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CHAPTER 1   INTRODUCTION

Florida is experiencing rapid growth and development. This applies not only to urban areas, but to rural areas as well. This growth is now resulting in congestion on facilities that previously did not have any. One area that is becoming a concern, particularly in Florida, is rural areas transitioning into a more developed area. Access to these areas is usually by two-lane highways, but within these areas, there may be an occasional traffic signal, and possibly segments of multilane highway as well.

The Highway Capacity Manual (HCM) contains an analysis procedure for basic two-lane highway segments that serves as the de facto standard in the U.S. However, this procedure does not provide for the capability of performing an integrated analysis of an extended length of two-lane highway that also contains occasional signalized intersections. Some transportation professionals have indicated that a facility-based evaluation methodology for two-lane highways would be much more useful to them than the separate and somewhat disparate two-lane highway segment and signalized intersection analysis methodologies. The Highway Capacity and Quality of Service (HCQS) committee has officially acknowledged that the current procedure is not appropriate for analyzing low-speed, two-lane highways in developed areas, and also cannot account for the presence of a traffic signal (TRB, 2006).

This issue was first addressed by the Level of Service (LOS) Task Team of the Florida Department of Transportation (FDOT). This task team consists of representatives from the FDOT central and district office who are in charge of the FDOT level of service analysis program in their respective geographic regions. The issue first surfaced when representatives from District 6 (Miami) were struggling with how to analyze some major two-lane highways in their region that included a signalized intersection every 3-7 miles. Through discussions by this task team, it become clear that there was not an analysis procedure contained in the HCM that could be applied to this situation, nor was there any guidance on how to go about analyzing this situation. This issue was brought forward to the Highway Capacity and Quality of Service (HCQS) committee in 2004. The committee formally acknowledged that this situation was not addressed in the HCM, and added language to the manual to indicate this. In April of 2002, a National Cooperative Highway Research Program (NCHRP) project was awarded to the Midwest Research Institute (MRI) to address issues regarding the two-lane highway LOS methodology in the HCM 2000 (Harwood, et al. 2003), including developing some preliminary recommendations on potential analysis approaches for two-lane highways with occasional signalized intersections. In 2007, a research needs statement produced by the HCQS committee pointed out that intersection related operational treatments for signalized and un-signalized intersections and access points can have an effect on two-lane highway operations and that two-lane highways should be analyzed as a facility, including highway sections and intersections (TRB, 2007).

In addition to the limitations of a segment-only two-lane highway analysis procedure in the HCM, this procedure has been subject to debate and criticism in several other areas as well. Brilon et al. (2006) questioned the shape of the fundamental speed-flow diagram. Brilon (2006)

and Harwood (1999) et al. pointed out that the Passenger Car Equivalency (PCE) values might not correctly reflect the influence of heavy vehicles. Luttinen (2002), Van as (2006), Cabagan (2006), and Romana et al. (2006) have questioned the applicability of the current performance measures and have investigated alternative ones. Washburn et al. (2002) and Luttinen (2001) previously identified issues with the estimation of Percent Time Spent Following (*PTSF*) that can lead to unrealistic values. Some of these issues might also be related to the modeling capabilities of the software program (TWOPAS, discussed in the next section) used in the development of the HCM analysis procedure (Krummins, 1991; Dixon et al., 2006).

Based on the simulation tool developed in CMS Project 2008-002, two-lane highways with occasional signalized intersections will be analyzed, or an analysis methodology for this situation will be developed, and the further investigation of the academic issues mentioned above will be pursued. Since the existing simulation technology does not have such modeling capability as the new one, the current research is limited to some extent. In the Yu study, which was under the supervision of Dr. Washburn, an analytical two-lane highway facility analysis procedure which took intersections into consideration was developed. Because no simulation program existed at the time with the capability to model the combination of two-lane highway segments and signalized intersections, the study was accomplished through a hybrid approach, in which the results of two different simulation programs were combined. However, the accuracy of the procedure is limited by the restrictions on combining results from disparate modeling programs.

The new simulation tool now can directly model two-lane highway facilities in combination with various features (such as a combination of passing lanes, signalized intersections, etc.), and this will provide the seamless data for analyzing more complex two-lane highway facilities. An analytical methodology for analyzing two-lane highways is expected to be facility-based, accurate, efficient and flexible for the research on any type of two-lane highways.

CHAPTER 2   REVIEW OF EXISITING TWO-LANE HIGHWAY ANALYSIS METHODOLOGIES

2.1   HCM 2010

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.

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

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

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.

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

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

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

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

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.

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

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

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 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

$$PD = \frac{\sum\limits_{H,S}(D_H + D_S)}{\sum\limits_{H,S}\left(\dfrac{L_H}{FFS_H} + \dfrac{L_S}{FFS_S}\right)} \times 100 \qquad\qquad (2\text{-}7)$$

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.

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

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.

2.3   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.

2.4 Empirical Based Performance Measures

2.4.1 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

$$PI = 11.4 + 0.0012X_1 + 0.0015X_2 + 0.016X_3 + 0.879X_4 - 0.794X_5 + 0.081X_6 \qquad (2\text{-}8)$$

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.

2.4.2   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 for model development, and another 4 sites in Oregon for model validation. Based on the field data, the prediction 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

$$\text{Follower density} = \frac{\text{Number of followers}}{\text{Average follower travel speed}} \qquad (2\text{-}9)$$

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.

2.5   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 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 field data. 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 over the duration of the trip.
- **Number and duration of cruise control applications.** A cruise control emulator was developed and incorporated into CORSIM. 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.

CHAPTER 3   DEVELOPMENT OF AN ANALYTICAL METHODOLOGY FOR TWO-LANE HIGHWAY FACILITY ANALYSIS

The new simulation tool developed, as described in CMS Project 2008-002, 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.

3.1   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:

1. 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, mainline traffic may also be affected by turning vehicles even during green time. Therefore, it is essential to know the location where such effects to mainline traffic become significant. The distance between this critical location and the intersection approach stop bar is defined as upstream effective length.

2. 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.

3.2   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.

3.2.1   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 mainline. 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.

3.2.2 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.

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.

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 speeds[1] 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 3-1). 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.

Table 3-1. 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

3. D-factor

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[1] In CORSIM, the desired speed of a vehicle is a function of link free-flow speed and driver type.

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.

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 the through movements. In order to keep the mainline flow rate the same upstream and downstream of the intersection, the volumes turning onto the mainline at the intersection were set equal to the volumes turning off the mainline at the intersection.

5. Signal timing

For two-lane highway facilities, it is preferred to keep mainline 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.

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 3-2 to 3-4. 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.

Table 3-2. Experimental design for the scenarios with 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 3-3. 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 3-4. 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

3.3 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 long links[2] 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 of the signal, the TWOPAS input Entering Percent Following (*EPF*) was estimated based on Dixon et al.'s (2003) methodology[3]. 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 signal were added to a tool named VTAPE (Vehicle Trajectory Analysis for Performance Evaluation). VTAPE can process CORSIM TS0

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

[3] Dixon et al. (2003) developed a method to estimate *EPF* immediately downstream of a signal by analyzing the downstream flow profile as a function of signal timing characteristics. From this, the effect of the signal on the downstream segment can be assessed by using the relationship between *EPF* and *PTSF*.

files, which contain time step data for each simulated vehicle. More detail about the rules implemented is included in the following sections.

3.3.1 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.

1. 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 4 to 5 seconds (Figure 3-1 for example).
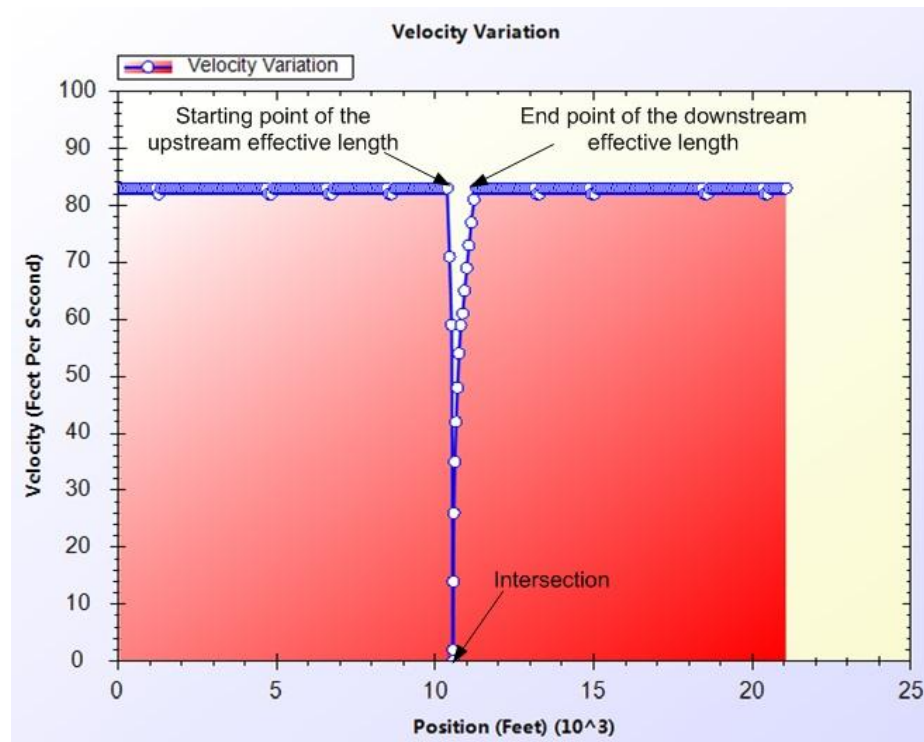
Figure 3-1. Trajectories of Type 1 upstream/downstream affected vehicle

2. 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 3-2 for example).
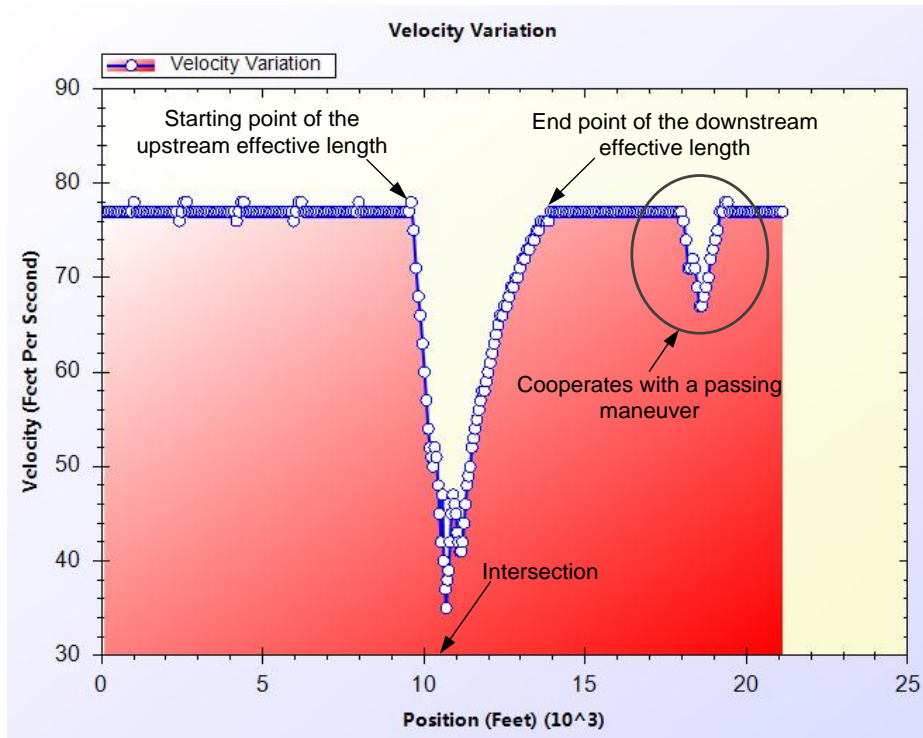
Figure 3-2. Trajectories of Type 2 upstream/downstream affected vehicle

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.

