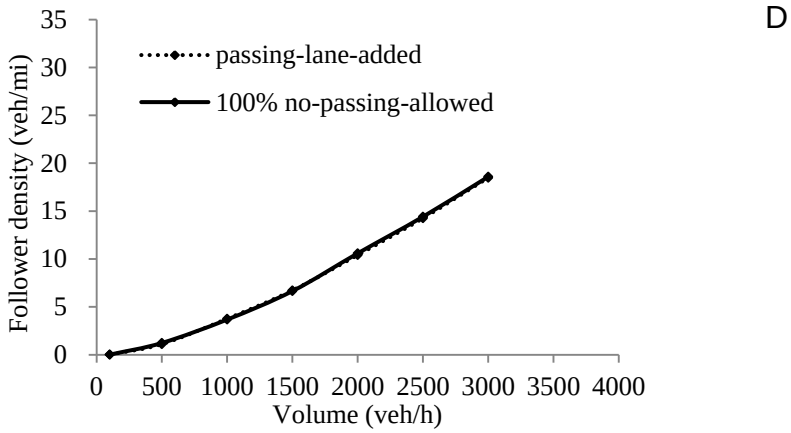
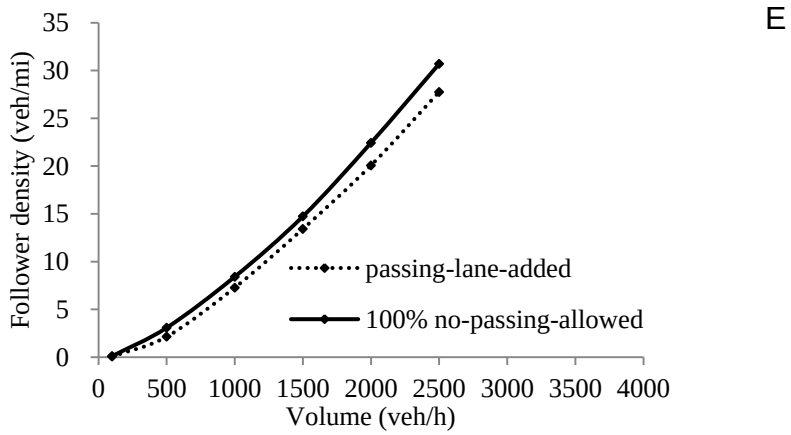


Figure 3-12. Directional follower density results for tests with a passing lane. A) EB follwer density, 50/50 Volume Split B) WB follwer density, 50/50 Volume Split C) EB follwer density, 60/40 Volume Split D) WB follwer density, 40/60 Volume Split E) EB follwer density, 70/30 Volume Split F) WB follwer density, 30/70 Volume Split





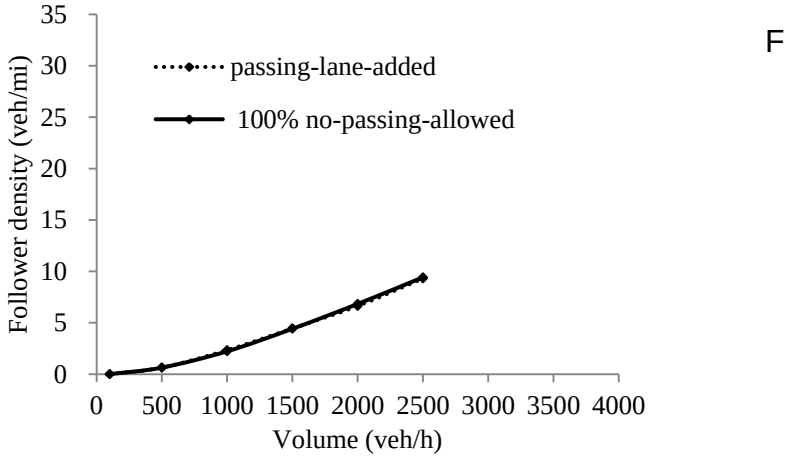


Figure 3-12. Continued

CHAPTER 4
DEVELOPMENT OF AN ANALYTICAL METHODOLOGY FOR TWO-LANE HIGHWAY FACILITY ANALYSIS

The new simulation tool developed, as described in Chapter 3, was used to develop an analytical methodology for analyzing two-lane highway facilities with various features, especially two-lane highway segments with signalized intersections. This methodology is generally consistent with the HCM methodologies for basic two-lane highway segments and signalized intersections.

4.1 Outline of Methodology

The new methodology used the Yu and Washburn study (2009) as the starting point. The main idea of this methodology is to divide a two-lane highway facility into appropriate segments with uniform features, then evaluate traffic operations on each segment by the unified performance measures, and finally aggregate performance measures over the entire facility and obtain LOS accordingly. Therefore, one of the key issues in this study is to properly segment a two-lane highway facility combined with signalized intersections. To achieve this objective, two primary tasks were completed as follows:

Determine upstream effective length of signalized intersection. The presence of a signalized intersection can significantly impact two-lane highway operations. Vehicles that are approaching the signalized intersection will start to decelerate and prepare to stop if facing red, or decelerate to follow a discharging queue during green. This causes interruptions to traffic flow. In situations where a left-turn bay is not present on the intersection approach, main line traffic may also be affected by turning vehicles even during green time. Therefore, it is essential to know the location where such

effects to main line traffic become significant. The distance between this critical location and the intersection approach stop bar is defined as upstream effective length.

Determine downstream effective length of signalized intersection. The presence of signalized intersections may also change the downstream traffic flow characteristics of a signal. In addition, downstream traffic flow will be affected by the right-turn and left-turn vehicles from minor streets. Traffic flow will generally return to its former state after some distance downstream of the signal. The location at which traffic flow returns to “equilibrium” is identified as the end of the downstream effective length.

Another key issue is to select appropriate service measure(s) for evaluating the traffic operations on two-lane highways with various features. *ATS*, *PTSF* and *PFFS* are employed in the current HCM methodology (TRB, 2010) for determining the level of service on basic two-lane highways. However, as control delay serves as the service measure for evaluating signalized intersections, those three service measures cannot be applied to complex two-lane highway facilities. The potential service measure percent delay was examined for its efficacy of representing two-lane highway operations, which is also described in this Chapter.

4.2 Simulation Experiments

In the Yu and Washburn (2009) study, CORSIM was used to develop the upstream effective length model, while TWOPAS was employed to derive the downstream effective length model. As the capability of modeling two-lane highway segments has now been incorporated into CORSIM, a complete two-lane highway facility with a signalized intersection can be set up in CORSIM for data collection.

4.2.1 Testing Facility

The experimental design primarily focuses on two-lane highway segments in combination with signalized intersections. Therefore, a two-lane highway facility including an intersection is analyzed.

The testing facility is 8-mi long, on level terrain, with no passing lane present, and with a signalized intersection at the 4-mi point. The free-flow speed is 60 mi/h on the major two-lane highway segment, and 45 mi/h on the intersecting road. Passing is allowed along the facility except for the NETSIM links that are used for modeling the signalized intersection. It should be noted that the presence of a left-turn bay on the signalized intersection approach could result in some difference in upstream effective length, compared with the situation of a no left-turn bay. When the through flow rate in the opposing direction is relatively high and no protected left-turn phase is provided, the availability of a left-turn bay can reduce the impedance to mainline flow caused by left-turn vehicles that are waiting for acceptable turning gaps. Assuming the storage of the left-turn bay is sufficient for the left-turn demand, the flow rate in the opposing direction will have little effect on the through movement in the analysis direction. Thus, it is not necessary to consider the opposing flow rate in the model for upstream effective length for the situation of a left-turn bay on the intersection approach. Based on such considerations, the models for estimating upstream effective length were developed separately for both conditions, with and without a left-turn bay. According to the guidelines provided by the Delaware DOT (2009) defining no-passing zone extension for departure legs at an intersection, 200-ft NETSIM links were used in modeling the signalized intersection for the main line. The minimum passing sight distance requirement used in passing models developed in Chapter 3 prevent passing

maneuvers happening near the intersection even with the only 200-ft long no-passing markings upstream of the signal. The observations of the simulation animations also confirmed that the design of 200-ft NETSIM links are reasonable.

A 5-mi long lead-up segment is included for obtaining well-developed platoons that will enter the facility where the data were collected. And in order to prevent passing vehicles aborting their passes or expediting their passes due to the end of a passing zone, a 5-mi long follow-up segment is used. The follow-up segment also serves as the lead-up segment for the opposite direction. Passing is allowed on both the lead-up segments and the follow-up segments. However, the outputs on these segments were excluded in the following analysis.

4.2.2 Simulation Scenarios

It has been discussed and identified in the Yu and Washburn study (2009) that the key factors that may affect the length of upstream effective length are peak volume, D-factor, percentages of left-turn and right-turn movements, effective green time, cycle length and availability of left-turn bay. Therefore, the simulation scenarios established in this study were developed based on the above contributing factors.

Factor 1: Peak volume. Peak hour traffic is primarily used in traffic analysis as it represents the worst case for traffic operations and highest capacity demands (TRB, 2010). In this study, 15-min peak flow rates were used in the simulation runs. Although two-lane highways usually serve relatively low traffic demands, a wide range of flow rates are included in the simulation experiments to fully capture the relationship between flow rate and upstream/downstream effective length. The minor street flow rate was set to half of the major street flow rate in all the experiments.

Factor 2: Heavy vehicle percentage. Heavy vehicles such as trucks, buses and recreational vehicles are generally large, and have lower performance in braking and accelerating, especially on grades. The existence of heavy vehicles in the traffic stream can significantly impact traffic operations. In the passing models developed in Chapter 3, the size of the leading vehicle may affect its following vehicle's desire to pass. Therefore, different options for heavy vehicle percentages are preferred in the simulation experiments. CORSIM currently employs nine vehicle types in FRESIM (where the two-lane highway modeling capability was incorporated), four of which represent trucks: Type 3, Type 4, Type 5, and Type 6 (Table 4-1). As Type 6 trucks (i.e., double-bottom trailer trucks) have much lower desired free flow speeds than other types of trucks, even on level terrain, only Type 3, Type 4 and Type 5 trucks were included in the experiments. It should be noted that the NETSIM component uses a different numbering system for vehicle types (Table 4-1). The vehicle type number of a specific vehicle on a two-lane highway segment (FRESIM link) will change into the corresponding vehicle type number once it moves onto a NETSIM link, and vice versa.

Factor 3: D-factor. The D-factor represents the proportion of traffic traveling in the peak direction. When a left-turn bay is absent in the analysis direction, the traffic in the opposing direction may restrict left-turning opportunities in the analysis direction, and as a result, increase the possibility of upstream queuing. Given this consideration, the D-factor is expected to be considered in the regression model for estimating upstream effective length if no left-turn bay is provided.

Factor 4: Percentages of left-turn and right-turn movements. Different turning percentage options were applied in the simulation experiments, as they are also an

important traffic characteristic. Even with the presence of a turn bay, the lane changing maneuver or decelerating maneuver executed by turning vehicles may also influence traffic through movement. In order to keep the conservation of main line flow rate, the equivalent amount of turning flows from minor road were used to compensate for the flows left from main line at the intersection.

Factor 5: Signal timing. For two-lane highway facilities, it is preferred to keep main line traffic moving without interruption for as long as possible. Therefore, a fully-actuated signal timing plan is more appropriate and efficient for a signalized intersection on two-lane highways. Three two-phase signal timing plans were implemented in the experiments to obtain different average effective green time and average cycle length, which were used for the regression model development.

Factor 6: Availability of left-turn bay. The models for upstream effective length estimation were developed separately for the situation with a left-turn bay and the one without a left-turn bay. As discussed before, the D-factor was added as a variable in the regression model for the situation without a left-turn bay to capture the impacts on upstream queuing by opposing flow rate.

Based on the combination of different inputs, a total of 90 scenarios with a left-turn bay and 216 scenarios without a left-turn bay were established in CORSIM, and 10 iterations of each scenario were run. The simulation time period is 15 minutes for each run. The details of the experimental design are listed in Tables 4-2 to 4-4. It should be noted that in the scenarios without a left-turn bay, the two-way flow rate of 2000 veh/h (50/50 directional split) on the major road will result in continuous queue growth. It was identified from the CORSIM experiments that growing queues occur when both

directional flow rates reach 900 veh/h. Therefore, in order to avoid unrealistic results, the highest flow rate option of 1000 veh/h for the analysis direction was eliminated from the experiments without a left-turn bay.

4.3 Vehicle Trajectory Based Data Processing Procedure

In the previous methodology developed by Yu and Washburn (2009), the variation of link-based average speed was used to determine upstream/downstream effective length. For the upstream effective length component, a 3-mi long facility with a signalized intersection at the 1-mi location in CORSIM was used to produce the simulation data. Then, the upstream section was divided into 132-ft[8] long links for identifying the variation of average speed versus distance. For the downstream effective length component, the two-lane highway simulation program TWOPAS was employed to obtain the simulation data. To replicate the traffic platooning characteristics downstream the signal, the TWOPAS input Entering Percent Following (*EPF*) was estimated based on Dixon et al.'s (2003) methodology. Next, the variation of average speed was used to determine downstream effective length. This hybrid simulation approach was applied because no simulation tool that is able to model both signalized intersection and two-lane highway segment existed at that time.

In this study, using the two-lane highway modeling capability now incorporated into CORSIM, the previous models were expected to be validated by using the new simulation program. And for more accurate results, vehicle trajectory information is desired. The rules for determining the vehicles that are affected by the presence of a

[8]This distance was determined based on trying to segment to a short distance, in order to obtain the upstream effective length as accurate as possible. Meanwhile, this distance cannot be so short such that a vehicle could pass completely over a link in one time step (1 second).

signal were added to a tool named VTAPE (Vehicle Trajectory Analysis for Performance Evaluation). VTAPE can process CORSIM TS0 files, which contain time step data for each simulated vehicle. More detail about the rules implemented is included in the following sections.

4.3.1 Determine Upstream Effective Length

In order to obtain the average result of upstream effective length in a certain scenario, it is important to find out where each affected vehicle begins to decelerate significantly due to the presence of a downstream intersection. Based on the vehicle trajectory information, two types of upstream affected vehicles were identified.

Type 1 upstream affected vehicle. This type of affected vehicle is the first to stop at the stop bar. It usually starts to decelerate for the yellow or red as it is getting close to the intersection. Normally, decelerating maneuvers under such a situation are completed within 2 to 4 seconds (Figure 4-1A for example).

Type 2 upstream affected vehicle. This type of affected vehicle joins the queue at the stop bar. When there is already a queue at the stop bar, either stopped during red time or has not been discharged completely during green time, the upcoming vehicles will be aware of the existence of the queue and prepare to decelerate earlier than the type 1 upstream affected vehicle. Under CORSIM's modeling logic, this type of affected vehicle usually starts to decelerate at no less than –4 ft/s^2 for several consecutive seconds (Figure 4-1B for example).

Therefore, the criteria for locating an upstream vehicle's critical position (i.e., the position where a vehicle starts to be affected by the signal) include two requirements:

- The vehicle must be no more than 2000 ft upstream of a signalized intersection. The threshold of 2000 ft was determined based on the longest upstream effective

length of around 1700 ft from the current experiments. The decelerating operations related to passing maneuvers are very unlikely to happen within this distance.

- The vehicle must decelerate at no less than -4 ft/s^2 at the current time step and the previous time step, and then the position at the previous time step is recorded as the critical position.

A number of observations of the simulation animations confirmed the effectiveness of the criteria.

In each scenario, the critical position for each upstream affected vehicle was collected. The average value was calculated from all of the individual vehicle values and used for the overall upstream effective length.

4.3.2 Determine Downstream Effective Length

Determining the downstream effective length is largely a function of the definition of when a vehicle that has departed from an upstream signal queue returns to normal two-lane highway operation. In the Yu and Washburn study (2009), the average speed difference (based on 100-ft long links) between the situation with an intersection and the situation without an intersection was used to determine downstream effective length. The location where the average speed difference falls below a certain threshold is considered as the end of the downstream effective length. However in this study, the downstream effective length is determined from an individual vehicle perspective for more accuracy. Extensive research on the vehicle trajectory data indicates that, a downstream vehicle can be considered as having left the downstream effective area when it travels stably. Two stable situations may exist: 1) If a vehicle is not in a following mode, it is considered as traveling stably when it reaches its own desired free-flow speed and stops accelerating for several successive seconds; 2) If a vehicle is in a following mode, it should at least reach the desired free-flow speed for driver type 1 and

stop accelerating for several successive seconds as an indicator for being stable. Two types of downstream affected vehicles were identified according to the vehicle trajectory information.

Type 1 downstream affected vehicle. This type of affected vehicle is the one that has completely stopped for the red before traveling downstream of the signal. It could be a vehicle from upstream flow or a turning vehicle from the intersecting road. This type of vehicle usually accelerates from zero speed when being discharged (Figure 4-1A for example).

Type 2 downstream affected vehicle. This type of affected vehicle may join a discharging queue at the intersection during the green, or turn from the intersecting road without stopping at the intersection. In this case, the vehicle normally accelerates from a relatively lower speed after entering the downstream segment (Figure 4-1B for example).

It should be noted that the maximum acceleration for Type 5 trucks in CORSIM is 1 ft/s^2. Therefore, when heavy vehicles are involved in a scenario, the average downstream effective length is extended.

The criteria for locating the end of the downstream effective length for each downstream affected vehicle also include two requirements:

- The vehicle must be delayed by an intersection. If a vehicle's speed is lower than the driver type 1's free-flow speed when it first gets onto the downstream link, it is considered as an intersection-delayed vehicle. Unaffected vehicles with driver types other than 1 normally travel at their own desired free-flow speeds, or at driver type 1's free-flow speed when in a platoon led by a drive type 1 vehicle. All turning vehicles are considered as downstream affected vehicles, as they always decelerate to a speed less than driver type 1's desired free-flow speed for the turning movement, even for the vehicles with the most aggressive driver type.

- The vehicle must reach the free-flow speed for driver type 1, and travel constantly or decelerate within two consecutive seconds. Based on the examination of a considerable amount of vehicular data, it was found that a vehicle may not reach a stable level if it stops accelerating for just 1 second. And a vehicle that has become stable may sometimes decelerate a little bit to maintain a safe headway when in a platoon. Within any consecutive 3 seconds after being stable for a vehicle, if its acceleration is zero or negative in any 2 seconds, it may have a positive acceleration for the third second. Therefore, 2 seconds for checking a vehicle's acceleration is appropriate for determining whether it has become stable or not. The position at the first time step of the two consecutive time steps is recorded as the ending position of the downstream effective length.

The ending position of the downstream effective length for each downstream affected vehicle was then determined based on the above criteria. For each scenario, the average value was calculated to obtain the overall downstream effective length.

4.4 Model Development

The development of mathematical models for estimating upstream and downstream effective length was performed through regression analysis, and is discussed in this section. For each scenario, the upstream/downstream effective lengths for all the vehicles were averaged over 10 runs. The aggregate results were used in the regression model development.

4.4.1 Upstream Effective Length Model Development

The regression model for estimating upstream effective length was expected to be similar to the form developed by Yu and Washburn (2009), as follows:

$$Y = \alpha + \sum_l \sum_m \beta_{lm} X_l^k X_m^q \tag{4-1}$$

where Y is the dependent variable, α is the intercept, β_{lm}, k, and q are parameters, and X_l and X_m are the independent variables.

The model for estimating the upstream effective length of a signal when left-turn bay is present was developed based on 90 scenarios, as follows:

$$Len_{eff\_up} = 266.66 + 3.047 \times \left(\frac{v_d}{100}\right)^2 + 8.626 \times Cycle - 0.972 \times \left(\frac{v_d}{100}\right) \times \%LT - 14.102 \times g \quad (4\text{-}2)$$

where

Len_{eff_up} = upstream effective length of signalized intersection (ft)

v_d = flow rate in the direction analyzed (pc/h)[9]

$Cycle$ = average cycle length (s)

$\%LT$ = percentage of left-turn vehicles in the direction analyzed

g = effective green time

The statistical results are shown in Table 4-5. From Table 4-5, it can be seen that all the explanatory variables are statistically significant at a 95% confidence level. The coefficient signs are logical; for example, increasing flow rate in the analysis direction or extending cycle length will increase the upstream effective length, increasing the percentage of left-turn vehicles or the effective green time will decrease the upstream effective length. With an adjusted *R*-squared value of 0.9798, it is indicated that 97.98% of the variance in the dependent variable (upstream effective length) was explained by variations in the independent variables. The well-behaved model residuals, illustrated in Figure 4-2, confirm the goodness-of-fit of the model as well. The relationships between the explanatory variables and the response are generally consistent with the previous model.

The model for determining upstream effective length under the situation of no left-turn bay takes the D-factor into consideration. However, the D-factor is not directly used

[9]In this study, the passenger car equivalent factors used to convert mixed vehicle flows into an equivalent passenger car flows are 2.2 for Type 3 trucks, 2.5 for Type 4 trucks, and 2.8 for Type 5 trucks.

in the model, but reflected in the variable of opposing flow rate. The model was developed based on 216 scenarios, as follows:

$$Len_{eff\_up} = 412.02 + 57.997 \times \left(\frac{v_d}{500}\right)^3 + 85.158 \times \left(\frac{v_o}{500}\right)^3 - 3.656 \times Cycle + 0.033 \times \left[\left(\frac{v_d}{500}\right) \times \%LT\right]^3 \quad (4\text{-}3)$$

where

Len_{eff_up} = upstream effective length of signalized intersection (ft)

v_d = flow rate in the direction analyzed (pc/h)

v_o = flow rate in the opposing direction (pc/h)

$Cycle$ = average cycle length (s)

$\%LT$ = percentage of left-turn vehicles in the direction analyzed

The statistical results are listed in Table 4-6. It can be seen from Table 4-6 that all the explanatory variables are statistically significant at a 95% confidence level. A new variable describing the opposing flow rate is included in this model. It should be noted that effective green time is not included in this model. Intuitively, one would consider this variable to have a significant effect; however, since the actuated signal timing parameters were set up to provide essentially “optimal” operation, the cycle length variable alone is sufficient to also account for the effect of green time. The analyst should be aware that if they are analyzing a situation where the signal timing is not optimized for the traffic flow conditions, the results from the upstream effective length model may not be completely accurate. The coefficient sign of the variable describing opposing flow rate is reasonable, because increasing the opposing flow rate will reduce the turning opportunities for the left-turn vehicles in the analysis direction, leading to the

increment in the upstream effective length when no left-turn bay is provided. The coefficient signs for the variables cycle length and percentage of left-turn vehicles are contrary to the ones in the model for the situation with a left-turn bay. For the variable cycle length, although a longer cycle length is usually considered as the cause of increased delay and queues, it may offer more turning possibilities for waiting left-turn vehicles, and consequently allow the following through vehicles to pass through the intersection. This positive influence may offset the negative influence created by longer cycle lengths in other aspects. In this way, the inverse relationship between upstream effective length and average cycle length could be explained. For the variable percentage of left-turn vehicles, it is logical that under the situation without a left-turn bay, increasing the proportion of left-turn vehicles will increase the upstream effective length, because left-turn vehicles have to yield the opposing through traffic when no protected left-turn phase is provided.

With the adjusted *R*-squared value of 0.9387, it is indicated that 93.87% of the variance in the dependent variable (upstream effective length) was explained by variations in the independent variables. The well-behaved model residuals, illustrated in Figure 4-3, confirm the goodness-of-fit of the model as well.

It should be noted that in some very rare cases where no vehicles turn left from the mainline (e.g., left turn is prohibited), it is appropriate to apply the model for the with left-turn bay situation to estimate the upstream effective length even no left-turn bay is provided, because it is no longer realistic to have a penalty for opposing flow in such situations to determine the upstream effective length.

A comparison was made between the two upstream effective length models (i.e., with and without a left-turn bay) using the same combination of inputs. The results confirm the expectation that the no left-turn bay scenario would always result in a longer upstream effective length than the with left-turn bay scenario (Figure 4-4).

4.4.2 Downstream Effective Length Model Development

The regression model for estimating downstream effective length is in the similar form described in Equation 4-1. Initially, models were developed separately for the scenarios with a left-turn bay and the scenarios without a left-turn bay, which is similar to the upstream effective length model development. However, the experiment results showed that an upstream left-turn bay has little impact on the downstream effective length. Therefore, the simulation data from both facility conditions were combined and used to develop one model for estimating downstream effective length.

The model for estimating downstream effective length of signal was developed based on 306 scenarios, as follows:

$$Len_{eff\_down} = 701.34 + 51.016 \times \left(\frac{V_d}{100}\right) + 42.353 \times \%HV + 13.833 \times Cycle - 1.701 \times \left(\frac{V_d}{100}\right) \times \%LT - 16.760 \times g \quad (4\text{-}4)$$

where

Len_{eff_down} = downstream effective length of signalized intersection (ft)

V_d = flow rate in the direction analyzed (veh/h)

$\%HV$ = percentage of heavy vehicles

$Cycle$ = average cycle length (s)

$\%LT$ = percentage of left-turn vehicles in the direction analyzed

g = effective green time

The statistical results are listed in Table 4-7. From Table 4-7, it can be seen that all the explanatory variables are statistically significant at a 95% confidence level. With an adjusted *R*-squared value of 0.9014, it is indicated that 90.14% of the variance in the dependent variable (downstream effective length) was explained by variations in the independent variables. The well-behaved model residuals, illustrated in Figure 4-5, confirm the goodness-of-fit of the model as well.

The relationships between the explanatory variables and the response are as expected. The positive relationship between downstream effective length and flow rate can be explained in that increasing flow rate results in a longer average queue length upstream of the signal. The longer an upstream queue is, the more time is required for the queued vehicles to accelerate and disperse, which eventually leads to an increasing downstream effective length. A similar reason can also be used in explaining the positive relationship between cycle length and downstream effective length. The existence of heavy vehicles in the traffic stream is another reason for the increment of downstream effective length, because of the relatively lower acceleration capability of heavy vehicles. In contrast, an increasing number of left-turn vehicles can reduce downstream effective length. Within the current experiments, the turning flow rate from the intersecting roads are as the same as the mainline turning flow rate in order to keep the conservation of mainline flow. However, as they are discharged during different phases, the average number of queued vehicles entering the mainline downstream segment actually decreases. Effective green time for the mainline through movement

also plays an inverse role in downstream effective length. Given a certain cycle length, a more effective green time means a reduced chance of queue build-up during a cycle. The factor opposing flow rate was also examined as an explanatory variable, as initially it was thought that this may affect the acceleration and dispersion of upstream queues. However, as was the case for the left-turn bay variable, this variable was found to have very little effect on the model results. Therefore, this variable was eliminated from the final model.

4.5 Examination of the Efficacy of Percent Delay as an Appropriate Service Measure for Two-Lane Highway Facilities

Yu and Washburn (2009) proposed Percent Delay (*PD*) as the primary service measure for a two-lane highway with a signalized intersection. *PD* is believed to be able to represent a driver's perception of freedom during driving. At the same time, as control delay is the only service measure that determines the level of service of a signalized intersection, it is convenient to transfer control delay to total travel delay to determine the overall *PD* of a complex two-lane highway facility. *PD* has been incorporated into CORSIM as an output for two-lane highway links (Table 3-4).

In the current HCM (TRB, 2010) two-lane highway methodology, two-lane highways are categorized into three classes based on different driver's expectation during driving. Class I two-lane highways usually serve to connect major traffic generators (e.g., cities, states), on which drivers expect to travel at high speeds and with more comfort. Thus, both *ATS* and *PTSF* are included in the Class I LOS criteria. Only *PTSF* is used in determining the LOS of Class II two-lane highways (e.g., shorter intra-city routes), on which high speed is not of great importance to drivers. Class III two-lane highways is a new class in the HCM methodology. Two-lane highways

traversing developed areas, or along scenic areas, are classified as Class III highways. Given that the speed limit typically reduces due to the relatively higher activity level within developed areas, the percent of free-flow speed is more meaningful than the absolute average travel speed in the determination of LOS. Therefore, *PFFS* serves as the service measure for Class III two-lane highways.

Follower density has been identified as an effective service measure for the evaluations of two-lane highway operations in previous studies: 1) follower density reflects the effects of flow rate, speed and percentage of followers (Van as, 2006); 2) follower density reflects that flow rate is the major factor that affects the performance of highways (Al-Kaisy & Durbin, 2008); and 3) follower density has a wide range of variation (Oregon Department of Transportation, 2010). Hence it is worth considering follower density is also suitable for a two-lane highway facility including signalized intersections. CORSIM can now also produce follower density as an output for two-lane highway links.

All the service measures discussed above were tested with the same two-lane highway facility in section 4.2.1 (the one with a left-turn bay). Only passenger cars were included in the tests, as well as all three signal timing plans (Table 4-4). In order to capture the performance of the facility over the entire range of flow rates, the capacity on the testing facility in the situation with each signal timing plan was estimated. It was found that the capacity ranges from 1408 veh/h to 1478 veh/h in the three different situations. Therefore, two flow rate levels of 1200 veh/h and 1400 veh/h were added to the existing flow rate options. Ten iterations of each experiment were run, and the average simulation results of the ten iterations were obtained and analyzed. As the

volume split is 50/50 in all the experiments, only the eastbound results are presented. As the service measure results only vary slightly among the three signal timing plans, given that all the other inputs remain the same, only the results from the scenarios with the first signal timing plan are presented here. Generally, it is expected that the relationship between a service measure and flow rate is close to a linear relationship and has a wide spectrum over the range of flow rate. A dashed straight line starting from the origin is added in each figure to help evaluate the nonlinearity of each relationship.