3.3.2 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 intersection 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 speed and stops accelerating for several successive seconds; 2) If a vehicle is in a following mode, it should at least reach the desired 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.

1. 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 3-1 for example).

2. 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 3-2 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 desired 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 speeds, or at driver type 1's desired speed when in a platoon led by a driver 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 speed for the turning movement, even for the vehicles with the most aggressive driver type.

- The vehicle must reach the desired 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.

3.4 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.

3.4.1 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 (using generalized notation):

$$Y = \alpha + \sum_l \sum_m \beta_{lm} X_l^k X_m^q \tag{3-1}$$

where Y is the dependent variable, α is the intercept, β_{lm}, k, and q are parameters, and X_1 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:

$$Len_{eff\_up} = 266.66 + 3.047 \times \left(\frac{v_d}{100}\right)^2 + 8.626 \times Cycle$$
$$- 0.972 \times \left(\frac{v_d}{100}\right) \times \%LT - 14.102 \times g \tag{3-2}$$

where

Len_{eff_up} = upstream effective length of signalized intersection (ft)
v_d = flow rate in the direction analyzed (pc/h)[4]
$Cycle$ = 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 3-5. From Table 3-5, it can be seen that all the explanatory variables are statistically significant at a 95% or greater 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 3-3, 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.

Table 3-5. Statistical results of the upstream effective length model with 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	

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[4]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.
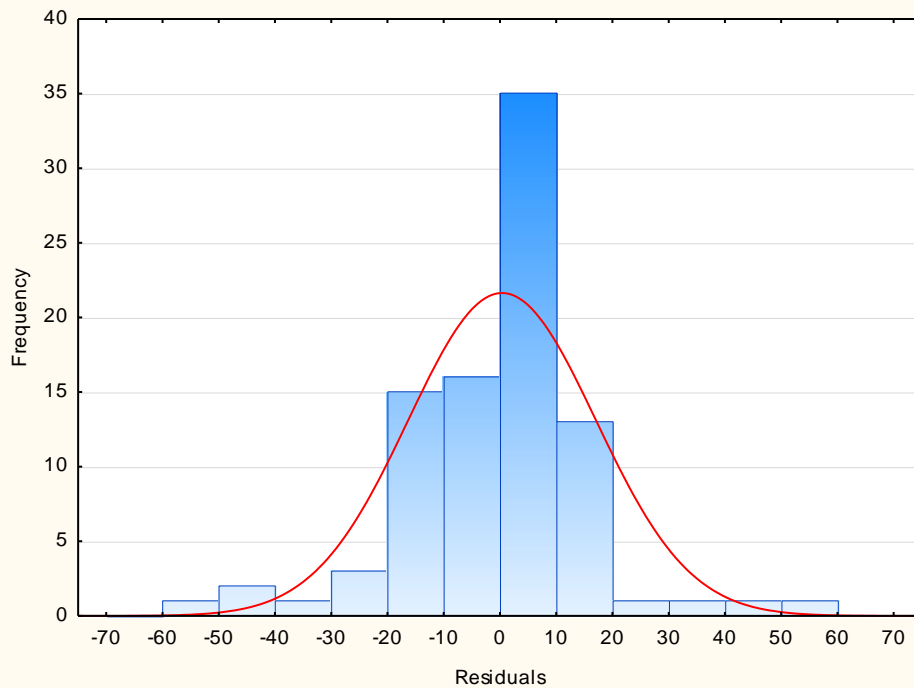
Figure 3-3. Residual histogram of the upstream effective length model with a left-turn bay

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

$$Len_{eff\_up} = 412.02 + 57.997 \times \left(\frac{v_d}{500}\right)^3 + 85.158 \times \left(\frac{v_o}{500}\right)^3 - 3.656 \times Cycle$$

$$+ 0.033 \times \left[\left(\frac{v_d}{500}\right) \times \% LT\right]^3$$

(3-3)

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 3-6. It can be seen from Table 3-6 that all the explanatory variables are statistically significant at a 95% or greater 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.

Table 3-6. 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	

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 3-4, confirm the goodness-of-fit of the model as well.
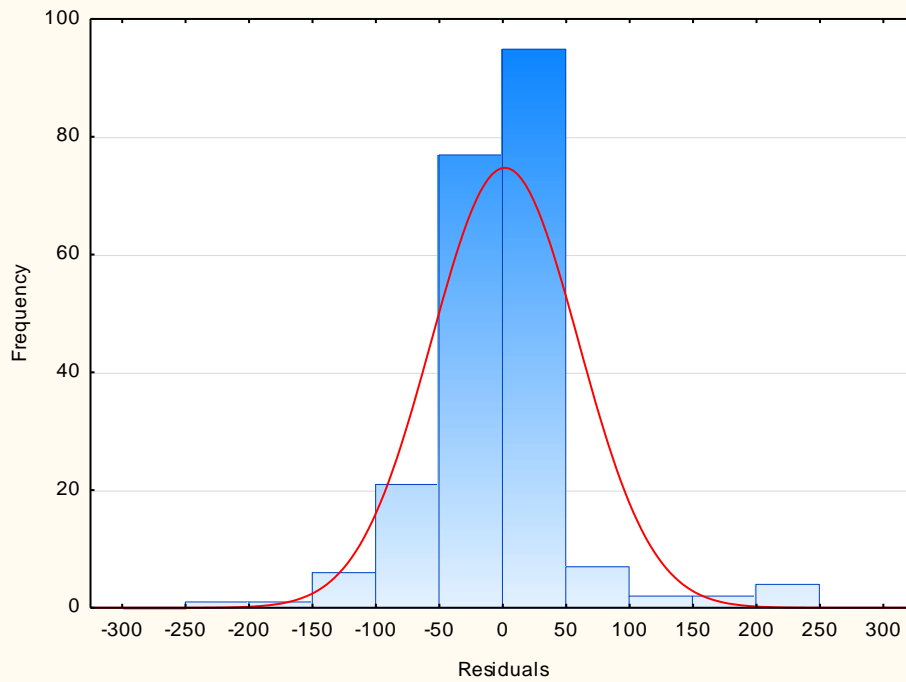
Figure 3-4. Residual histogram of the upstream effective length model w/o a left-turn bay

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 3-5).

Figure 3-5. Comparison of predicted upstream effective lengths using two estimation models

3.4.2   Downstream effective length model development

The regression model for estimating downstream effective length is in the similar form described in Equation 3-1. Initially, models wer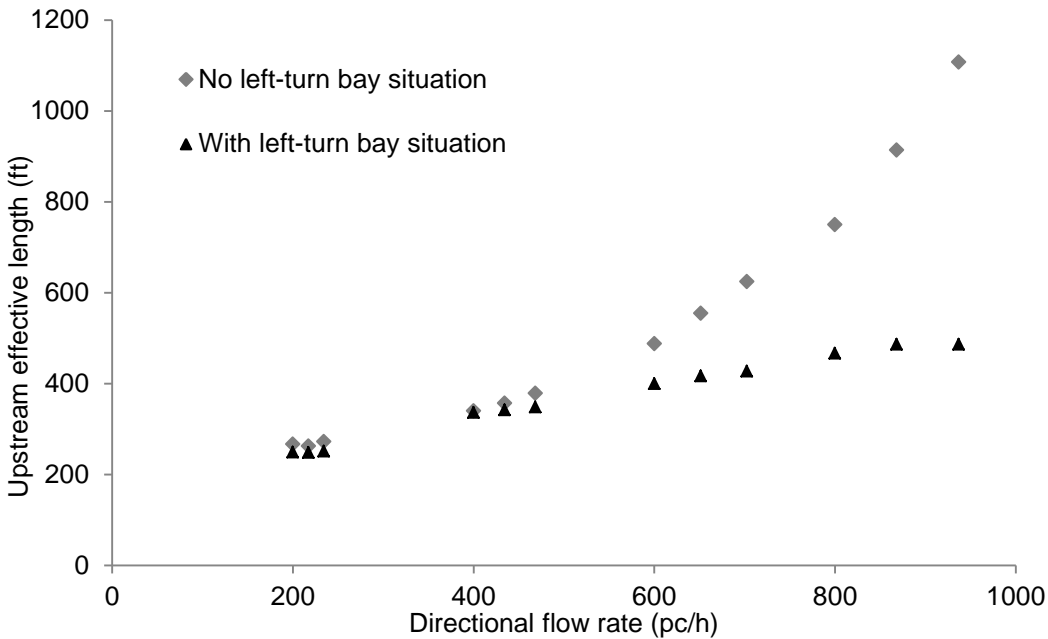e developed separately for the scenarios with a left-turn bay and the scenarios without a left-turn bay, which is similar to the upstream effective length model development. However, the experiment results showed that an upstream left-turn bay has little impact on the downstream effective length. Therefore, 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:

$$Len_{eff\_down} = 701.34 + 51.016 \times \left(\frac{V_d}{100}\right) + 42.353 \times \%HV + 13.833 \times Cycle$$

$$- 1.701 \times \left(\frac{V_d}{100}\right) \times \%LT - 16.760 \times g \tag{3-4}$$

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

$$Cycle = \text{average cycle length (s)}$$

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

g = effective green time

The statistical results are listed in Table 3-7. From Table 3-7, it can be seen that all the explanatory variables are statistically significant at a 95% or greater 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 3-6, confirm the goodness-of-fit of the model as well.

Table 3-7. Statistical results of the downstream effective length model

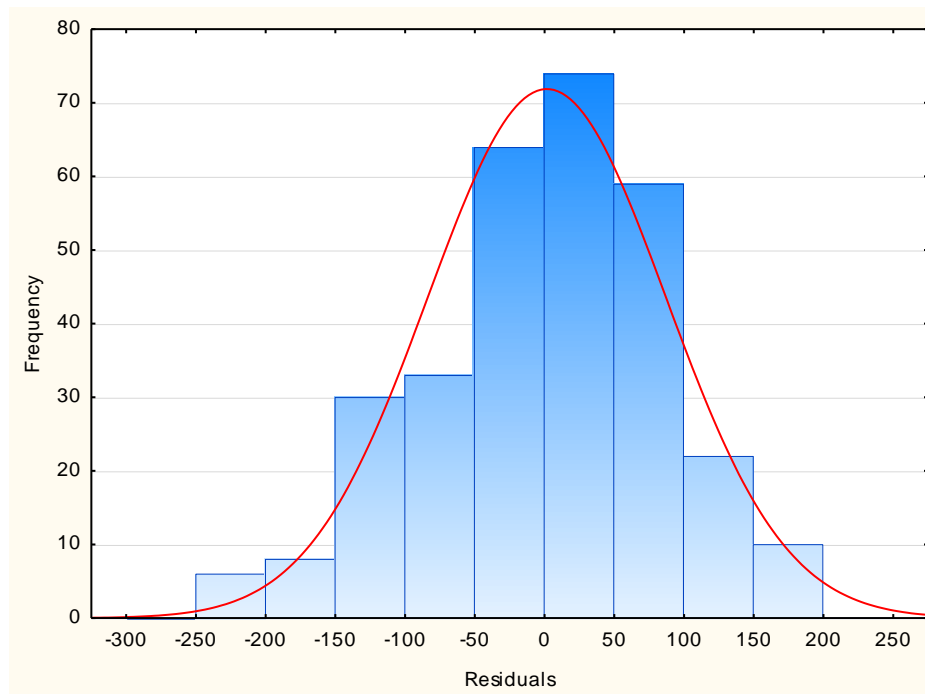| Explanatory variables	Parameter	t-value	Adjusted R^2
Constant	701.3438	24.2451	
$(V/100)$	51.0164	7.9484	
%HV	42.3531	41.8964	0.9014
Cycle	13.8329	3.9211	
$(V/100) \times \%LT$	-1.7009	-5.0018	
g	-16.7603	-2.7970	

Figure 3-6. Residual histogram of the downstream effective length model

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.

3.5   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* was previously incorporated into CORSIM as an output for two-lane highway links as part of the previous CMS project (Washburn and Li, 2010).

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, follower density certainly has appeal for being the primary service measure for uninterrupted two-lane highways, and even as a secondary service measure for the evaluation of two-lane highways with occasional signalized intersections. CORSIM is now able to 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 3-4). 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.

1. *ATS* vs. flow rate

Figure 3-7 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 (see Washburn and Li, 2010, for these results). 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.
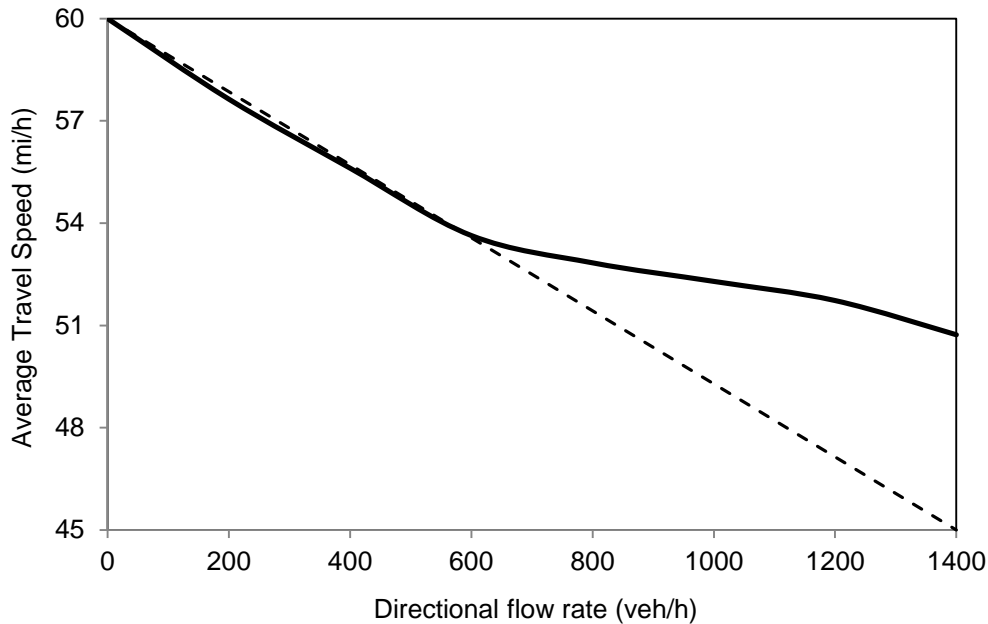
Figure 3-7. Directional speed-flow results

2. *PTSF* vs. flow rate

Figure 3-8 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 (see Washburn and Li, 2010, for these results). 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.
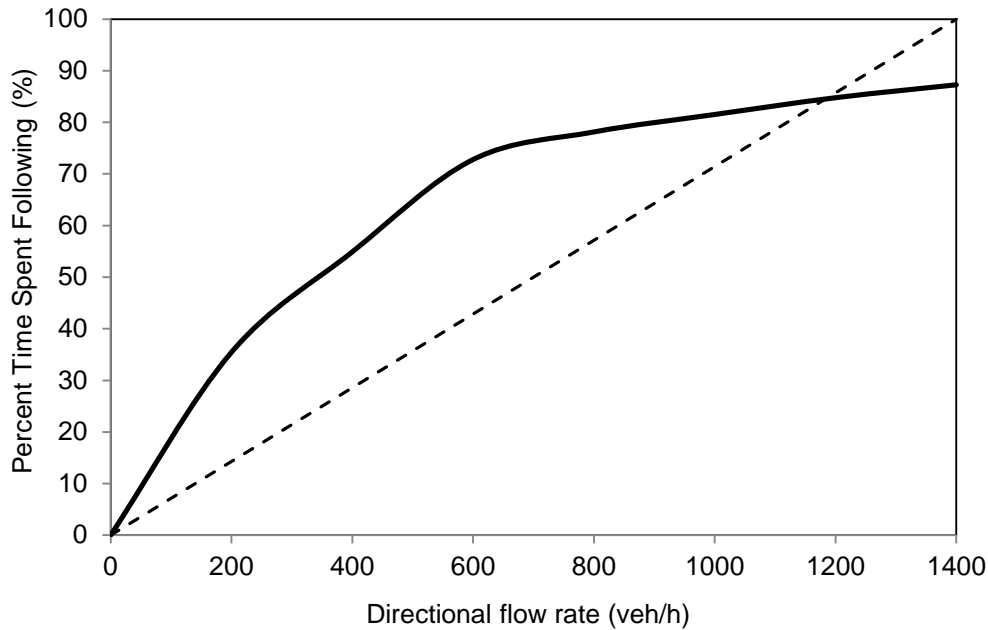
Figure 3-8. Directional *PTSF*-flow results

3. *PD* vs. flow rate

Figure 3-9 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.
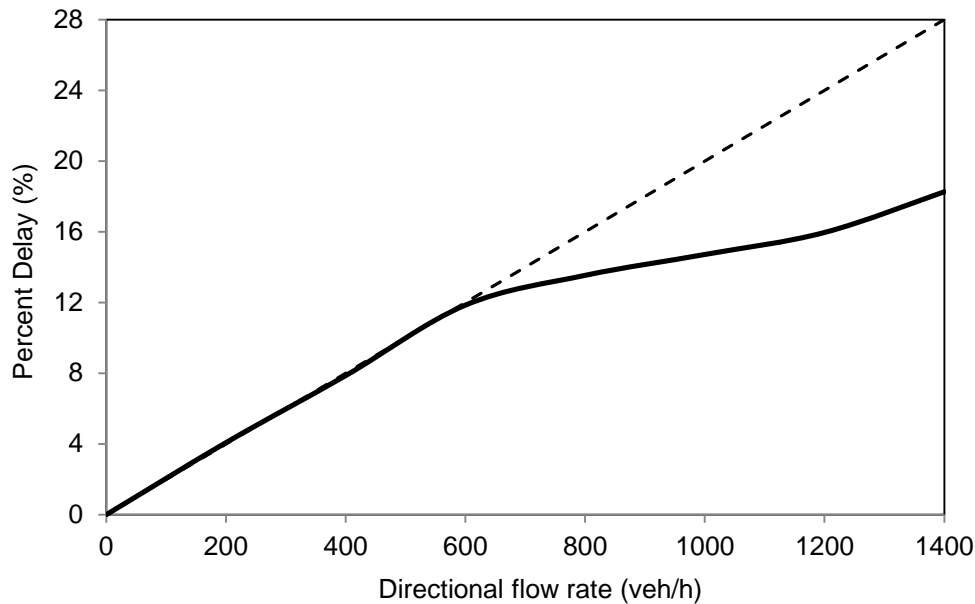
Figure 3-9. Directional *PD*-flow results

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 3-9) 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.

4. Follower density vs. flow rate

Figure 4-10 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.
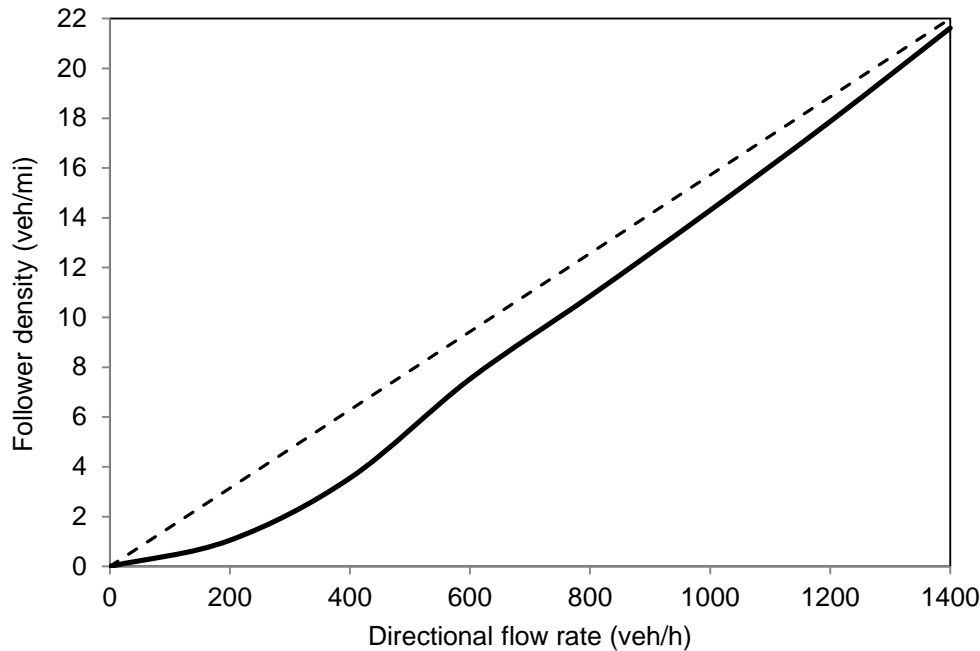
Figure 3-10. Directional follower density-flow results

The relationship between follower density and flow rate is closest to a linear relationship according to Figure 3-10. 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.