***ATS* vs. flow rate.** Figure 4-6 illustrates the relationship between directional *ATS* and directional volume. The results show a similar trend to the one from the basic two-lane highway test results (Figure 3-7A). The difference in the results between the signal timing plans is slight, until the flow rate gets close to the capacity. Signal timing plan III, which has the smallest maximum green time for the mainline, results in the lowest *ATS* at the highest flow rate.

Although a strong inverse relationship exists between *ATS* and flow rate, *ATS* cannot reflect a driver's perception of the performance of a signalized intersection, which is evaluated based on control delay (TRB, 2010). In addition, as speed limit reduction is usually applied in the vicinity of a signalized intersection, the absolute value of *ATS* has little meaning in evaluating such facilities. Therefore, *ATS* is not appropriate for assessing the performance of a two-lane highway facility with signalized intersections.

***PTSF* vs. flow rate.** Figure 4-7 illustrates the relationship between directional *PTSF* and directional volume. The results are similar in trend to those from the basic

two-lane highway test results (Figure 3-8A). The difference in the results between the signal timing plans is not significant. Nevertheless, due to the limited passing opportunities in the proximity of a signalized intersection, this passing related service measure is not necessary for a driver's expectation of travel in the intersection influence area. Thus, *PTSF* does not suit the performance evaluation of a two-lane highway facility with signalized intersections.

***PD* vs. flow rate.** Figure 4-8 illustrates the relationship between directional *PD* and directional volume. Similar to the *ATS* results, the difference in the *PD* results between the signal timing plans is slight until the flow rate approaches the capacity. Signal timing plan III, which has the smallest maximum green time for the mainline, leads to the greatest *PD* at the highest flow rate. As *PD* considers delays experienced both on basic two-lane highway segments and in the influence area of an intersection, it is appropriate for the analysis of complex two-lane highways.

The *PD* grows slowly in the moderate to high flow rate range, which is consistent with the variation of the *ATS* results. However, a *PD* value greater than 18.3 (the peak *PD* value in Figure 4-8) for a two-lane highway facility including signalized intersections does not necessarily mean LOS F. Given the same length of two-lane highway, increasing the intersection density will increase the total travel delay, which as a result increases the *PD* result. For consistency with other LOS methodologies in the HCM, LOS F is not considered to be applicable until the demand exceeds the capacity.

Follower density vs. flow rate. Figure 4-9 illustrates the relationship between directional follower density and directional volume. The results with different signal timing plans have little difference. As mentioned before, follower density reflects the

effects of flow rate, speed and percentage of followers. However, follower density might be more useful for uninterrupted two-lane highway facilities in that it does not describe very well the performance of a signalized intersection from a driver's perspective, and it is not appropriate to be applied to a two-lane highway facility on which speed limit reduces in some areas.

The relationship between follower density and flow rate is closest to a linear relationship according to Figure 4-9. This also confirms the good eligibility of follower density as a service measure for uninterrupted two-lane highways. Further study is desired on this service measure, including field data collection, simulation calibration, and the determination of LOS criteria.

***PFFS* vs. flow rate.** Figure 4-10 illustrates the relationship between directional *PFFS* and directional volume. The *PFFS*-flow curve shares the similar trend with the *ATS*-flow curve (Figure 4-6) because the free-flow speed in the current tests remains the same along the mainline. *PFFS* is the service measure for Class III two-lane highways in the HCM 2010. Although Class III highways represent two-lane highways traversing developed areas, they are still considered uninterrupted highway facilities from the HCM perspective and different from two-lane highway facilities that include signalized intersections. It is possible to extend *PFFS*'s application into the analysis of a two-lane highway facility with a signalized intersection, as the delay incurred in the intersection influence area can be translated into the overall average travel speed, based on which the overall *PFFS* can be determined. If the posted speed limit varies along a two-lane highway facility with intersections, it is more appropriate to consider the weighted average *PFFS* based on different speed limits over the entire facility in that

case. Therefore, extra calculation steps might be involved in an analysis procedure using *PFFS* as a service measure, which might make it somewhat more cumbersome to use than *PD.*

In summary, *PD* is the most appropriate service measure among the five candidate measures presented here for a two-lane highway facility with signalized intersections. Meanwhile, it has been confirmed that follower density can serve as an effective service measure for the assessment of uninterrupted two-lane highway operations.

4.6 Overall Evaluation Methodology

The evaluation methodology uses the same idea as the Yu and Washburn study (2009), as mentioned in section 4.1. The detailed evaluation procedure is described below, followed by the validation of the methodology.

4.6.1 Evaluation Methodology

The vehicle trajectory approach for determining upstream and downstream effective length is in fact consistent with the definition of control delay (Figure 4-11). Therefore, the delay experienced within upstream and downstream effective segments can be fully accounted for by control delay. In this sense, it is rational to divide a two-lane highway facility with signalized intersections into basic two-lane highway segments and intersection influence areas.

The procedure for evaluating the traffic operations on a two-lane highway with a signalized intersection can be divided into seven steps, as follows:

Step 1. Determine the upstream and downstream effective length of the signalized intersection for obtaining the intersection influence area.

Step 2. Segment the facility according to the location of the intersection influence area. The areas other than the intersection influence area are treated as basic two-lane highway segments.

Step 3. Determine the delay on each basic two-lane highway segment by calculating the difference between actual travel time and free-flow travel time.

Step 4. Determine the control delay of the intersection.

Step 5. Sum up the delays calculated in Step 3 and Step 4.

Step 6. Determine the free-flow travel time along the entire facility.

Step 7. Determine the Percent Delay and the corresponding level of service.

The HCM 2010 analysis procedures are recommended to accomplish Step 3 and Step 4.

4.6.2 Validation of the Methodology

To validate the efficacy of this methodology, a two-lane highway facility in combination with two signalized intersections was employed (Figure 4-12) in the tests with three different flow rate levels. The validation analysis procedure is given in the steps below.

Step 1. Given a facility, determine the upstream effective length and downstream effective length, and then divide the facility into appropriate segments.

Step 2. Establish a testing facility in CORSIM based on the segments obtained in Step 1.

Step 3. Run the simulation test to generate the overall Percent Delay (*PD*) for the entire facility, and the control delay for each signalized intersection.

Step 4. Establish a basic two-lane highway facility in CORSIM, with the same inputs as the one in Step 2, except for the signalized intersections.

Step 5. Run the simulation test to obtain *ATS* on the basic two-lane highway facility set up in Step 4.

Step 6. Use the control delays obtained from Step 3 and the *ATS* on the basic two-lane facility from Step 5 to calculate the overall *PD*.

Step 7. Compare the calculated *PD* with the aggregate *PD* produced by CORSIM.

From the comparison results in Table 4-8, it can be seen that the calculated *PD* is lower than the aggregate *PD* in every flow rate level. This difference was caused by the underestimation of control delay in CORSIM. In CORSIM, only the link immediately upstream of the signal has a nonzero value of control delay; thus, the link immediately downstream of the signal always has a control delay of zero. Control delay includes initial deceleration delay, queue move-up time, stopped delay and the final acceleration delay (TRB, 2010). Therefore, control delay also occurs on the link immediately downstream of signal. If the control delay calculated in CORSIM is adjusted to an appropriate value to account for the downstream delay, the aggregate *PD* gets very close to the calculated *PD*.

The comparison was also made between the aggregate *PD* from CORSIM output and the calculated *PD* based upon the HCM calculations for *ATS* and control delay. The difference in the results is caused primarily by the underestimation of the average travel speed on two-lane highways in the HCM 2010 (Figure 4-13[10]). If the average travel speed on a basic two-lane highway segment calculated by the HCM 2010 is adjusted to an appropriate value, the aggregate *PD* becomes very close to the calculated *PD*.

[10]The HCM *ATS* results in Figure 4-13 were calculated by Equation 15-6 in the HCM (TRB, 2010) (refer to Equation 2-1 in this document).

4.6.3 Percent-delay-based LOS criteria

The valid thresholds for the *PD*-based LOS criteria should satisfy two requirements: 1) for an analysis of a two-lane highway facility without signals, analysts should obtain the same LOS if switching from the HCM service measures to *PD*; 2) for an analysis of a two-lane highway facility with a signal, analysts will not obtain a better LOS compared with a no-signal situation. To determine the thresholds for the *PD*-based LOS criteria, the analytical methodology described in section 4.6.1 was used to carry out two sets of calculations. One set of calculations is based on an 8-mi basic two-lane highway facility, and the other set of calculations is based on an 8-mi two-lane highway facility with a signalized intersection at the 4-mi point. The percent delay was calculated at seven different flow rate levels on both testing facilities.

As in the current HCM two-lane highway methodology (TRB, 2010), three different LOS criteria are used for three different two-lane highway classes, and all three LOS criteria were employed as references to determine the *PD* thresholds. The procedure for determining the *PD* thresholds is summarized as follows:

Step 1. Develop the relationships between flow rate and the HCM service measures (i.e., *ATS*, *PTSF*, and *PFFS*) for the two-lane highway facility without signals based on the HCM methodology.

Step 2. Determine the corresponding flow rates at the LOS boundary values for each service measure.

Step 3. Develop the relationship between flow rate and *PD* for the two-lane highway facility without signals.

Step 4. Determine the boundary values for each *PD*-based level of service based on the flow rate thresholds obtained from Step 2.

Figures 4-14 to 4-17 illustrate the thresholds for the *PD*-based LOS criteria based on the HCM two-lane highway LOS criteria. The relationship between flow rate and *PD* for the two-lane highway facility without a signal is also plotted in each figure to confirm that inserting a signal on to a two-lane highway will not result in a better LOS.

As the sets of *PD* thresholds obtained based on the HCM LOS criteria, which are different for each two-lane highway class, were found to be different from each other, the *PD*-based LOS criteria are determined separately for each two-lane highway class. It should be noted that both *PTSF* and *ATS* are employed as the service measure for a Class I two-lane highway. Therefore, two sets of *PD* thresholds based on the Class I *PTSF* criteria and the Class I *ATS* criteria were combined to determine one set of *PD* thresholds. The details of the PD-based LOS criteria are listed in Table 4-9.

4.7 Additional Guidance for Facility Segmentation

It is common in the field that the posted speed limit will be reduced in the vicinity of a signalized intersection on a two-lane highway. Considering the impracticality of speed limit reduction design as an input in a model, the guidance for facility segmentation regarding this factor is discussed in this section.

4.7.1 Segmentation Guidance

When speed limit reduction design is applied in the upstream segment of an intersection, two different situations may exist:

- Vehicles have to decelerate due to a speed limit reduction prior to entering the influence area of a signalized intersection
- Vehicles enter the influence area of a signalized intersection, and are regulated by a speed limit sign indicating the reduced limit.

The CORSIM modeling logic dictates that vehicles usually start to decelerate at –4 ft/s^2 to adjust their speeds under the reduced speed limit requirement. Therefore, using

VTAPE for determining the upstream effective length for each vehicle mentioned in section 4.3.1, it is difficult to distinguish the vehicles decelerating due to speed limit reduction from the vehicles decelerating due to the signal. However, it is still believed that the delay experienced in the influence area of a signalized intersection (combination of upstream effective segment and downstream effective segment) should only come from the control delay.

Based on this understanding, given a two-lane highway facility with a speed limit reduction upstream of the signal, the facility segmentation is performed as follows:

Step 1. Calculate the upstream effective length based on given conditions.

Step 2. Locate the position where the speed limit drops.

Step 3. Check if the position of the speed limit change is further upstream from the intersection than the position where the calculated upstream effective segment begins. If so, divide the unaffected segment into two basic two-lane highway segments with different free-flow speeds (Figure 4-18A). Otherwise, the overlapped part will be considered as a part of the upstream effective segment without special treatment (Figure 4-18B).

4.7.2 Example

An example is presented for illustrating the evaluation procedure when speed limit reduction design is applied upstream of a signal.

The input data for a two-lane highway facility that includes a signalized intersection has the following characteristics:

- Directional flow rate = 800 pc/h (in both directions)
- PHF = 1.00
- 5% left-turns

- 100% passing zones in both directions (except for the vicinity of the intersection)
- Level terrain
- 0% heavy vehicles
- 12-ft lane widths
- 6-ft shoulders
- 0 access points/mi (on basic two-lane highway segments)
- 60 mi/h base free-flow speed on most of the facility , 50 mi/h free-flow speed on the 800-ft link immediately upstream of signal in the analysis direction
- 8-mi segment length, the intersection is located in the middle
- Left-turn bay is provided
- Average cycle length (during 15-min analysis period) = 57.3 s
- Effective green time for major road (during 15-min analysis period) = 33.8 s

Step 1. Determine upstream effective length. Upstream effective length is estimated using Equation 4-2. Then:

$$Len_{eff\_up} = 266.66 + 3.047 \times \left(\frac{v_d}{100}\right)^2 + 8.626 \times Cycle$$
$$- 0.972 \times \left(\frac{v_d}{100}\right) \times \%LT - 14.102 \times gC \times Cycle$$

$$Len_{eff\_up} = 266.66 + 3.047 \times \left(\frac{800}{100}\right)^2 + 8.626 \times 57.3$$
$$- 0.972 \times \left(\frac{800}{100}\right) \times 5 - 14.102 \times 33.8$$
$$= 440.4\,(\text{ft})$$

Step 2. Locate the position where the speed limit drops. Based on the inputs, the speed limit drops 800 ft in advance of the signalized intersection.

Step 3. Check if the speed limit drops prior to where the upstream effective segment begins. Because 800 ft is greater than 440.3 ft, the speed limit drops prior to where the upstream effective segment begins.

Step 4. Subdivide upstream segment. The 4-mi upstream segment is divided into 3 segments with different features. The length of each segment is determined as follows:

L_{eff_up}: upstream effective segment

$$Len_{eff\_up} = 440.3\,(\text{ft}) = 0.083\,(\text{mi})$$

L_{up_2}: basic two-lane segment with free-flow speed of 50 mi/h

$$Len_{up\_2} = 800 - 440.3 = 359.6\,(\text{ft}) = 0.068\,(\text{mi})$$

L_{up_1}: basic two-lane segment with free-flow speed of 60 mi/h

$$Len_{up\_1} = 4 \times 5280 - 800 = 20320\,(\text{ft}) = 3.848\,(\text{mi})$$

Step 5. Determine downstream effective length. Downstream effective length is estimated using Equation 4-4, as follows:

$$\begin{aligned} Len_{eff\_down} = {} & 701.34 + 51.016 \times \left(\frac{V_d}{100}\right) + 42.353 \times \%HV + 13.833 \times Cycle \\ & - 1.701 \times \left(\frac{V_d}{100}\right) \times \%LT - 16.760 \times g \end{aligned}$$

$$\begin{aligned} Len_{eff\_down} = {} & 701.34 + 51.016 \times \left(\frac{800}{100}\right) + 42.353 \times 0 + 13.833 \times 57.3 \\ & - 1.701 \times \left(\frac{800}{100}\right) \times 5 - 16.760 \times 33.8 \\ & = 1267.6\,(\text{ft}) \end{aligned}$$

Step 6. Subdivide downstream segment. The 4-mi downstream segment is divided into 2 segments with different features. The length of each segment is determined as follows:

L_{eff_down}: downstream effective segment

$$Len_{eff\_down} = 1267.6\,(\text{ft}) = 0.240\,(\text{mi})$$

L_{down_1}: basic two-lane segment with free-flow speed of 50 mi/h

$$Len_{down\_1} = 4 \times 5280 - 1267.6 = 19852.4\,(\text{ft}) = 3.760\,(\text{mi})$$

Step 7. Estimate *ATS* on basic two-lane segment. The average travel speed on each basic two-lane segment is estimated with the procedure in Chapter 15 of the HCM 2010 (TRB, 2010).

The basic two-lane highway segments with free-flow speed of 60 mi/h:

$$ATS_{basic\_60} = 47.0\,(\text{mi/h})$$

The basic two-lane highway segments with free-flow speed of 50 mi/h:

$$ATS_{basic\_50} = 37.2\,(\text{mi/h})$$

Step 8. Estimate control delay. The control delay for the analysis approach is estimated with the procedure in Chapter 18 of the HCM 2010 (TRB, 2010).

$$Delay_{control} = 13.5\,(\text{s})$$

Step 9. Determine the delay on each segment and total delay along the facility. The basic two-lane highway segment L_{up_1}:

$$Delay_{up\_1} = \frac{L_{up\_1}}{ATS_{basic\_60}} - \frac{L_{up\_1}}{FFS_{up\_1}} = \frac{3.848}{47.0} - \frac{3.848}{60.0} = 0.0177\,(\text{h}) = 63.72\,(\text{s})$$

The basic two-lane highway segment L_{up_2}:

$$Delay_{up\_2} = \frac{L_{up\_2}}{ATS_{basic\_50}} - \frac{L_{up\_2}}{FFS_{up\_2}} = \frac{0.068}{37.2} - \frac{0.068}{50.0} = 0.0005\,(\text{h}) = 1.8\,(\text{s})$$

The influence area of the signalized intersection is the combination of the upstream effective segment and downstream effective segment. The delay occurring in this area is accounted for by the control delay in Step 6.

$$Delay_{intersection} = Delay_{control} = 13.5\,(\text{s})$$

The basic two-lane highway segment L_{down_1}:

$$Delay_{down\_1} = \frac{L_{down\_1}}{ATS_{basic\_60}} - \frac{L_{down\_1}}{FFS_{down\_1}} = \frac{3.760}{47.0} - \frac{3.760}{60.0} = 0.0173\,(\text{h}) = 62.28\,(\text{s})$$

Therefore, the total delay can be calculated as:

$$Delay_{total} = Delay_{up\_1} + Delay_{up\_2} + Delay_{intersection} + Delay_{down\_1} = 141.3\,(\text{s})$$

Step 10. Determine *PD* and the level of service (LOS)

$$PD = \frac{Delay_{total}}{\frac{L_{up\_1}}{FFS_{up\_1}} + \frac{L_{up\_2}}{FFS_{up\_2}} + \frac{L_{eff\_up}}{FFS_{up\_2}} + \frac{L_{eff\_down}}{FFS_{down\_1}} + \frac{L_{down\_1}}{FFS_{down\_1}}} = 29.32\%$$

As the facility in this example is close to a Class III two-lane highway, the LOS can be determined by comparing the *PD* value with the *PD* criteria for Class III highways in Table 4-9. By applying the criteria, it is indicated that the analysis direction operates at LOS C.

4.8 Comparison of the New Models to the Previous Models

As the research approach used in the current study is different from the one used in the Yu and Washburn study (2009), the differences between the current analytical methodology and the previous one are understandable from the following aspects.

Simulation tool. The current methodology was developed based on the new version of CORSIM with the modeling capability of two-lane highways incorporated. Thus, a two-lane highway facility with various features (e.g., passing lane, signalized intersection, etc.) can be modeled in CORSIM as a whole. However, as such modeling capability was not available before, the previous methodology was developed by a hybrid simulation approach. In the Yu and Washburn study (2009), CORSIM was employed to model a two-lane, two-way road upstream of a signal, while TWOPAS was used to model a downstream two-lane highway segment. Although the TWOPAS input parameter *EPF* can be used to represent the effects of a signalized intersection, the traffic flow simulated in TWOPAS was not related to the traffic flow simulated in CORSIM. Thus, some inconsistencies may be ignored. Moreover, the passing models employed in the new CORSIM are different from the ones applied in TWOPAS, which is also a possible source of the differences.

Experimental design. The most different component in the current experimental design from the previous one is signal timing. As it is desired to keep major flows unimpeded as much as possible, three fully-actuated signal timing plans were used in the current experiments, as opposed to pretimed control used in the previous experiments. Fully-actuated signal control is more responsive and efficient than pretimed signal control. But at the same time, different signal control types will definitely result in different simulation data, which becomes a source of the differences.

Another source of differences can be the facility design used to determine the downstream effective length in the previous study. The average travel speed at each data collection station on a two-lane highway segment representing the situation without

a signal was compared with the one on a two-lane highway segment representing the segment downstream of the signal. However, as no lead-up segment (for obtaining normal platoons on the analysis two-lane highway segment) was included in simulating the two-lane highway segment without a signal, the average travel speed was close to the free-flow speed at the beginning and leveled off after some distance. This issue may result in the overestimation of the downstream effective length.

The differences may also come from the different centerline markings on the upstream link. The current experiments used a 4-mi long upstream segment with passing-allowed all the way except in the vicinity of the signal. However, the previous study used a 1-mi long upstream segment with no-passing-allowed all the way down to the signal (because CORSIM did not have the ability to model passing maneuvers in the oncoming lane at that time). Although it was indicated that passing maneuvers rarely happen when vehicles approach an intersection, that distance is usually less than 2000 ft if passing is allowed. The length of the passing zone has impacts on the pattern of platoons, which can lead to significant differences between the two methodologies.

Algorithms for determining upstream and downstream effective length. The previous methodology takes the combination of upstream effective length and acceleration distance as the influence area of a signal. The downstream effective length begins after the acceleration distance. Speed variation (based on 132-ft link average speed in CORSIM and 100-ft link average speed in TWOPAS) is the criterion for determining the beginning of the upstream effective length and the end of the downstream effective length.

In the current methodology, since vehicle trajectory data was used for the model development, the criteria for determining upstream and downstream effective length mainly focused on each individual vehicle's operation. Because the algorithms are consistent with the concept of control delay, the combination of upstream effective length and downstream effective length is considered as the influence area of a signal. The delay occurring within the influence area of a signal can then be fully represented by control delay.

Table 4-1. CORSIM truck types

Performance description	FRESIM vehicle type	NETSIM vehicle type
Single-unit truck	3	2
Semi-trailer truck with medium load	4	6
Semi-trailer truck with full load	5	7
Double-bottom trailer truck	6	8

Table 4-2. Experimental design for the scenarios w/ a left-turn bay

	Option 1	Option 2	Option 3	Option 4	Option 5
Flow rate (analysis direction) (veh/h)	200	400	600	800	1000
Heavy vehicles (%)	0	6	12	-	-
Signal timing plan	Plan I	Plan II	Plan III	-	-
Left-turn vehicles (%)	5	10	-	-	-

Table 4-3. Experimental design for the scenarios w/o a left-turn bay

	Option 1	Option 2	Option 3	Option 4
Flow rate (analysis direction) (veh/h)	200	400	600	800
D-factor	0.5	0.6	0.7	
Heavy vehicles (%)	0	6	12	-
Signal timing plan	Plan I	Plan II	Plan III	-
Left-turn vehicles (%)	5	10	-	-

Table 4-4. Signal timing plans

	Major phase (permitted left-turn phase)				Minor phase (permitted left-turn phase)			
	$Green_{max}$ (s)	$Green_{min}$ (s)	Yellow (s)	All-Red (s)	$Green_{max}$ (s)	$Green_{min}$ (s)	Yellow (s)	All-Red (s)
Plan I	44	10	4	1	22	6	4	1
Plan II	38	10	4	1	19	6	4	1
Plan III	50	10	4	1	25	6	4	1

Table 4-5. Statistical results of the upstream effective length model w/ a left-turn bay

Explanatory variables	Parameter	t-value	Adjusted R^2
Constant	266.6569	16.7318	
$(v_d/100)^2$	3.0468	22.8773	
Cycle	8.6256	10.4711	0.9798
$(v_d/100) \times \%LT$	-0.9715	-9.5560	
g	-14.1018	-14.1122	

Table 4-6. Statistical results of the upstream effective length model w/o a left-turn bay

Explanatory variables	Parameter	t-value	Adjusted R^2
Constant	412.0206	12.6299	
$(v_d/500)^3$	57.9968	15.9946	
$(v_o/500)^3$	85.1575	19.9209	0.9387
Cycle	-3.6558	-4.6047	
$[(v_d/500) \times \%LT]^3$	0.0327	10.1975	

Table 4-7. Statistical results of the downstream effective length model

Explanatory variables	Parameter	t-value	Adjusted R^2
Constant	701.3438	24.2451	
$(V/100)$	51.0164	7.9484	
%HV	42.3531	41.8964	0.9014
Cycle	13.8329	3.9211	
$(V/100)\times \%LT$	-1.7009	-5.0018	
g	-16.7603	-2.7970	

Table 4-8. Comparison results for validation

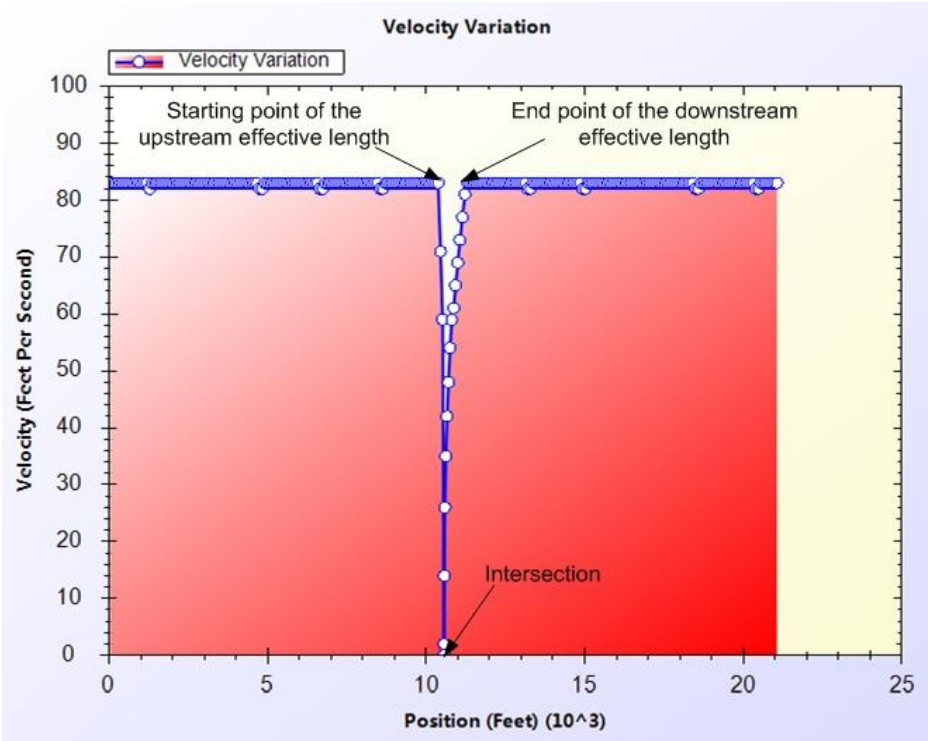
Directional flow rate (pc/h)	CORSIM output	Percent delay (%) *Calculated result (CORSIM)	**Calculated result (HCM)
200	5.20	4.56	10.00
600	11.74	10.79	22.10
1000	15.89	14.15	41.10

* The results were calculated based on the average travel speeds on the basic two-lane highway segments and the control delays produced by CORSIM.
** The results were calculated based on the proposed methodology in section 4.6.1.

Table 4-9. LOS criteria for two-lane highway facilities based on *PD*

Level of service	*PD* (%) Class I Highways	Class II Highways	Class III Highways
A	≤ 9	≤ 12	≤ 9.5
B	> 9–14	> 12–16	> 9.5–21.5
C	> 14–20.5	> 16–23	> 21.5–36.5
D	> 20.5–30	> 23–36.5	> 36.5–55.5
E	> 30	> 36.5	> 55.5

A



B

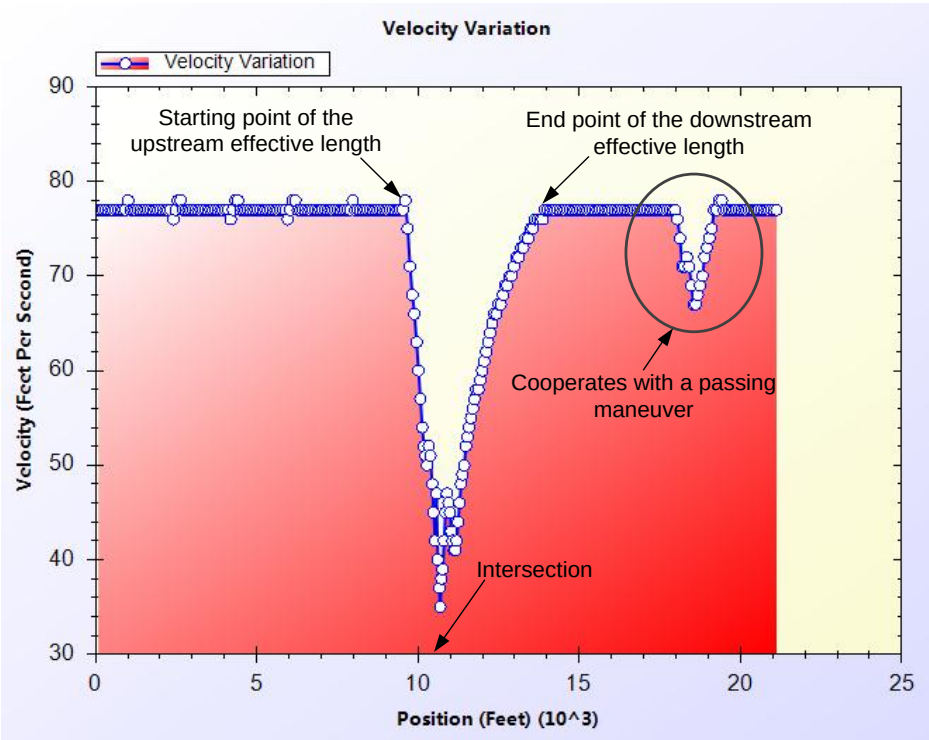


Figure 4-1. Trajectories of intersection affected vehicles. A) Type 1 upstream/downstream affected vehicle B) Type 2 upstream/downstream affected vehicle