5. *PFFS* vs. flow rate

Figure 3-11 illustrates the relationship between directional *PFFS* and directional volume. The *PFFS*-flow curve shares the similar trend with the *ATS*-flow curve (Figure 3-7) 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*.
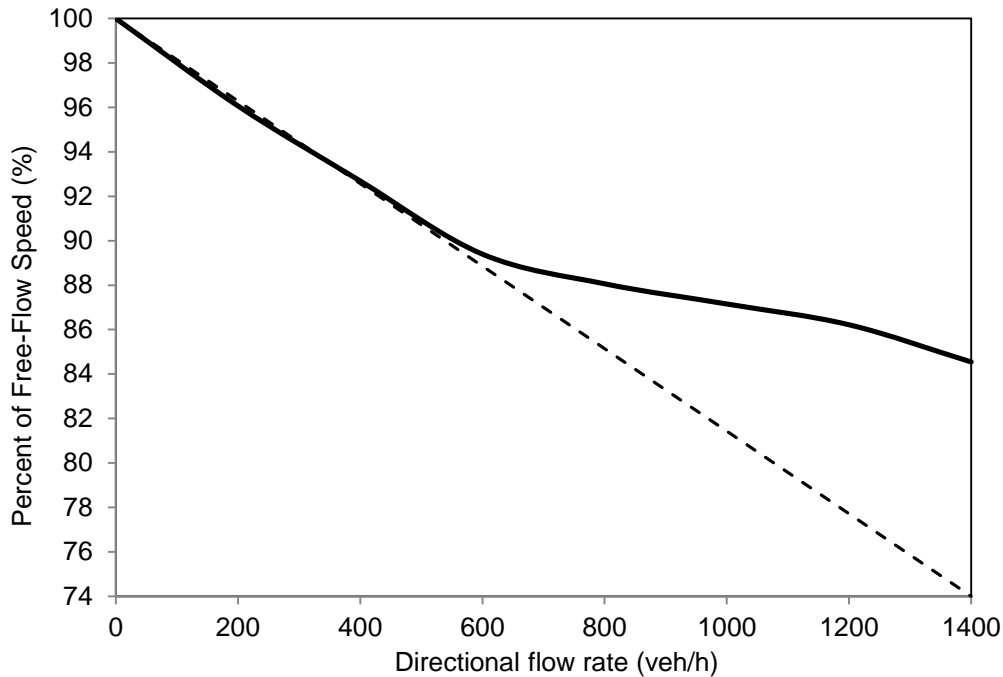
Figure 3-11. Directional *PFFS*-flow results

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.

3.6   Overall Evaluation Methodology

The evaluation methodology uses the same idea as used in 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.

3.6.1   Evaluation methodology

The vehicle trajectory approach for determining upstream and downstream effective length is in fact consistent with the definition of control delay (Figure 3-12). 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.
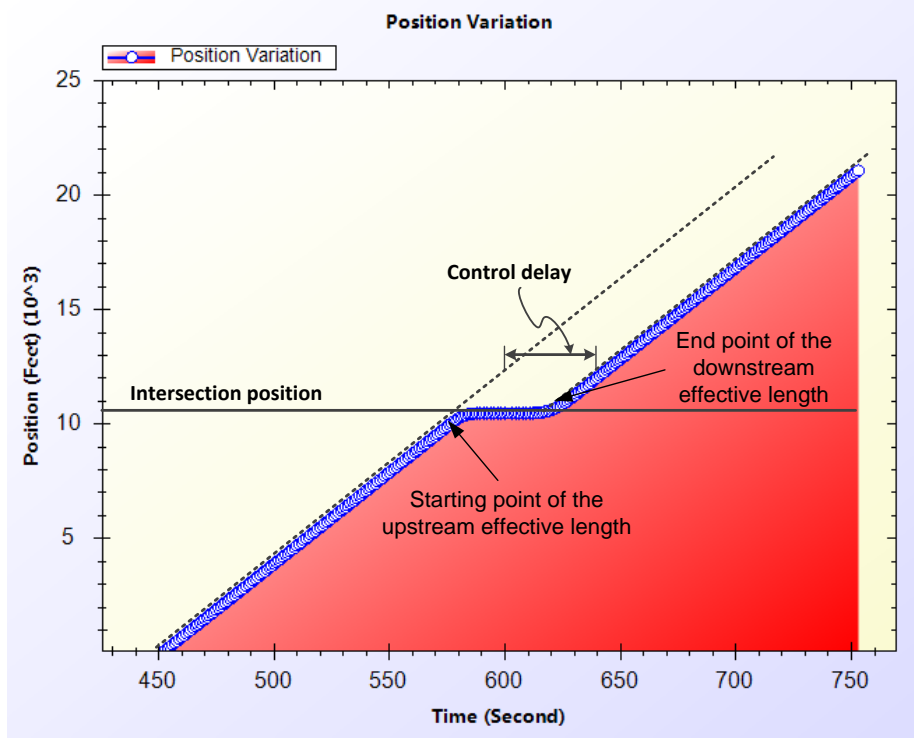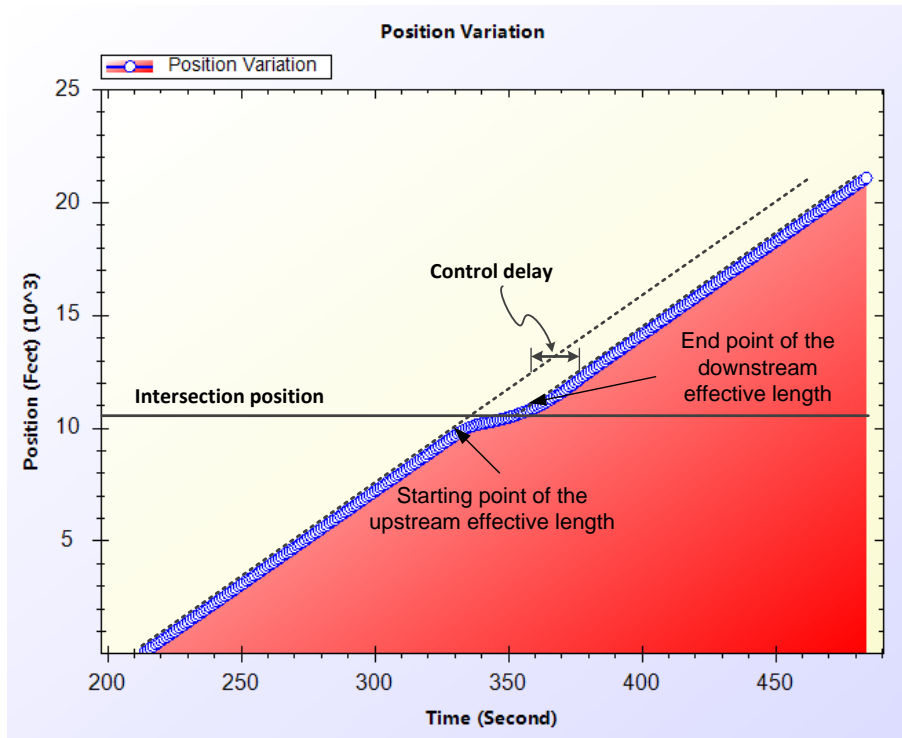
Figure 3-12. Trajectory of delayed vehicle on time-space plane. A) Delayed vehicle with a complete stop at intersection B) Delayed vehicle without a complete stop at intersection C) Delayed vehicle with a complete stop at intersection and a relatively long upstream/downstream effective length
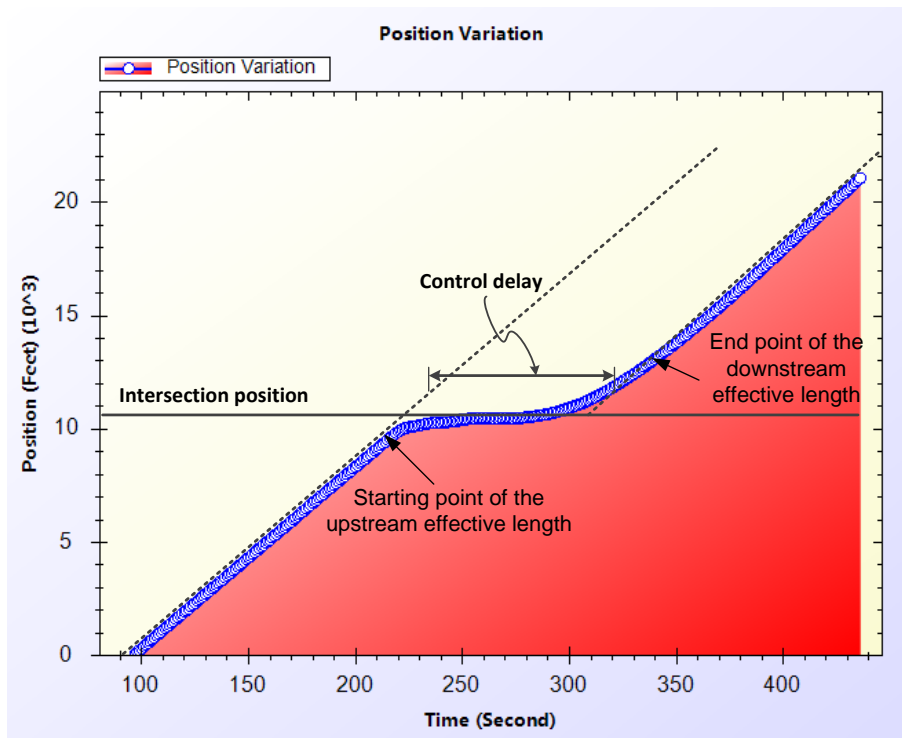
B

C

Figure 3-12. Continued

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.