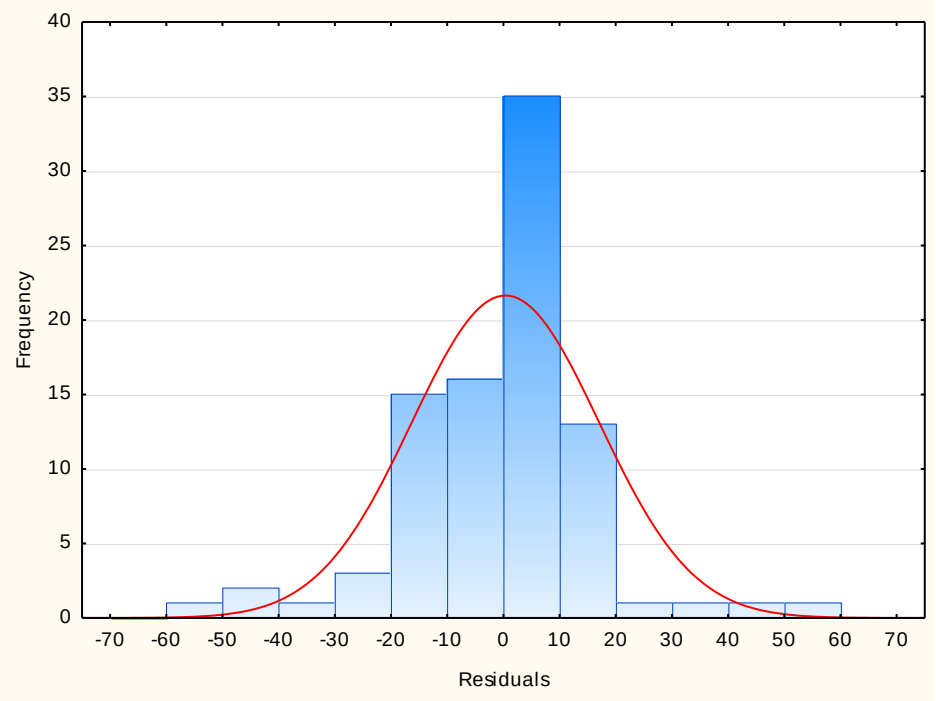


Figure 4-2. Residual histogram of the upstream effective length model w/ a left-turn bay

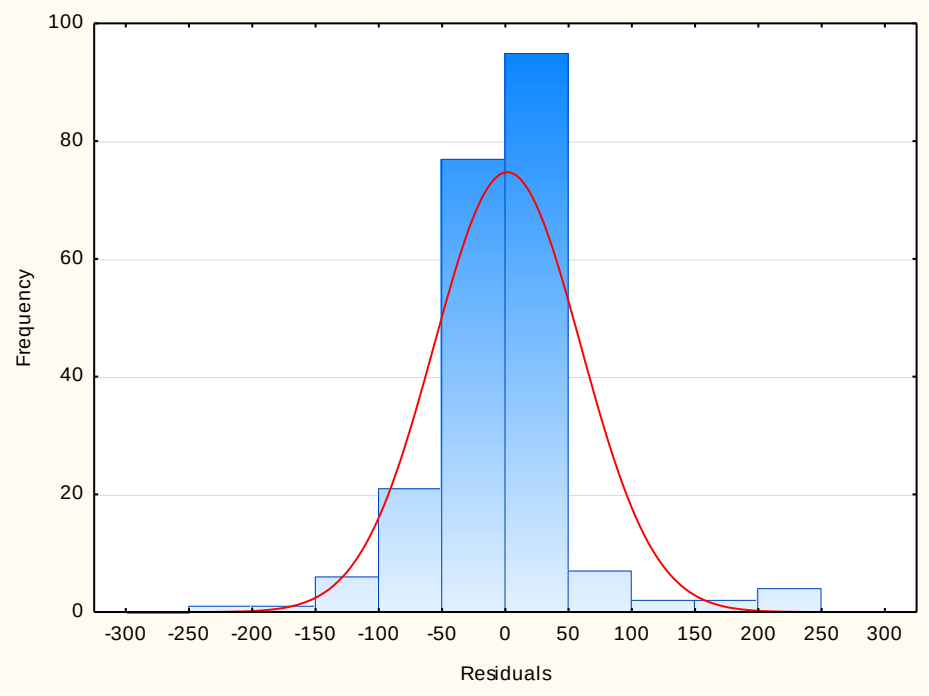


Figure 4-3. Residual histogram of the upstream effective length model w/o a left-turn bay

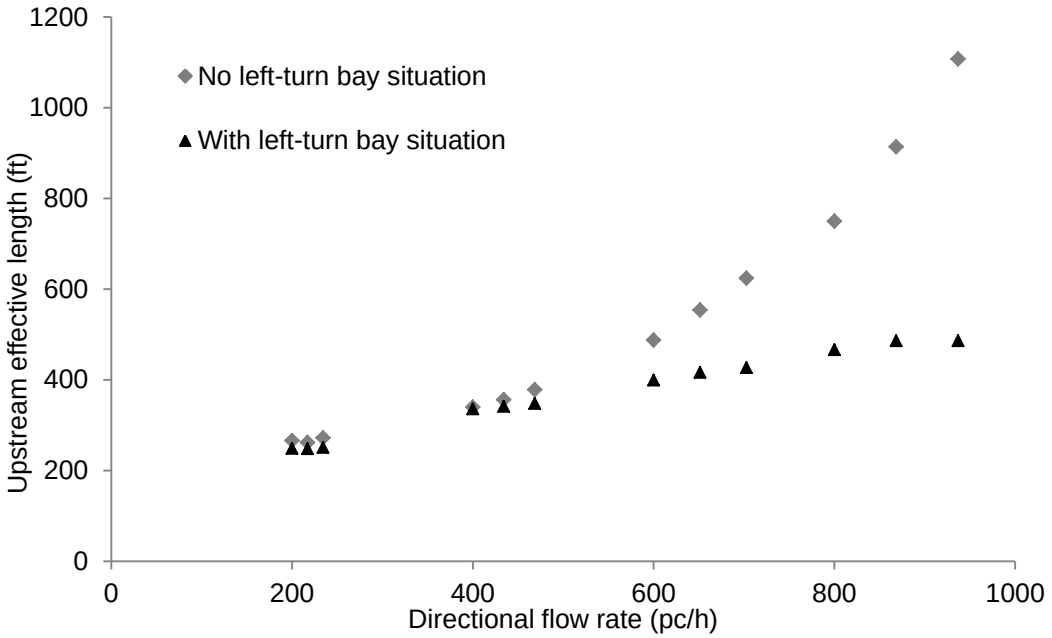


Figure 4-4. Comparison of predicted upstream effective lengths using two estimation models