3.6.2   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 3-13) 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.
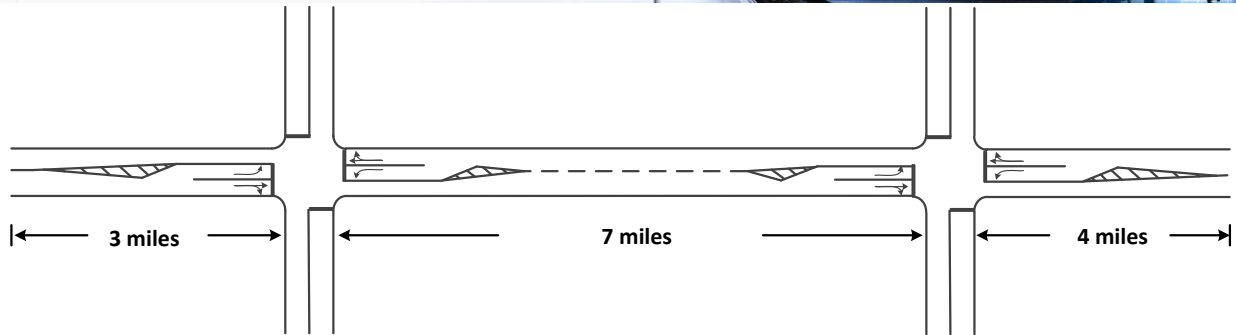
Figure 3-13. Illustration of a two-lane highway facility with two signalized intersections

From the comparison results in Table 3-8, 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 3-14[5]). 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*.

Table 3-8. Comparison results for validation

Directional flow rate (pc/h)	Percent delay (%)		
	CORSIM output	*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

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[5]The HCM *ATS* results in Figure 3-14 were calculated by Equation 15-6 in the HCM (TRB, 2010) (refer to Equation 2-1 in this document).
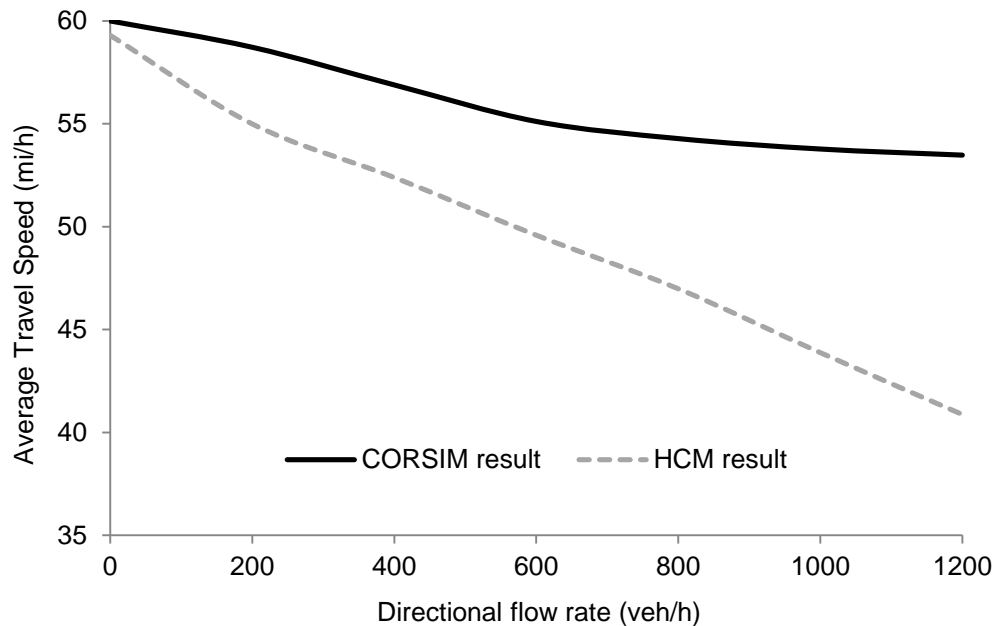
Figure 3-14. Comparison of CORSIM *ATS* and HCM *ATS*

3.6.3 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 3-15 to 3-18 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.
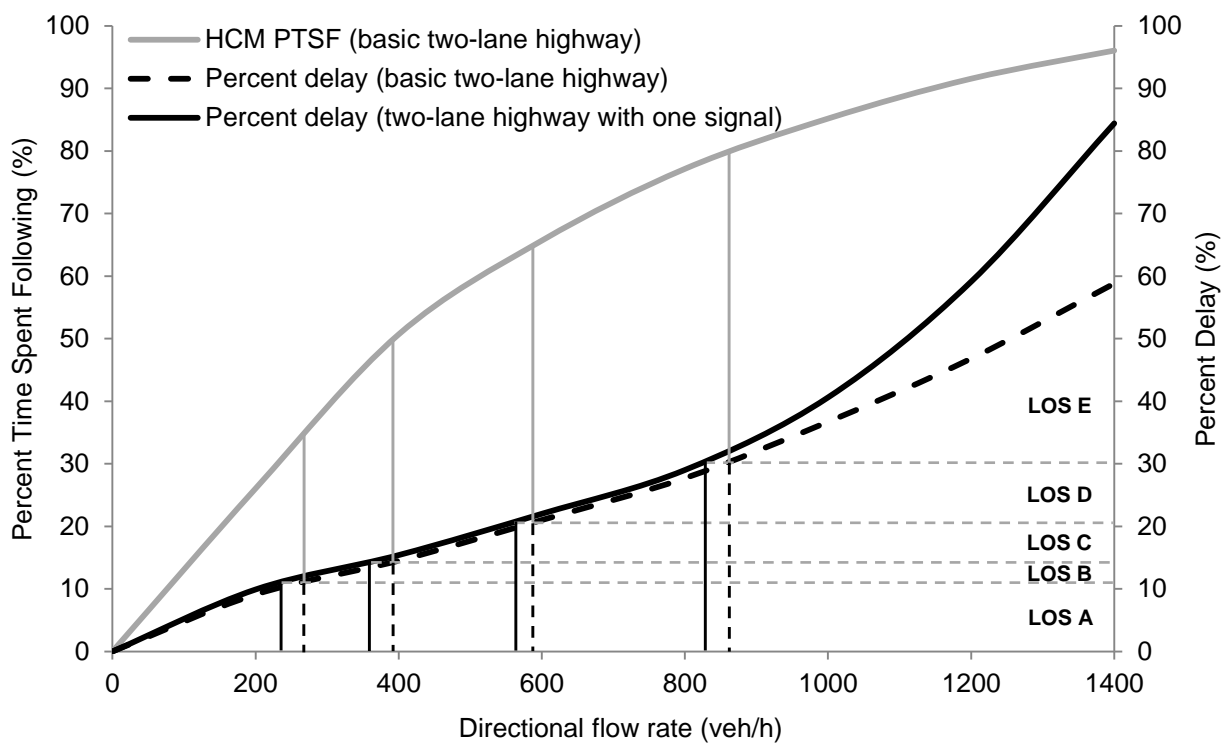

Figure 3-15. *PD* LOS criteria based on *PTSF* LOS criteria for Class I two-lane highways
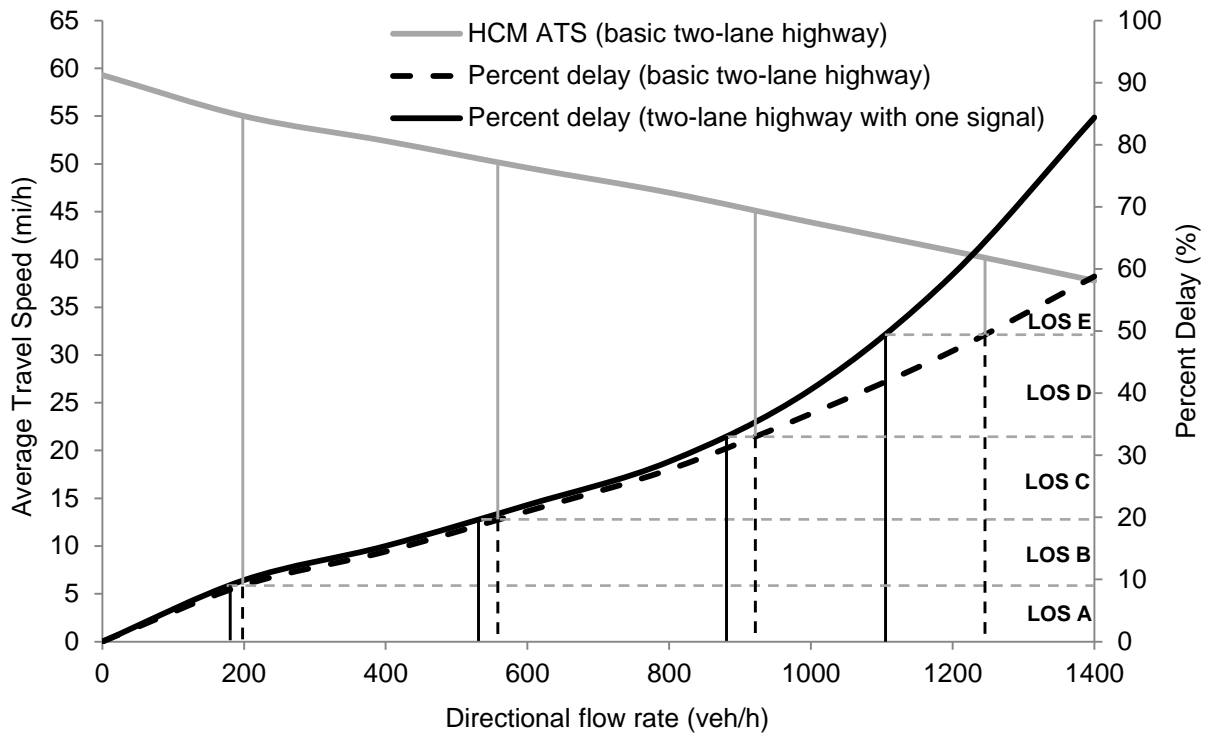
Figure 3-16. *PD* LOS criteria based on *ATS* LOS criteria for Class I two-lane highways
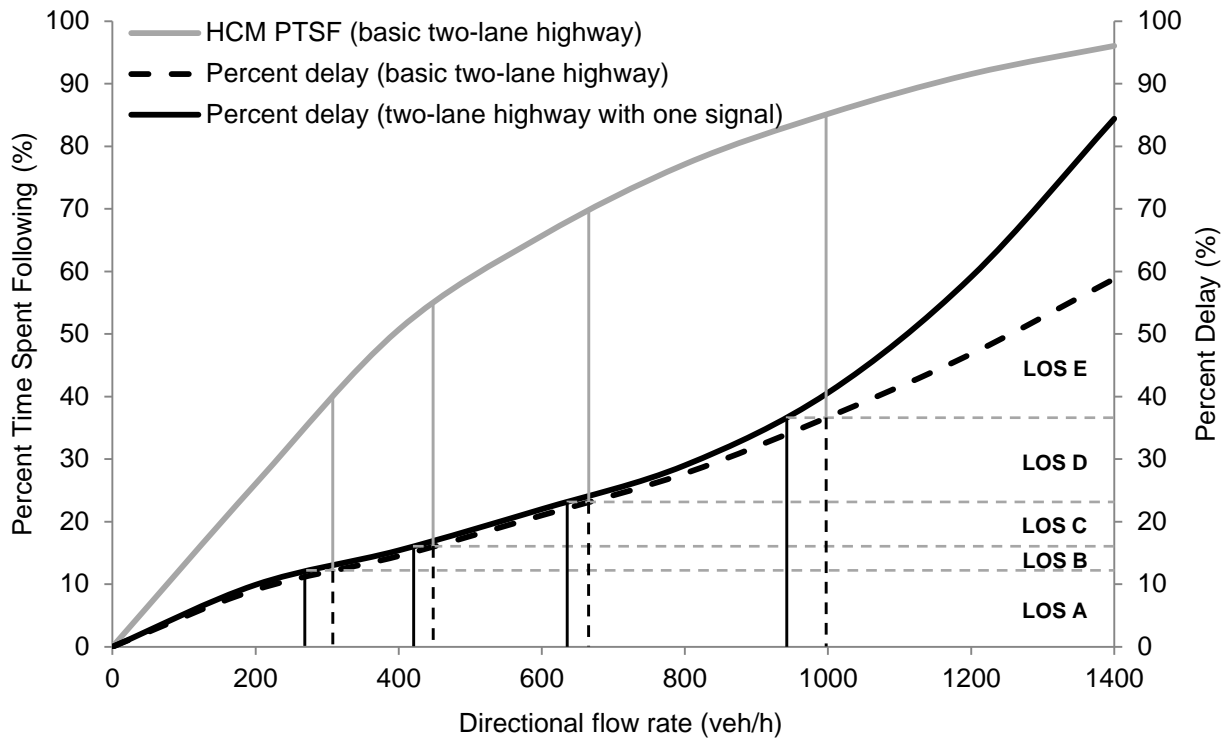
Figure 3-17. *PD* LOS criteria based on *PTSF* LOS criteria for Class II two-lane highways
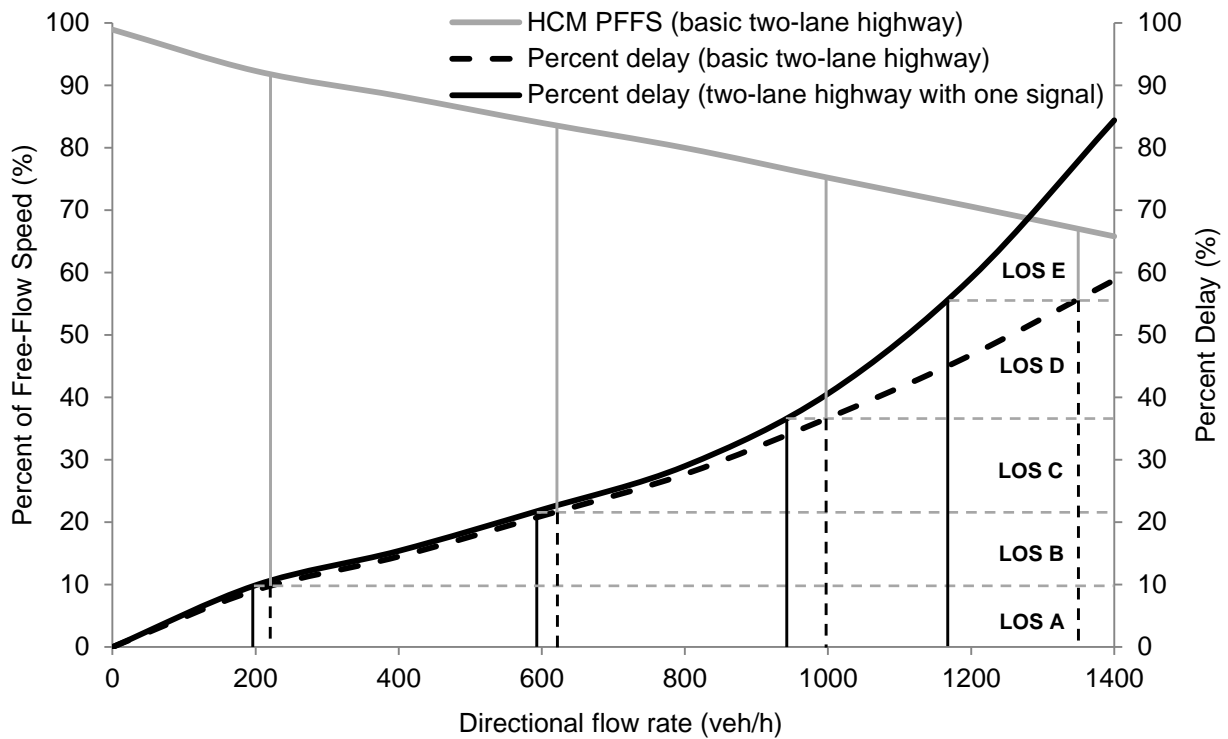
Figure 3-18. *PD* LOS criteria based on *PFFS* LOS criteria for Class III two-lane highways

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 3-9.

Table 3-9. 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

3.7 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.

3.7.1 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 3-19A). Otherwise, the overlapped part will be considered as a part of the upstream effective segment without special treatment (Figure 3-19B).
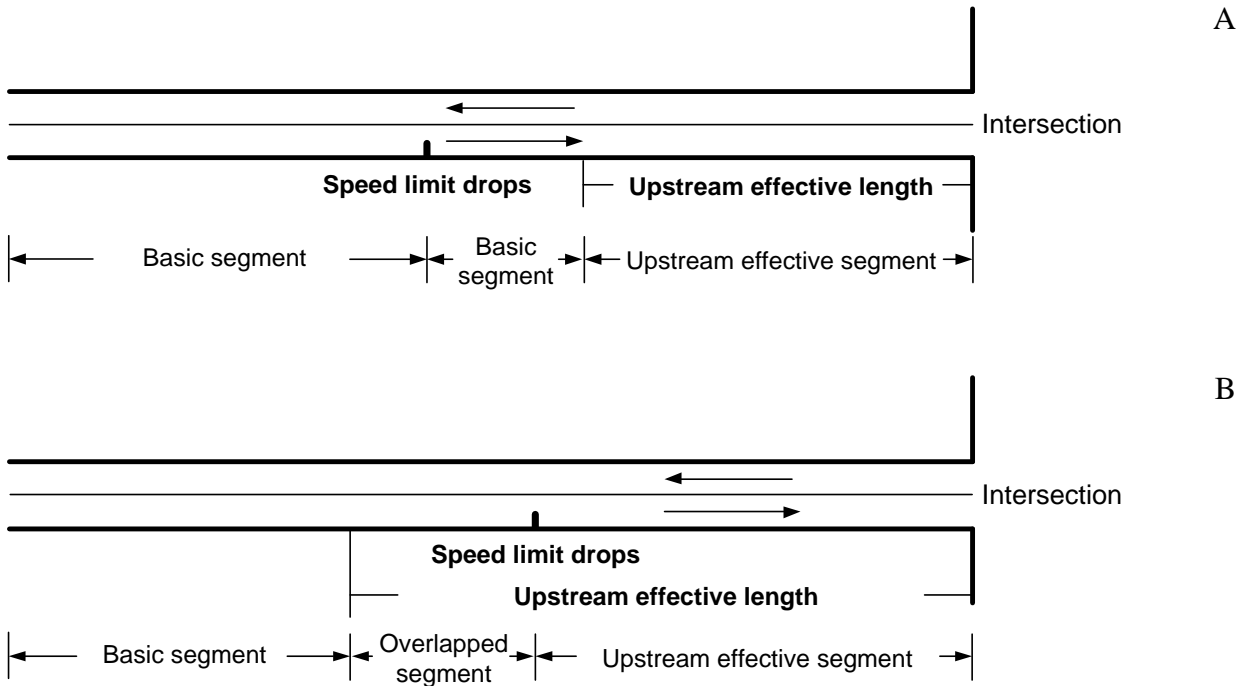
Figure 3-19. Illustration of facility segmentation. A) Speed limit drops before the starting point of upstream effective length B) Speed limit drops after the starting point of upstream effective length

3.7.2 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 3-2. 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 3-4, as follows:

$$Len_{eff\_down} = 701.34 + 51.016 \times \left(\frac{V_d}{100}\right) + 42.353 \times \%HV + 13.833 \times Cycle$$

$$- 1.701 \times \left(\frac{V_d}{100}\right) \times \%LT - 16.760 \times g$$

$$Len_{eff\_down} = 701.34 + 51.016 \times \left(\frac{800}{100}\right) + 42.353 \times 0 + 13.833 \times 57.3$$

$$- 1.701 \times \left(\frac{800}{100}\right) \times 5 - 16.760 \times 33.8$$

$$= 1267.6\,(\text{ft})$$

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 Lup_1:

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

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

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

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.

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

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

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

Therefore, the total delay can be calculated as:

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

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

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

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 3-9. By applying the criteria, it is indicated that the analysis direction operates at LOS C.

3.8   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.

1. 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 have not been accounted for. 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.

2. 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.

3. 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, and are more accurate. 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.

CHAPTER 4   COMPARISON OF THE CORSIM AND HCM 2010 TWO-LANE HIGHWAY TRAFFIC FLOW RELATIONSHIPS

With the recently enhanced version of CORSIM, that is, incorporating the ability to model two-lane highway operations, a natural question arises as to how the performance measures estimated from CORSIM compare to those estimated by the HCM 2010 two-lane highway analysis methodology. This chapter provides an extensive comparison of Percent Time-Spent-Following (*PTSF*) and Average Travel Speed (*ATS*), the two primary performance measures used in the HCM analysis methodology, as estimated from the HCM and CORSIM, including the speed-flow relationship and the *PTSF*-flow relationship. Guidance is provided for setting up corresponding networks between CORSIM and the HCM.

4.1   Experimental Design for CORSIM and 2010 HCM Comparison

In order to compare the results between the 2010 HCM and CORSIM, it is important to have a comparable set of inputs. Table 4-1 shows an example of common inputs for the HCM and CORSIM and also shows which inputs are not applicable for each method. The inputs in Table 4-1 must have the same values where applicable in CORSIM and the HCM in order to ensure that the conditions are as similar as possible for each of the corresponding HCM and CORSIM testing scenarios.

There are several factors that are inputs in CORSIM, but not in the HCM, that could cause majors differences in the results. These factors include existing upstream conditions, truck type distribution, and passing zone configuration. A facility with a changing grade configuration could also have major effects on the results as the HCM does not have an input for specifying where the grade changes. However, all testing scenarios in this chapter have a constant grade along the entire facility, which eliminates uncertainties with changing grades. Several preliminary tests were run to see which CORSIM inputs for these factors should be used to give similar results to the HCM. These testing procedures are described in the next section.