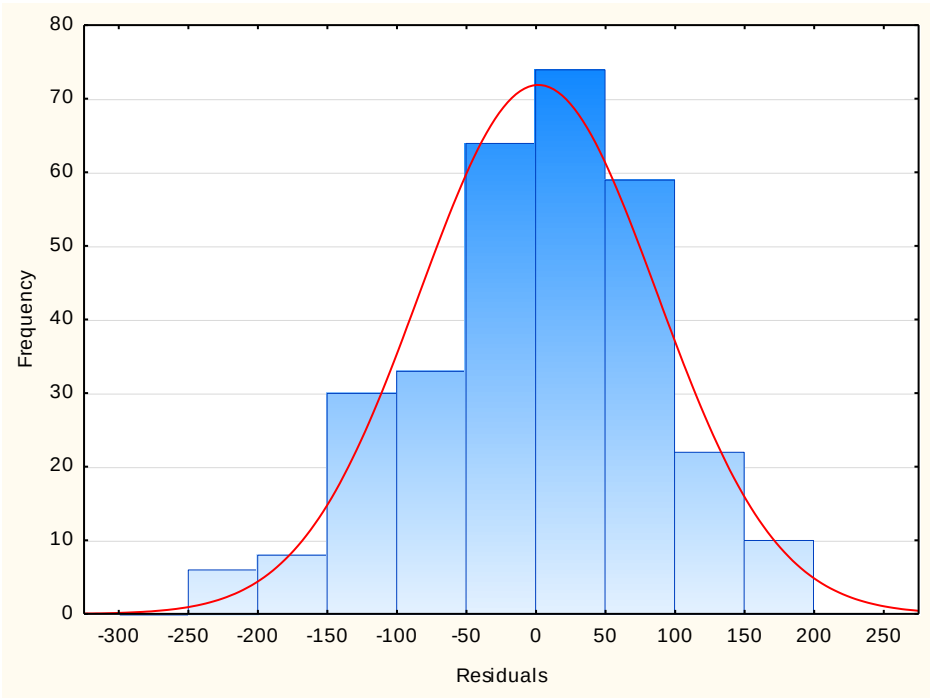


Figure 4-5. Residual histogram of the downstream effective length model

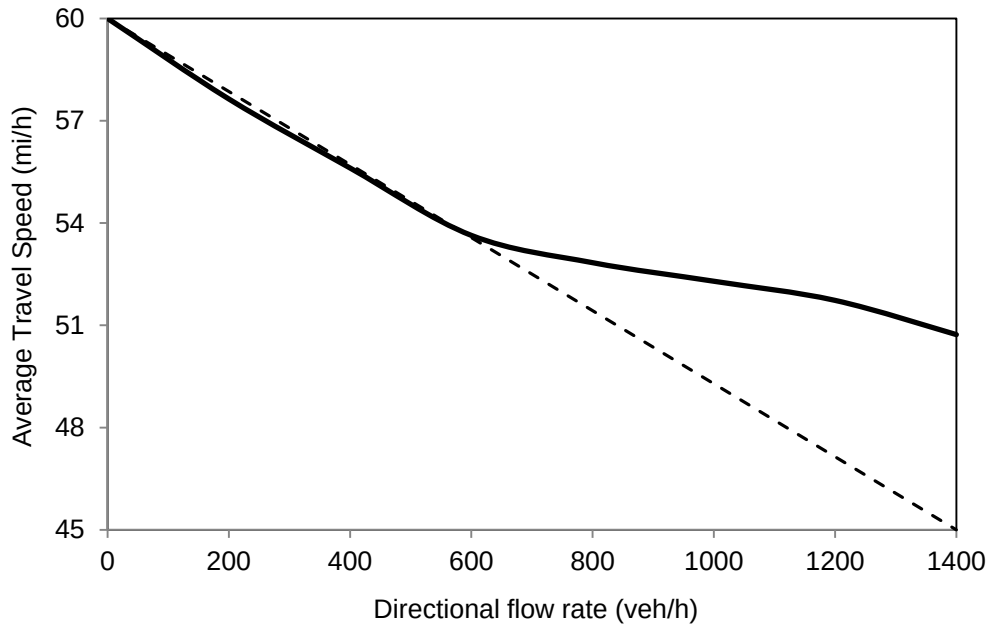


Figure 4-6. Directional speed-flow results

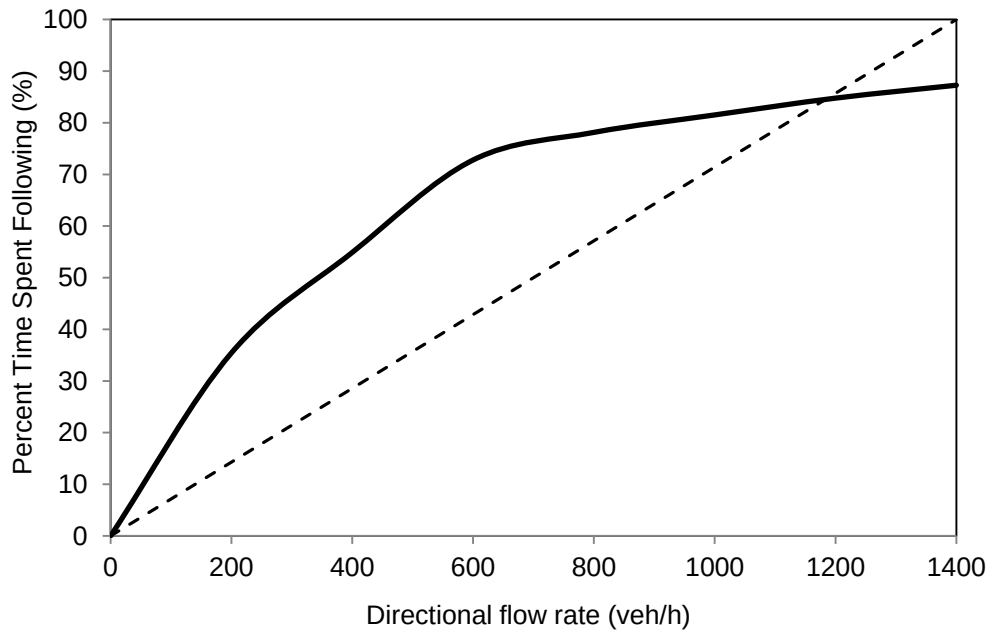


Figure 4-7. Directional *PTSF*-flow results

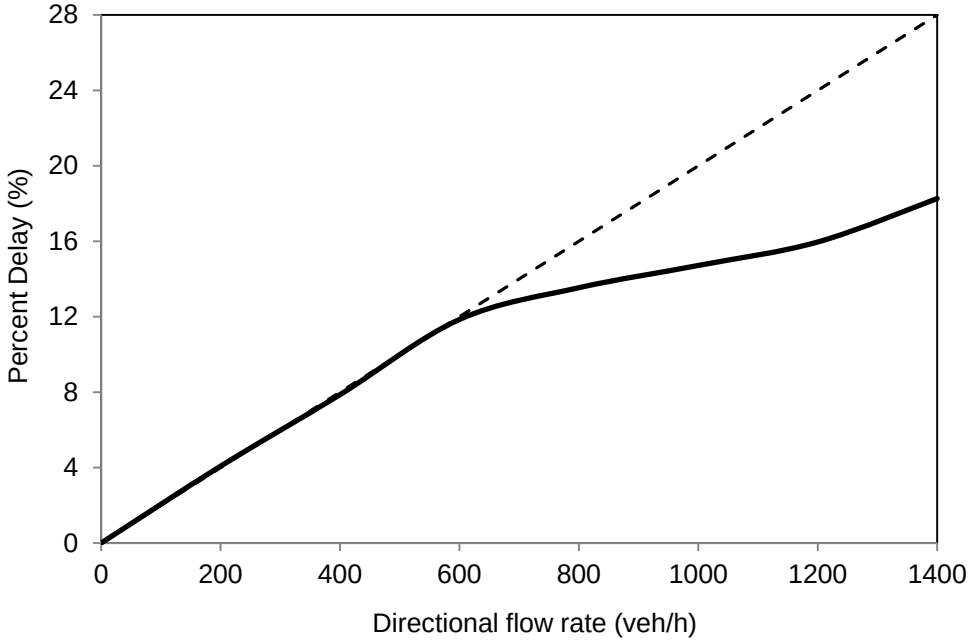


Figure 4-8. Directional *PD*-flow results

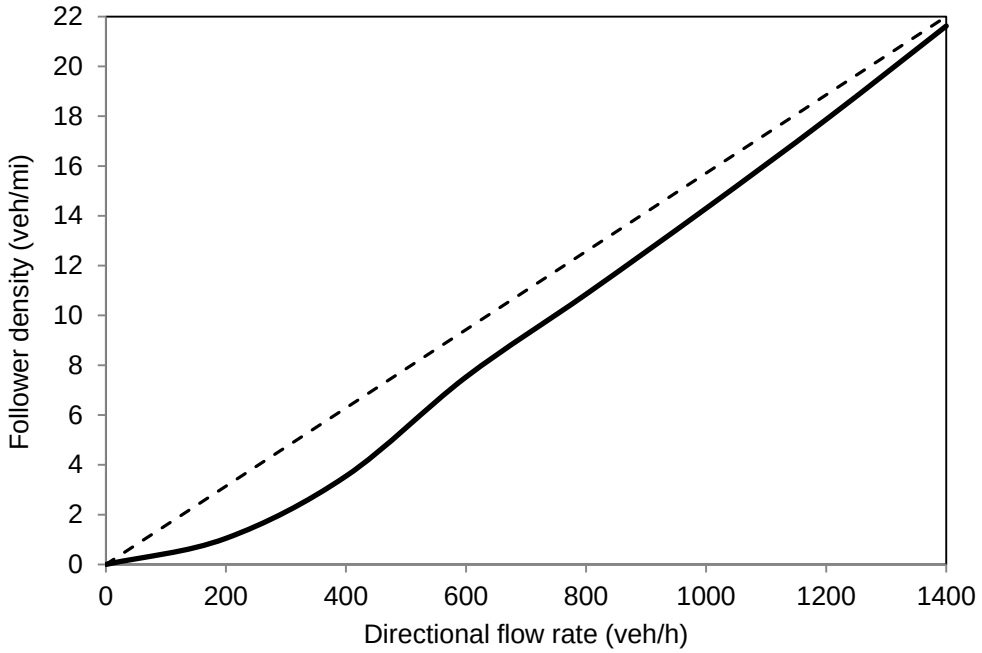


Figure 4-9. Directional follower density-flow results

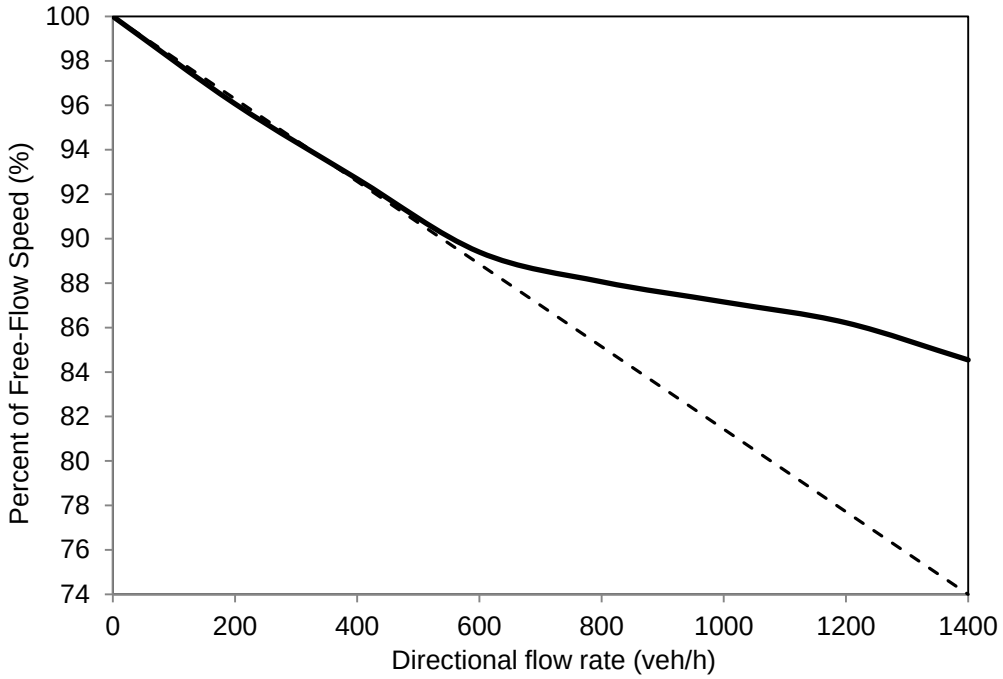


Figure 4-10. Directional *PFFS*-flow results

A

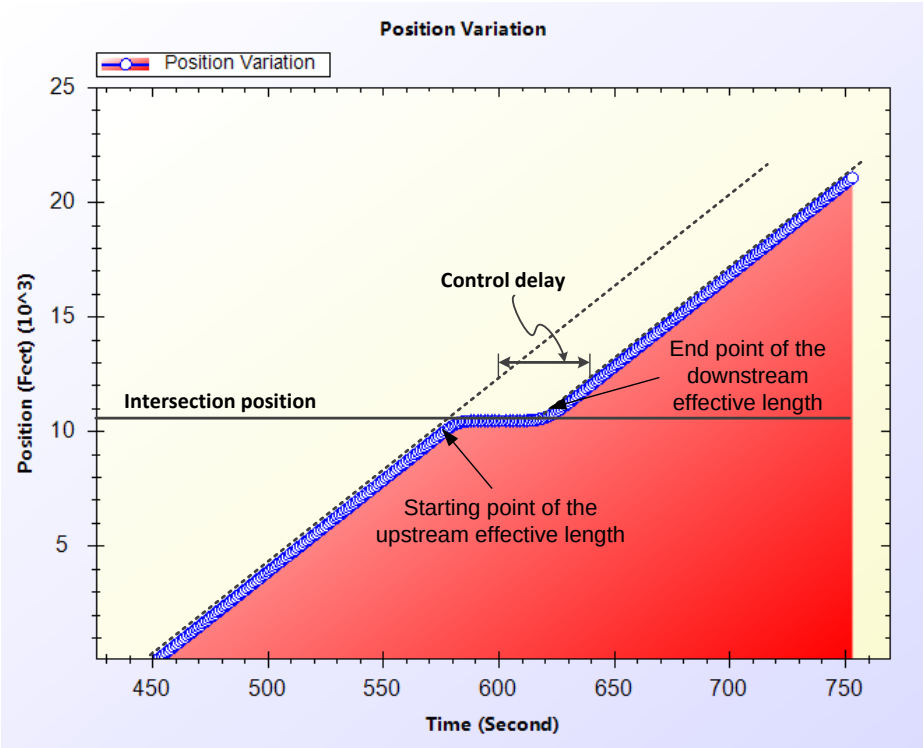
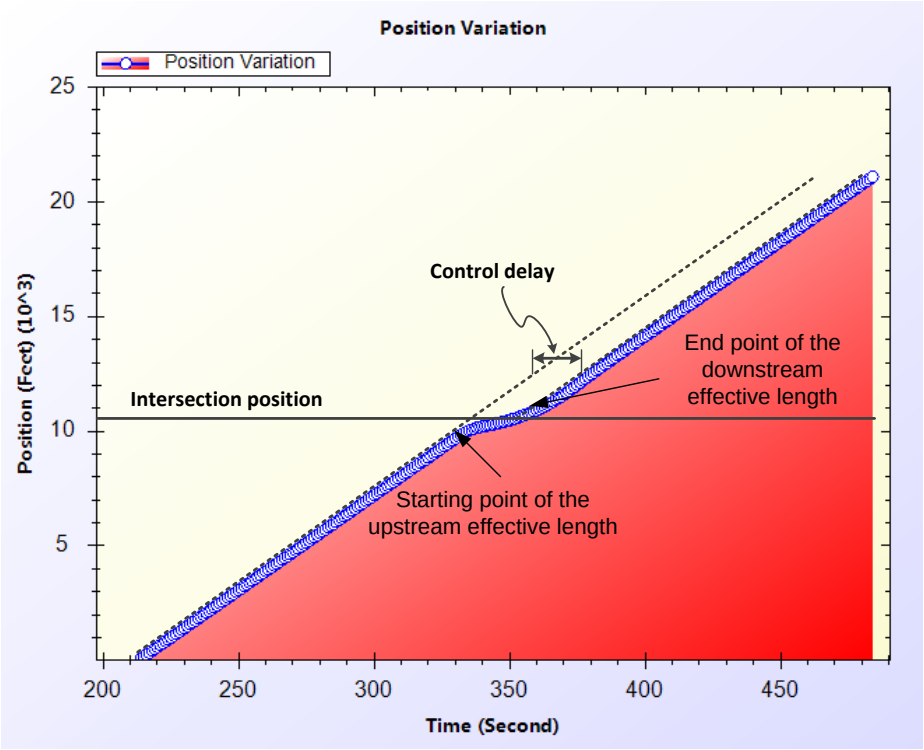


Figure 4-11. Trajectory of delayed vehicle on time-space plane. A) Delayed vehicle with a complete stop at intersection B) Delayed vehicle without a complete stop at intersection C) Delayed vehicle with a complete stop at intersection and a relatively long upstream/downstream effective length

B



C

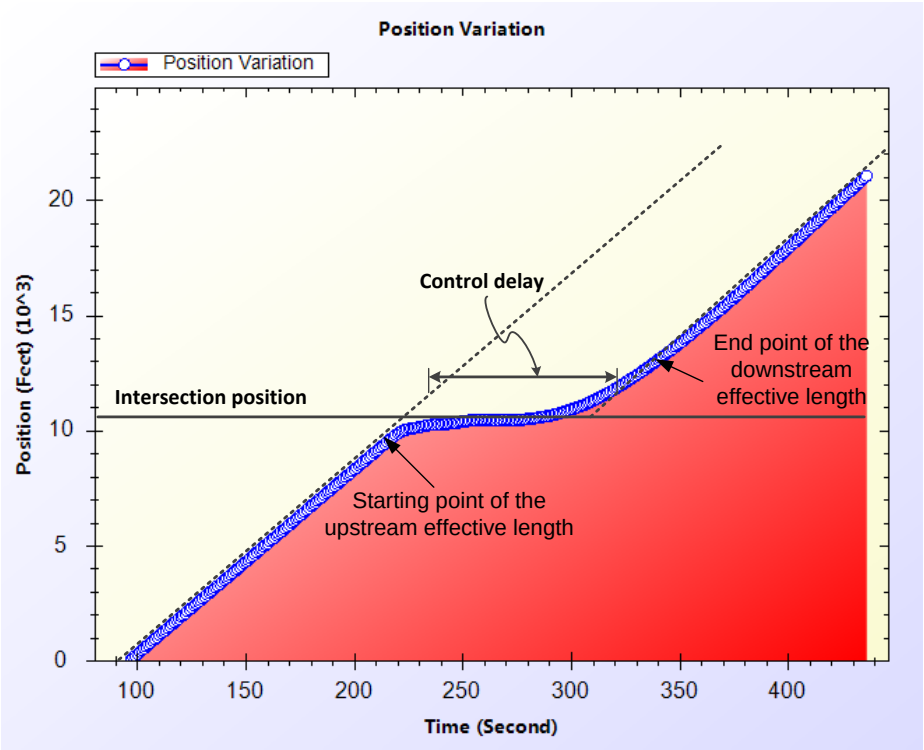


Figure 4-11. Continued

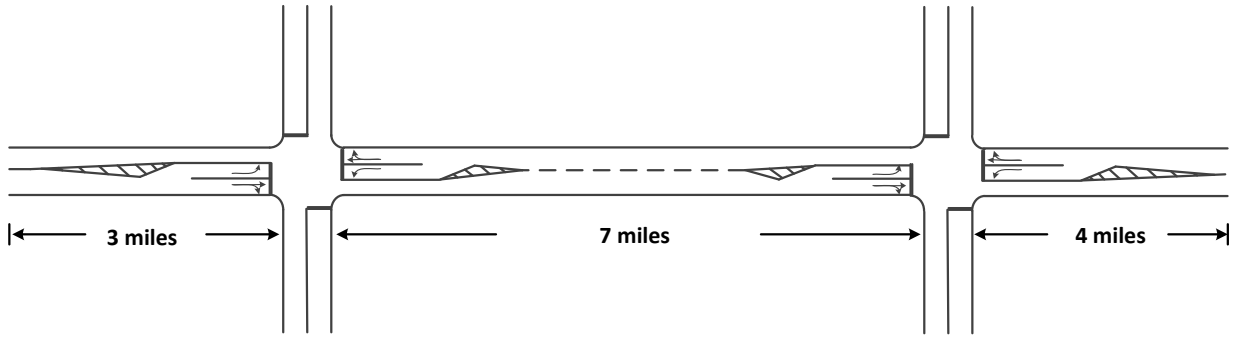


Figure 4-12. Illustration of a two-lane highway facility with two signalized intersections