4.1.1   Preliminary experiments

There are several inputs required for the HCM that are not applicable in CORSIM and conversely as shown in Table 4-1. The inputs that are not applicable in CORSIM are programmed internally in the algorithms. There are certain factors that have an impact on CORSIM results that are not mentioned in the HCM such as existing conditions upstream of the facility, truck type distribution, and passing zone configurations. Before any major comparison between CORSIM and the HCM was made, preliminary tests for existing conditions upstream of the facility, truck type distribution, and passing lane configuration were performed. Also, a preliminary test was done that compared the speed vs. flow rate relationship and *PTSF* vs. flow rate between CORSIM and the HCM. The results of these tests were used as guidance for matching the CORSIM inputs as closely as possible to the HCM inputs.

Table 4-1. Example inputs for HCM and CORSIM

Inputs	HCM	CORSIM
Geometric data		
facility length (mi)	10	10
lane width (ft)	12	N/A
shoulder width (ft)	6	N/A
access point density (points/mi)	0	N/A
grade (%)	0	0
radius of curvature (ft)	N/A	0
superelevation (%)	N/A	0
percentage of no-passing zones (%)	0	N/A
passing lane length (ft) (if applicable)	5280	5280
highway class	1	N/A
Demand data		
length of analysis period (h)	1	1
PHF	1	N/A
base FFS (mi/h)	60	N/A
FFS (mi/h)	65	65
heavy vehicle percentage (%)	0	0
directional split	50/50	N/A
two-way flow rate (veh/h)	1200	N/A
eastbound flow rate (veh/h)	N/A	600
westbound flow rate (veh/h)	N/A	600

Adapted from Transportation Research Board (TRB). Highway Capacity Manual. TRB, National Research Council, Washington D.C. 2010.

Three main preliminary tests were done in CORSIM for guidance on how to properly compare the 2010 HCM with CORSIM. The CORSIM values that best reflected what the HCM most likely assumes were chosen as the inputs for the main CORSIM and HCM comparison after analyzing the results of these preliminary tests.

1. Existing Upstream Conditions

When analyzing a facility in the HCM, there are preexisting conditions upstream of the facility such as platoon structure on some unknown length of roadway. The HCM was based on TWOPAS simulation where one of the inputs was percentage of traffic flow entering as platoons. This established the incoming traffic conditions for the analysis facility. In CORSIM, vehicles are generated from an entry node and no upstream conditions have been established. This has a large impact on the entering platoon structure, especially for short facilities with low traffic volumes because the vehicles are spaced out when they are generated and can possibly move through the entire facility before coming near other vehicles. This could lead to an unrealistically low *PTSF*. In order to have accurate results, there needs to be a lead up length to the facility that allows platoons to form and incites vehicles to interact prior to the section of roadway for which data will be collected. There should also be a follow up length that is identical in distance to the

lead up length in an effort to have similar traffic conditions coming from both the eastbound and westbound directions. The results of the analysis for any facility should then only be extracted from the links that make up the analysis facility and not from the lead up or follow up segments. Three different facility lengths were tested with varying lead up lengths in order to determine what the lead up length should be and how the performance measures were affected. The facilities have lengths of three, five, and ten miles. Each facility was tested with different lead up lengths ranging from one to six miles. The three facilities were tested with passing allowed and passing not allowed. Based on the analysis of the test results, a lead up length of five miles was chosen because it led to a small difference in the *PTSF* results for both passing allowed and passing not allowed. This allowed the five-mile lead up segment to be used for all passing zone scenarios. Five miles was also chosen as the follow up segment length in order to maintain symmetry in the facility.

2. Truck Type Distribution

The HCM has an input for the percentage of trucks, but does not allow the analyst to specify what types of trucks make up that percentage. CORSIM has an input for the percentage of trucks and also allows the analyst to specify which types of trucks make up that percentage. The truck types range from 3 to 6. Type 3 indicates single unit trucks that are 35 ft long. Type 4 trucks have a medium-sized load and are 53 ft in length. Type 5 trucks are fully-loaded with a length of 53 ft, and type 6 trucks are 64 ft long double-bottom trailers (Table 3-1).

Trucks in the traffic stream have a large impact on *ATS*. Seven different truck type splits were tested and each scenario that had some percentage of type 6 trucks had results that were considerably different from the scenarios with no type 6 trucks. All seven tests either underestimated or overestimated the HCM results for speed. The HCM shows a linear relationship between speed and flow while the CORSIM truck type tests all showed a curve shape that initially has a fairly steep negative slope for low flow rates and then the curve slope decreases to the point where it is fairly flat over the moderate to high flow rates, which supports Luttinen's (2000) and Brilon and Weiser's (2006) findings about the speed-flow relationship on two-lane highways. Ultimately, the truck percentage split of 50/25/25 for Type 3, 4 and 5 trucks was chosen to be used for the major comparison of the HCM and CORSIM because it has the most realistic distribution of truck types and matched better than the other tests.

3. Passing zone configuration

The HCM allows the user to specify that the facility has some percentage of no-passing zones. However, CORSIM specifies passing zones on a link by link basis. In CORSIM, the user can select exactly where the no-passing zones are located. The HCM does not allow the user to select where the no-passing zones are along the facility. Therefore, even when both methods have the same percentage of no-passing zones, the configuration is ambiguous. This problem could potentially lead to differences in the performance measure results.

Since the results for *PTSF* and *ATS* were similar between all three testing facilities (Table 4-2), facility A was chosen as the 50% no-passing zone configuration for the major HCM and CORSIM comparison. Facility A was chosen because it is the simplest of the three facilities with

the first five miles being specified as a no-passing zone and the last five miles being specified as a passing zone. Any of the facilities would have been adequate for the major comparison since the passing zone configuration does not have an impact on the average of the performance measure results.

Table 4-2. Scenarios for CORSIM no-passing zone configuration test

Facility A		Facility B		Facility C	
Link length (mi)	Passing allowed	Link length (mi)	Passing allowed	Link length (mi)	Passing allowed
5	N	2.5	N	1	N
5	Y	2.5	Y	1	Y
		2.5	N	1	N
		2.5	Y	1	Y
				1	N
				1	Y
				1	N
				1	Y
				1	N
				1	Y

4.1.2   CORSIM/HCM experimental design

The procedure that was chosen for comparing the 2010 HCM to CORSIM was to use the same inputs for both tools, to the extent possible, and compare the outputs. A wide range of inputs was used to analyze how these two methods compared to each other for a variety of situations. A variety of combinations was used to see if there was a certain scenario that showed drastically different results from the other scenarios. This experiment involves variations of the directional volumes, heavy vehicle percentages, grades, no-passing zone percentages, and the presence or absence of a passing lane. The input values that were selected for the comparison are described as following.

1. Flow rates and splits
        Six different two-way flow rates were used in this experiment in order to capture possible differences with low, medium, and high volumes. The values used were 200, 700, 1200, 1700, 2200, and 3200 veh/h. These six volumes were each analyzed under three different splits, 50/50, 60/40, and 70/30. The base conditions were 0% heavy vehicles, 0% no-passing zones, and 0% grade.

2. Percent heavy vehicles
        Two different percentages of heavy vehicles were used in this experiment in order to analyze the changes in performance measures due to heavy vehicles and to analyze the effects of heavy vehicles on attempted passes. The heavy vehicle percentages that were used were 0% and 10%. These values were chosen because the performance measures are expected to show a

noticeable change for an increase from no trucks to 10% trucks in the traffic stream. All heavy vehicles are assumed to be trucks and not recreational vehicles. The base conditions were modified to incorporate the increased truck percentage in the traffic stream. The truck type distribution is based on the results of the preliminary truck type distribution test.

3. Grades

Two different grades were used in this experiment because upgrades can have a large impact on truck speeds and this variation in grade is expected to have a large effect on the number of platoons, attempted passes, and following vehicles. The values that were chosen for grade were 0% and 6%. The slope of 6% was added as a test variable by building on the previous cases, which included the base conditions and the cases that incorporated the percentage of heavy vehicles.

4. Percent no-passing zones

The percentage of no-passing zones is another variable that affects two-lane highway performance measures. The 2010 HCM and CORSIM have different methods for inputting the percentage of no-passing zones. The differences between these two tools make it impossible to ensure that the two-lane highway analyzed in the 2010 HCM is the same as the two-lane highway created in CORSIM.

In the 2010 HCM, the percentage of no-passing zones is accounted for in the percent no-passing zone adjustment factor, f_{np}, which is different for *ATS* and *PTSF* calculations. However, the 2010 HCM does not specify which section of the highway is a no-passing zone. It only specifies that, somewhere along the highway, a certain percentage is a no-passing zone.

CORSIM does not include a direct input for the percentage of no-passing zones. Instead, the user specifies what the center-line striping condition is for each link (passing allowed in one-direction, passing allowed in both directions, or passing not allowed in either direction). For example, to specify a 50% no-passing zone on a ten-mile long highway segment with ten one-mile links, the user could select no passing allowed on the first five links and passing allowed on the last five links. The user could choose any five links to allow no passing and it would still be a 50% no-passing zone. The advantage in CORSIM is that the user can chose the passing zone configuration or where the passing zone section is located along the highway. The preliminary test for passing configuration was also used to compare the two tools' sensitivity to this factor.

The other passing zone scenarios that were tested in this experiment, besides 0%, were 50% and 100% no-passing zones. These changes were made in the existing files of the previously-tested cases, which all consisted of 0% no-passing zones. In CORSIM, the segments that were changed from 0% no-passing zones were the ones closest to the eastbound entry node. For example, the 50% no-passing zone case would have five miles of no-passing allowed closest to the eastbound entry node and would continue with five miles of passing allowed. Links with passing allowed in one direction were not tested in this study.

5. Passing lanes

   This experiment also tested the effects of passing lanes on two-lane highway performance measures. All the scenarios described previously had no passing lanes. Changes were made to those scenarios to incorporate a passing lane with a length of 5280 feet.
There are 432 different trials based on all of the different combinations of four volumes, three directional splits, three percentages of heavy vehicles, three percentages of no-passing zones, two grades, and two passing lane length scenarios. Table 4-3 shows a summary of all the variable combinations being used for this study.

Table 4-3. Variables used in HCM and CORSIM testing

Flow rates (veh/h)	Splits	% HV	% NPZ	% Grade	Passing lane length (ft)
200	50/50	0	0	0	0
700	60/40	10	50	6	5280
1200	70/30	-	100	-	-
1700	-	-	-	-	-
2200	-	-	-	-	-
3200	-	-	-	-	-

The 2010 HCM methodology was programmed into a numerical calculations worksheet and every scenario was analyzed. The numerical calculations worksheet allowed quick and easy changes to be made to the inputs for each testing scenario.