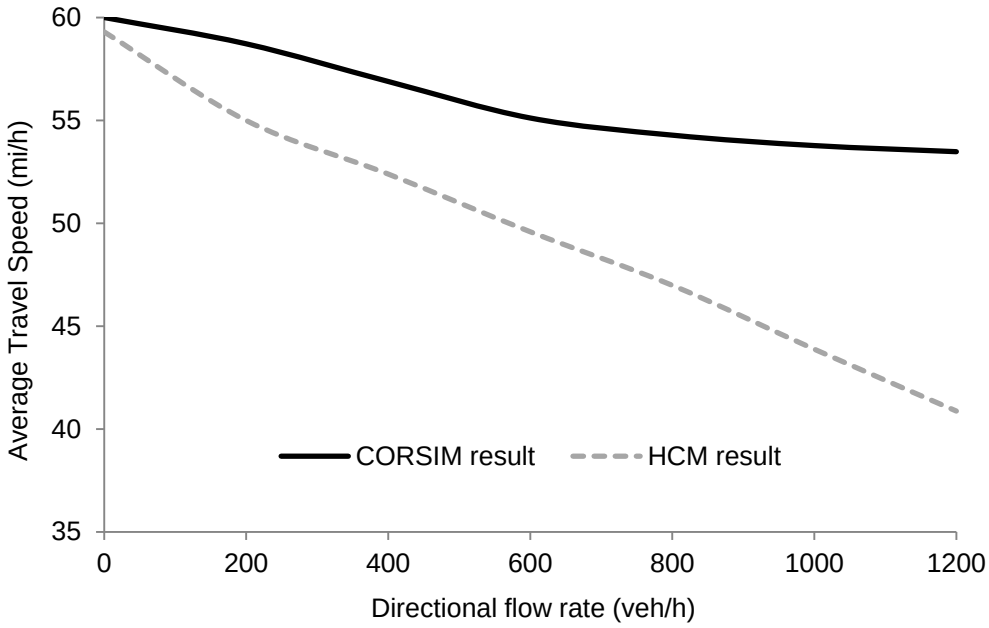


Figure 4-13. Comparison of CORSIM *ATS* and HCM *ATS*

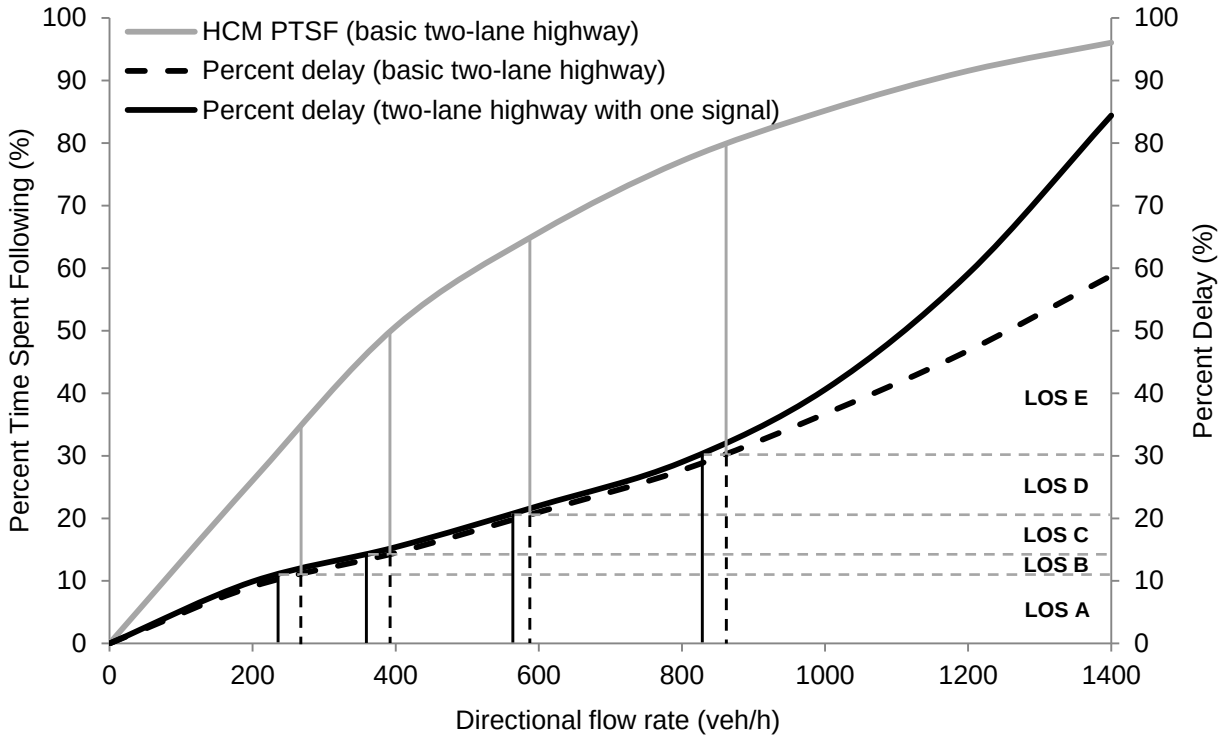


Figure 4-14. *PD* LOS criteria based on *PTSF* LOS criteria for Class I two-lane highways

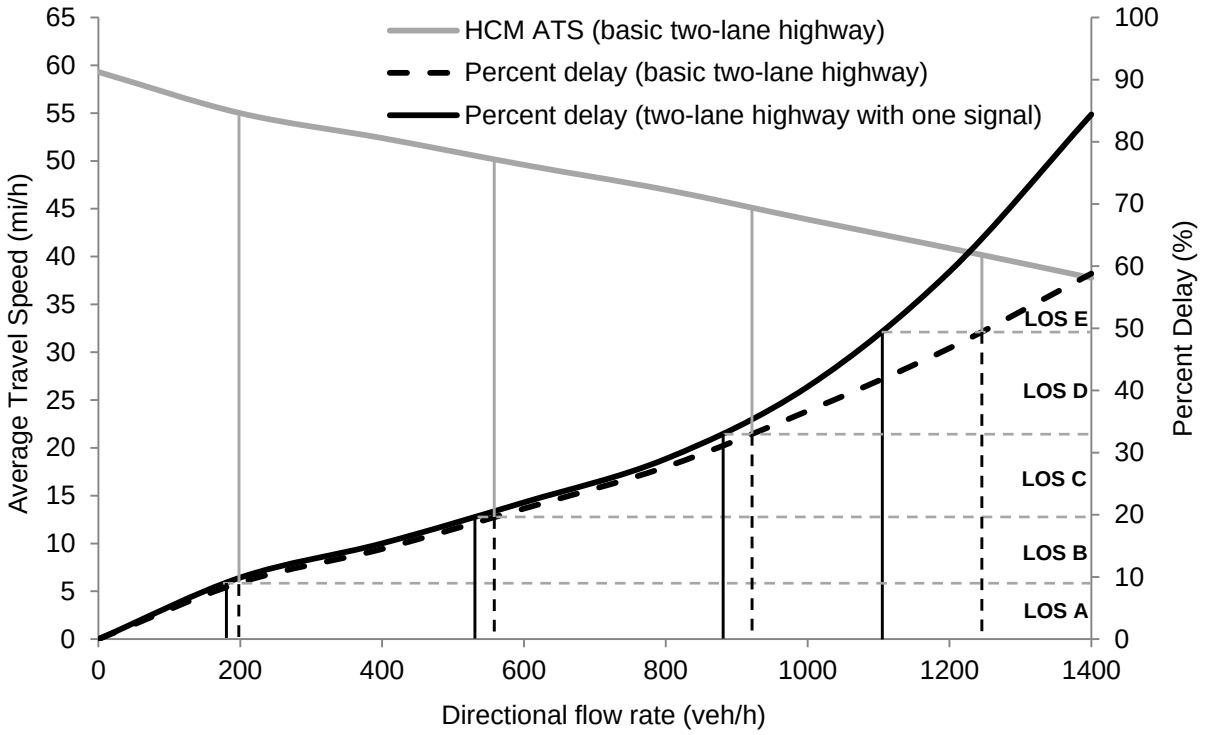


Figure 4-15. *PD* LOS criteria based on *ATS* LOS criteria for Class I two-lane highways

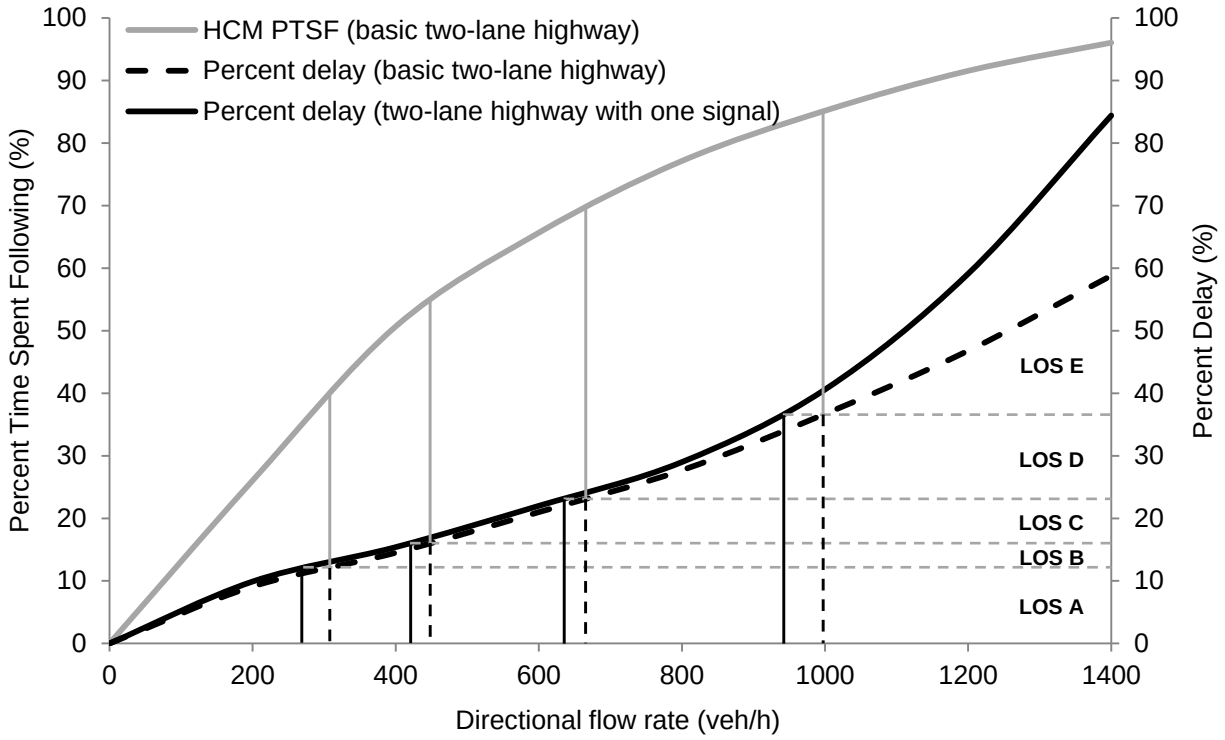


Figure 4-16. *PD* LOS criteria based on *PTSF* LOS criteria for Class II two-lane highways

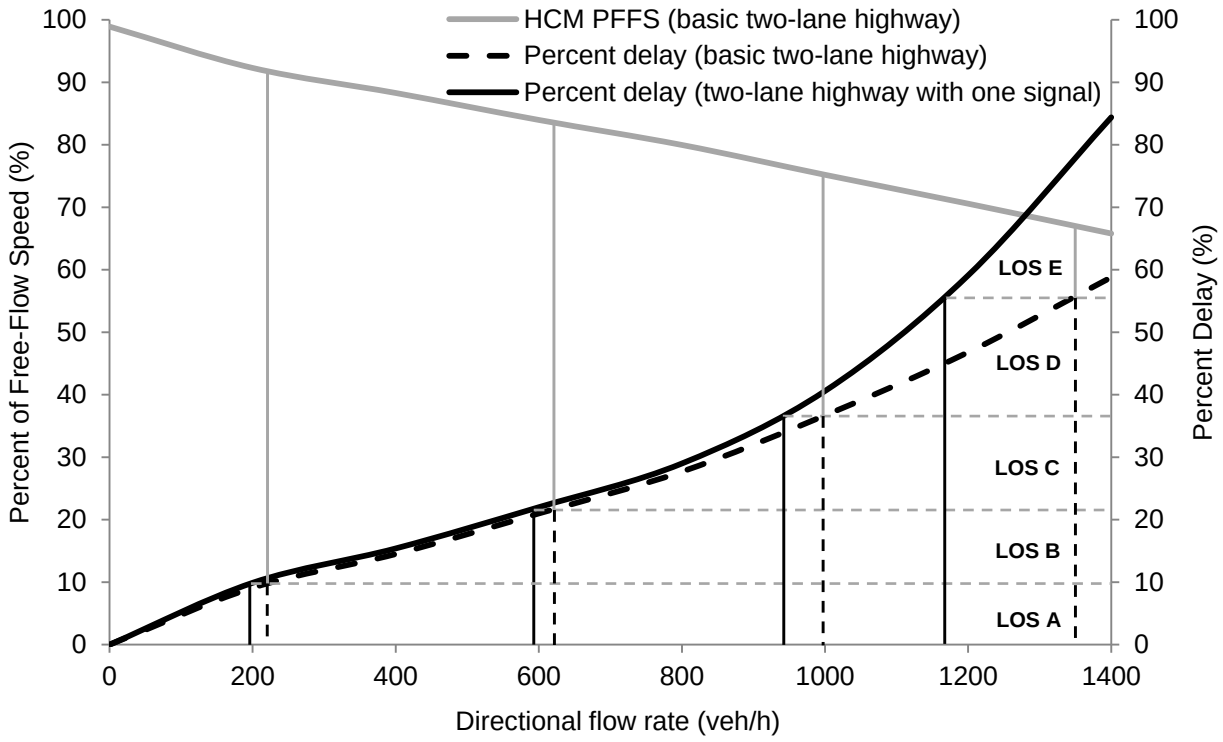


Figure 4-17. *PD* LOS criteria based on *PFFS* LOS criteria for Class III two-lane highways

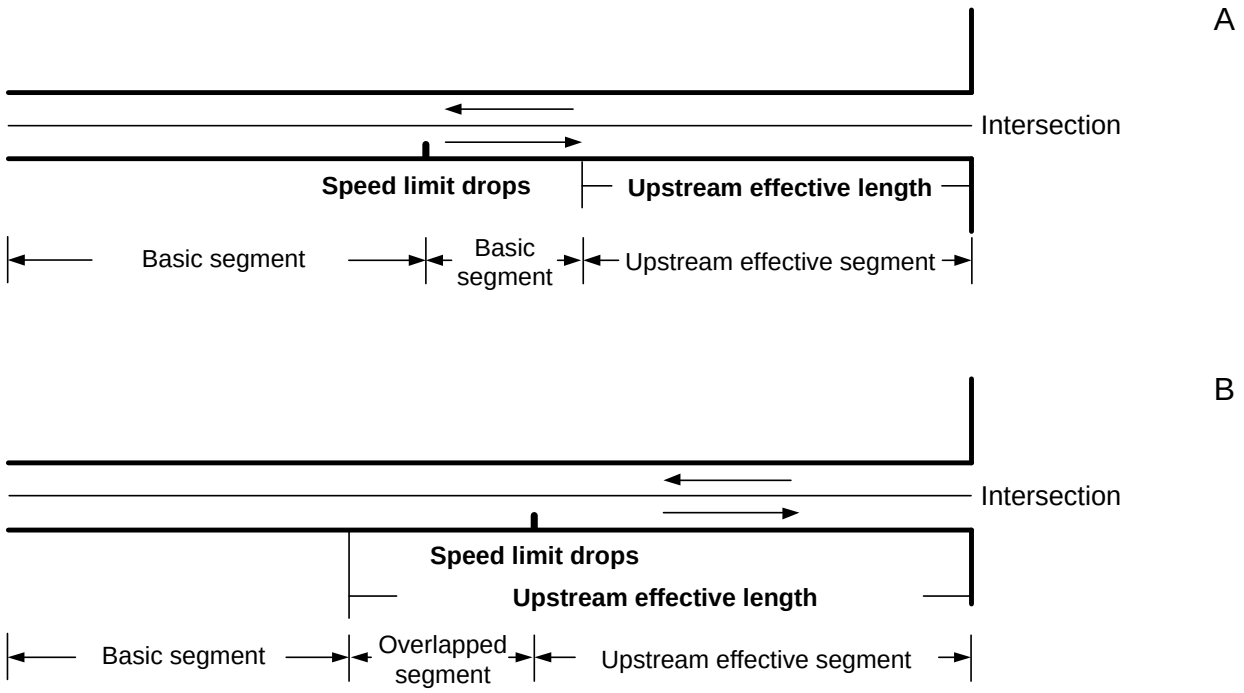


Figure 4-18. Illustration of facility segmentation. A) Speed limit drops before the starting point of upstream effective length B) Speed limit drops after the starting point of upstream effective length

CHAPTER 5
SUMMARY AND RECOMMENDATIONS

5.1 Summary

The work presented in this dissertation focuses on the development of a simulation program and analytical methodology for two-lane highways. As the capability of modeling two-lane highways has been incorporated into CORSIM, the following two-lane highway facilities can be modeled within CORSIM:

- Basic two-lane highway segments with passing maneuvers (including passing one vehicle or multiple vehicles at a time) in the oncoming lane.
- Two-lane highway segments with a passing lane.
- Two-lane highway segments connecting to signalized intersections.

The simulation program also allows users to modify newly added parameters for modeling two-lane highway facilities or operations. The new performance measure outputs (e.g., *PTSF*, follower density, and *PD*) and passing statistics outputs are available for different purposes of analysis.

As the two-lane highway modeling logic applied in the simulation program is reasonably consistent with theories and field observations from previous studies, the modeling results are generally reasonable and consistent with expected traffic-flow theory for two-lane highways.

In the accomplishment of the simulation modeling capability built in CORSIM, it allows users to develop comprehensive traffic networks including a variety of facility types for different purposes of analysis.

Based upon this new simulation capability in CORSIM, the previous methodology for two-lane highway facility analysis developed by Yu and Washburn (2009) was updated. The new methodology retains the concept of facility segmentation from the

previous methodology, but was developed in a different way. First, the testing facility including both two-lane highway segments and a signalized intersection was established integrally in CORSIM, while the previous methodology used a hybrid simulation approach. Second, the algorithms to determine basic two-lane highway segments and intersection influence areas were developed based upon the information of individual vehicles, instead of aggregate link performance that was used in developing the previous methodology.

The service measure percent delay proposed in the previous methodology was examined and verified for its efficacy at determining the level of service of a two-lane highway facility including signalized intersections. A percent delay based LOS criteria derived from the simulation results of a set of experiments is proposed. In addition, guidance for facility segmentation when speed limit reductions are applied in the vicinity of a signalized intersection is provided.

The application of the methodology can be extended to other complex two-lane highway facilities that include not only signalized intersections, but also other features (e.g., passing lanes).

It is also concluded that follower density, which reflects the levels of flow rate, speed, and percentage of followers, can be an efficient service measure for uninterrupted two-lane highway facilities, as the experiment results indicate that the relationship between follower and flow rate is closest to a linear relationship, compared with the performance measures of average travel speed, percent time-spent-following, and percent free-flow speed.

5.2 Recommendations

Although the modeling capability of two-lane highways incorporated into CORSIM is based on existing traffic-flow theory and field observations for two-lane highways, there are several potential studies that can be done to further refine and/or validate the current modeling logic, as well as improve the capabilities, as follows:

- Refinement of desire to pass (*DTP*) and willingness to move over (*WTMO*) algorithms through direct traveler input (e.g., through focus groups).
- Incorporation of additional performance measures as proposed in the literature.
- Modification of the graphical network editor (TRAFED) to be able to accommodate editing of two-lane highway networks.
- Collection and use of field data to calibrate and validate the two-lane highway modeling logic.

Another potential future study is to use field data to validate the new methodology for two-lane highway facilities with signalized intersections. Due to lack of field data in this study, the methodology is only validated within CORSIM. Input data, such as truck fleet composition, acceleration ability, and desired free-flow speed distribution, may affect the upstream/downstream effective length models in the methodology. Thus, adjustments might be necessary when applying the methodology in practice.

It is also recommended that the algorithms to determine the influence area of a signalized intersection be applied in the determination of control delay in CORSIM, as they share the same conceptual definition.

CORSIM produces a nonlinear relationship between average travel speed and flow rate, whereas the HCM 2010 provides a linear relationship. This can lead to significant differences in simulation and HCM results for two-lane highway segments.

Thus, it is recommended that field studies be done to further investigate the speed-flow relationship for two-lane highways.

The LOS criteria for two-lane highways certainly need more investigation. Follower density is a promising service measure for uninterrupted two-lane highway facilities because it reflects flow rate, speed and percentage of followers, which are the essential factors that determine the performance on a two-lane highway facility, and it has a near-linear relationship with flow rate indicated by simulation results. For two-lane highway facilities with signalized intersections, it is proposed that follower density can serve as a supplementary performance measure to percent delay, which is the primary performance measure used to determine the level of service of such facilities.

LIST OF REFERENCES

AASHTO. (2001). *A Policy on Geometric Design of Highways and Streets, Fourth Edition.* Washington, D.C.: AASHTO (American Association of State Highway and Transportation Officials).

Al-Kaisy, A., & Durbin, C. (2008). Evaluating New Methodologies for Estimating Performance on Two-Lane Highways. *Canadian Journal of Civil Engineering*, 777-785.

Al-Kaisy, A., & Karjala, S. (2010). Car-Following Interaction and the Definition of Free-Moving Vehicles on Two-Lane Rural Highways. *Journal of Transportation Engineering*, 925-931.

Al-Kaisy, Ahmed; Freedman, Zachary;. (2010). Estimating Performance on Two-Lane Highways: Case Study Validation of a New Methodology. *Transportation Research Record, 2173*, 72-79.

Brilon, W., & Weiser, F. (2006). Two-Lane Rural Highways: The German Experience. *Transportation Research Record, 1988*, 38-47.

Carlson, P. J., Miles, J. D., & Johnson, P. K. (2006). Daytime High-Speed Passing Maneuvers Observed on Rural Two-Lane, Two-Way Highways: Findings and Implications. *Transportation Research Record, 1961*, 9-15.

Catbagan, J. L., & Nakamura, H. (2006). Evaluation of Performance Measures for Two-Lane Expressways in Japan. *Transportation Research Record, 1988*, 111-118.

Catbagan, J. L., & Nakamura, H. (2008). Desired Speed Distributions on Two-Lane Highways Under Various Conditions. *Transportation Research Record, 2088*, 218-226.

Catbagan, J. L., & Nakamura, H. (2009). Probability-Based Follower Identification in Two-Lane Highways. *Transportation Research Board 88th Annual Meeting* (pp. 09-0103). Washington, DC: Transportation Research Board.

Cohen, S. (2002). Application of Car-Following Systems to Queue Discharge Problem at Signalized Intersections. *Transportation Research Record, 1802*, 205-213.

DelDOT. (2009, May). *DelDOT Manual on Uniform Traffic Control Devices (MUTCD) – Part 3: Markings – Frequently Asked Questions.* Retrieved Dec 19, 2009, from Delware Department of Transportation: http://www.deldot.gov/information/business/drc/manuals/deldot_mutcd_part_3_faq.pdf

Dey, P. P., Chandra, S., & Gangopadhyay, S. (2008). Simulation of Mixed Traffic Flow on Two-Lane Roads. *Journal of Transportation Engineering*, 361-369.

Dixon, M. P., Kyte, M., & Sarepalli, S. S. (2003, January). *Effects of Upstream Signalized Intersections on Two-Lane Highway Operations*. Retrieved 8 23, 2010, from http://www.ltrc.lsu.edu/TRB_82/TRB2003-002075.pdf

Dixon, M. P., Sarepali, S., & Young, K. A. (2002). Field Evaluation of Highway Capacity Manual 2000 Analysis Procedures for Two-Lane Highways. *Transportation Research Record, 1802*, 125-132.

Durbin, C. T. (2006). *Traffic Performance on Two-Lane, Two-Way Highways: Examination of New Analytical Approaches.* Master Thesis of Montana State University.

Facility Analysis and Simulation Team, Transportation Development Division, Oregon Department of Transportation. (2010, December). *Modeling Performance Indicators on Two-Lane Rural Highways: The Oregon Experience.* Retrieved 9 19, 2011, from OREGON.gov: http://www.oregon.gov/ODOT/TD/TPAU/docs/A_Background/ModelingPerf.pdf

Farah, H., & Toledo, T. (2010). Passing behavior on two-lane highways. *Transportation Research Part F*, 355-364.

FHWA. (2003). *Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD).* Washington, D.C.: Federal Highway Administration (FHWA).

Glennon, J. (1988). New and Improved Model of Passing Sight Distance on Two-Lane Highways. *Transportation Research Record, 1195*, 132-137.

Glennon, J. C. (1988). New and Improved Model of Passing Sight Distance on Two-Lane Highways. *Transportation Research Record, 1195*, 132-137.

Harwood, D. W., Gilmore, D. K., Richard, K. R., & Dunn, J. M. (2008). *Passing Sight Distance Criteria.* Washington, D.C.: Transportation Research Board.

Harwood, D. W., May, A. D. , Anderson, I. B. , Leiman, L. , & Archilla, A. R. . (1999). *Final Report, NCHRP Project 3-55(3).* Midwest Research Institute, University of California, Berkeley.

Harwood, D. W., May, A. D., Anderson, I. B., Leiman, L., & Archilla, A. R. (1999). *CAPACITY AND QUALITY OF SERVICE OF TWO-LANE HIGHWAYS.* Berkeley, California, USA.: Midwest Research Institute, University of California-Berkeley.

Harwood, D. W., Potts, I. B., Bauer, K. M., Bonneson, J. A., & Elefteriadou, L. (2003). *NCHRP Project 20-7 (160): Two-Lane Road Analysis Methodology in the Highway Capacity Manual.* Washington, D.C.: Transportation Research Board, National Research Council.

Harwood, D., Gilmore, D.K., Richard, K.R., Dunn, J.M, & Sun, C. (2008). *NCHRP Report 605: Passing Sight Distance Criteria.* Washington, D.C: Transportation Research Board, TRB, National Research Council.

Hassan, Y., Easa, S. M., & Halim, A. O. Abd El. (1996). Passing Sight Distance on Two-Lane Highways: Review and Revision. *Transportation Research Part A, Vol. 30, No. 6*, 453-467.

Hassan, Y., & Easa, S. M. (1996). Passing Sight Distance on Two-Lane Highways: Review and Revision. *Transportation Research Part A*, 453-467.

Higgins, L. L., & Beesinger, J. (2006). *Driver Perceptions of Risk: Potential Approaches to Improving Driver Safety.* College Station, TX: Southwest Region University Transportation Center.

ITT Corporation, S. D. (2007). *Interactive Highway Safety Design Model (IHSDM) Traffic Analysis Module Engineer's Manual.* New York: ITT Corporation.

Khoury, J. E. (2007). Incorporating Uncertainty into the Estimation of the Passing Sight Distance Requirements. *Computer-Aided Civil and Infrastructure Engineering*, 347-357.

Kim, J. (2006). *A Capacity Estimation Method for Two-lane, Two-way Highways Using Simulation Modeling.* Dissertation of the Pennsylvania State University.

Kim, J.-T., Courage, K. G., Washburn, S. S., & Bonyani, G. (2003). Framework for Investigation of Level-of-Service Criteria and Thresholds on Rural Freeways. *Transportation Research Record, 1852*, 239-245.

Koorey, G. (2002). Assessment of Rural Road Simulation Modelling Tools. *IPENZ Transportation Group Technical Conference 2002.* Rotorua, New Zealand.

Leiman, L., Archilla, A. R., & May, D. A. (1998). *NCHRP Project 3-55(3) Capacity and Quality of Service of Two-Lane Highways: TWOPAS Model Improvements.* Washington, D.C.: Transportation Research Board, National Research Council.

Lemessi, M. (2001). AN SLX-BASED MICROSIMULATION MODEL FOR A TWO-LANE ROAD SECTION. *Proceedings of the 2001 Winter Simulation Conference* (pp. 1064-1071). Arlington, VA: Inst Elect & Electronic Engineers.

McLean, J. R. (1989). *Two-Lane Highway Traffic Operations – Theory and Practice.* New York: Gordon and Breach Science Publishers.

Morriss, J. L. (2005). *Identification of Preferred Performance Measures For The Assessment of Level of Service on Two-Lane Highways.* Master's Thesis of the University of Florida.

Polus, A., & Cohen, M. (2009). Theoretical and Empirical Relationships for the Quality of Flow and for a New Level of Service on Two-Lane Highways. *Journal of Transportation Engineering*, 380-385.

Romana, M. G., & Pérez, I. (2006). Measures of Effectiveness for Level-of-Service Assessment of Two-Lane Roads: An Alternative Proposal Using a Threshold Speed. *Transportation Research Record, 1988*, 56-62.

TRB. (2010). *Highway Capacity Manual 2010.* Washington, D.C.: Transportation Research Board.

Van as, C. (2006). *The Development of An Analysis Method for The Determination of Level of Service of Two-Lane Undivided Highways in South Africa.* Pretoria, South African: South African National Roads Agency Limited.

Vanvalkenburg, G. W., & Michael, H. L. (1971). Criteria for No-Passing Zones. *Highway Research Record, 377*, 1-19.

Warnshuis, P. (1967). Simulation of Two-Way Traffic on An Isolated Two-Lane Road. *Transportation Research*, 75-83.

Washburn, S. S., & Li, J. (2010). *Development of a Simulation Program for Two-Lane Highway Analysis.* Gainesville, FL: Center for Multimodal Solutions for Congestion Mitigation, University of Florida.

Weaver, G. D., & Glennon, J. C. (1972). Design and Striping for Safe Passing Operations. *Highway Research Record, 388*, 36–39.

Wooldridge, M. D., Messer, C. J., Heard, B. D., Raghupathy, S., Parham, A. H., Brewer, M. A., et al. (2001). *Design Guidelines for Passing Lanes on Two-lane Roadways (Super 2) .* College Station, Texas: Texas Transportation Institute, The Texas A&M University System.

Yan, X., Radwan, E., Zhang, F., & Parker, J. C. (2008). Evaluation of Dynamic Passing Sight Distance Problem Using a Finite-Element Model. *Journal of Transportation Engineering*, 225-235.

Yu, Q., & Washburn, S. S. (2009). Operational Performance Assessment for Two-Lane Highway Facilities. *Journal of Transportation Engineering*, 197-205.

BIOGRAPHICAL SKETCH

Jing Li was born in Dunhuang, China. She studied traffic and transportation engineering at Beijing Jiaotong University, and received her bachelor's degree in 2004. After that, she entered the Graduate School of Beijing Jiaotong University in 2005 and received her master's degree in transportation engineering. Jing started her PhD program at the University of Florida in fall 2008, and became Research Assistant in Jan. 2009 under the supervision of Dr. Scott Washburn. During her PhD study, Jing worked on the projects two-lane highway simulation and analysis. Jing has been awarded Master of Science degree in civil engineering at the University of Florida. Jing Li's primary research interest is traffic simulation and traffic operation analysis.