4.1.3   CORSIM testing facility

The CORSIM two-lane highway testing facility used in this experiment is ten miles long. The peak hour demand volume for the analysis direction is generated from node 8100 and travels eastbound. The opposing demand volume is generated from node 8200 and travels westbound. The passing lane scenarios have one passing lane that is one mile in length. The passing lane runs along the eastbound direction of the highway and is the fifth link away from node 8100. There are four one-mile upstream links and five one-mile downstream links. Figure 4-1 shows the CORSIM two-lane highway schematic. Ten runs were executed in CORSIM for each testing scenario and the average *PTSF* and *ATS* for the ten runs was recorded.
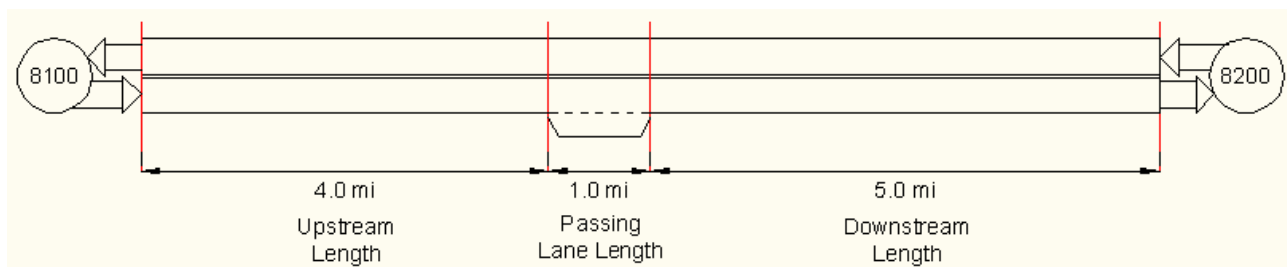
Figure 4-1. Two-lane highway facility in CORSIM with passing lane

4.2   CORSIM and 2010 HCM Comparison Results

Each combination of variables described in Table 4-3 was tested in CORSIM and the 2010 HCM. The *PTSF* and *ATS* were plotted against two-way flow rate for the three different splits to allow for easy comparison of the HCM and CORSIM. This process was repeated for all three passing-zone cases, both grade cases, and both percentages of heavy vehicles for scenarios with no passing lane. For the cases with passing lanes, performance measures were plotted against facility length for the 60/40 split only. The results of this test are discussed in this section.

4.2.1   0% no-passing zone with 0% grade test results

The 2010 HCM and CORSIM followed the same increasing trend for the *PTSF* plots for both 0% and 10% heavy vehicles as shown in Figure 4-2 and Figure 4-3. When the two-way flow rate reached 3200 veh/h, the HCM showed a *PTSF* estimate of 98.6% for the 70/30 split while CORSIM showed a *PTSF* estimate of 91.1%. It is unrealistic that the *PTSF* would be nearly 100% because the traffic stream always breaks up into several platoons due to slow vehicles. Figure 4-4 shows the platoon structure for a two-way flow rate of 3200 veh/h under a 70/30 directional split. Although the directional flow rate is 2240 veh/h, there are still some gaps between vehicles.
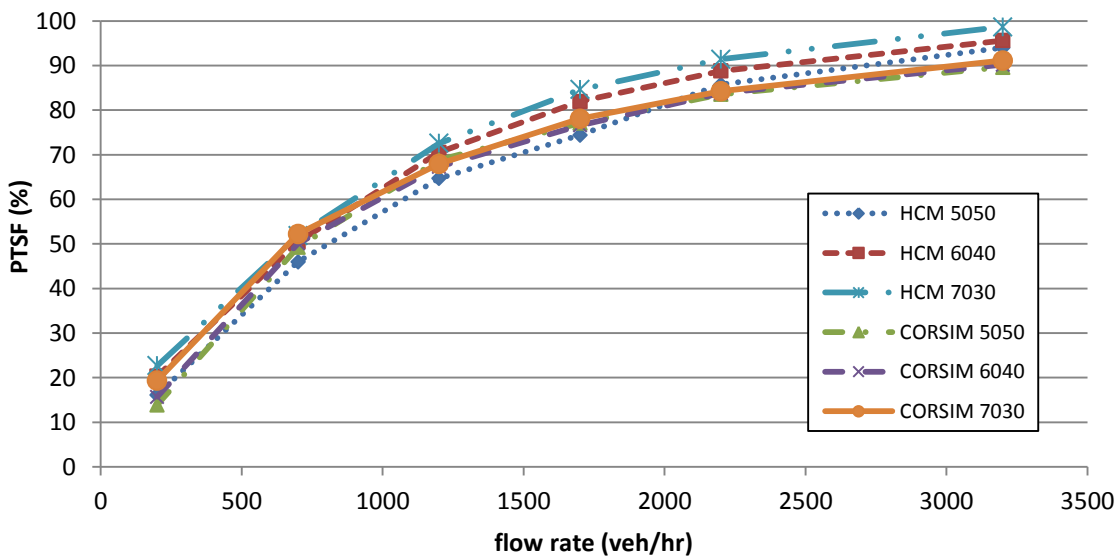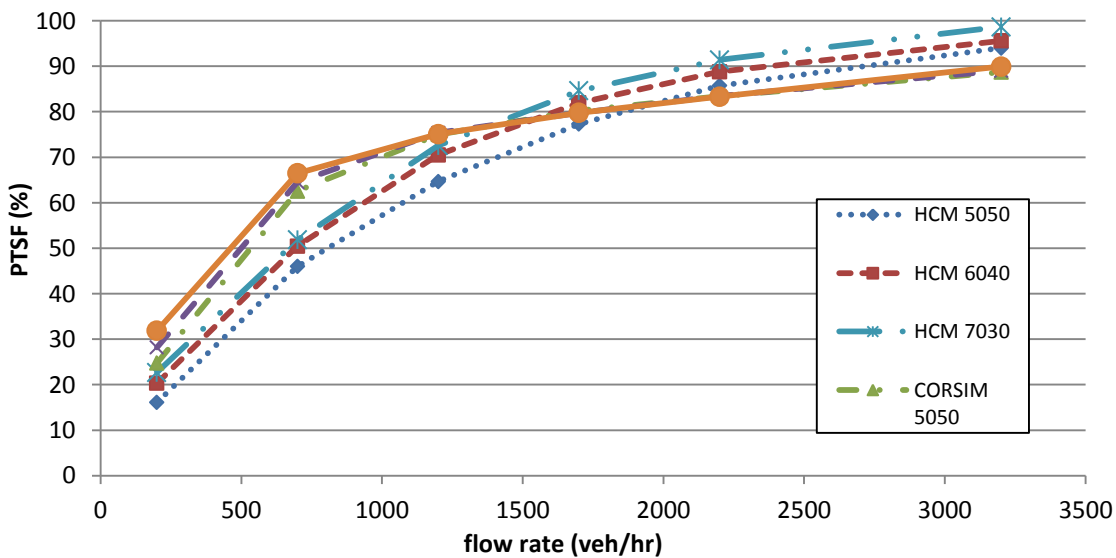

Figure 4-2. *PTSF* vs. two-way flow rate - 0% grade, 0% NPZ, 0% HV, no passing lane
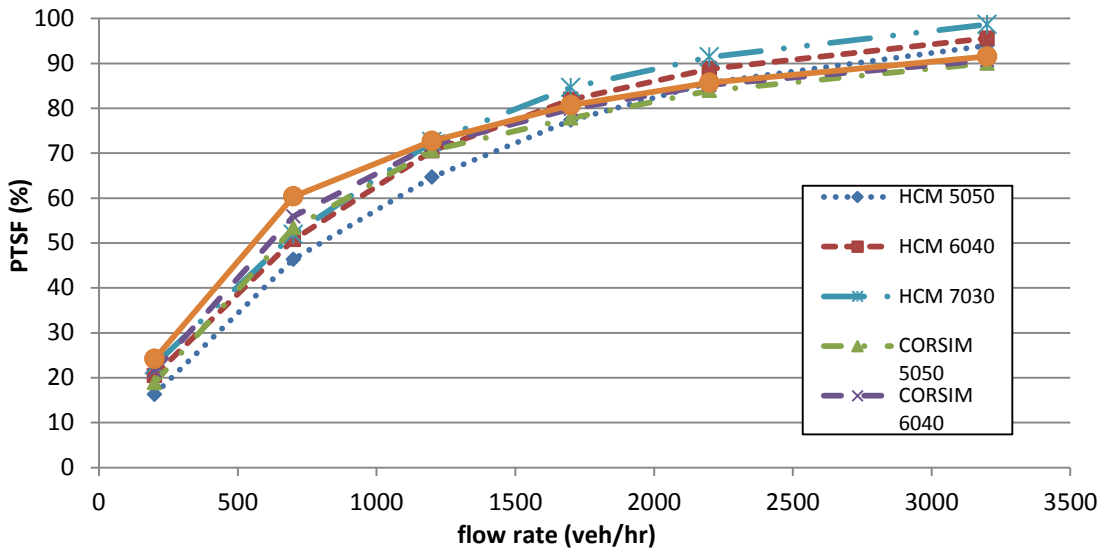
Figure 4-3. *PTSF* vs. two-way flow rate - 0% grade, 0%NPZ, 10%HV, no passing lane
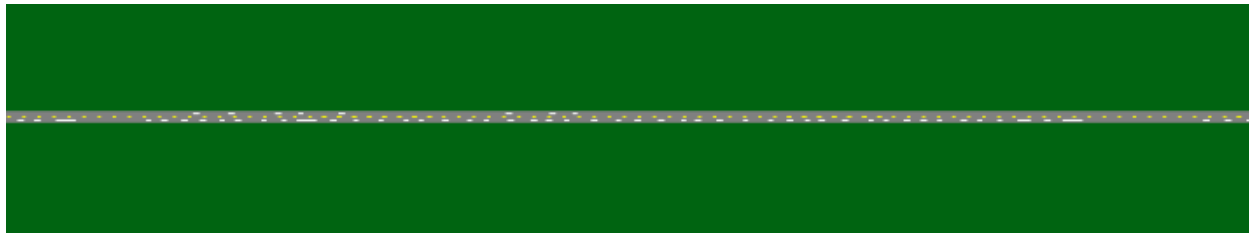
Figure 4-4. Platoon structure for 3200 veh/h flow rate

The *ATS* plots are consistent with the preliminary speed-flow tests. The HCM has a linear decreasing trend and CORSIM has a curve shape that initially has a fairly steep negative slope for low flow rates and then the curve slope decreases to the point where it is fairly flat over the moderate to high flow rates for both 0% and 10% heavy vehicles. The *ATS* values are similar between 0% and 10% heavy vehicles. The *ATS* plots are shown in Figure 4-5 and Figure 4-6. The plots show that flow rate may not have such a large effect on speed as the HCM shows.
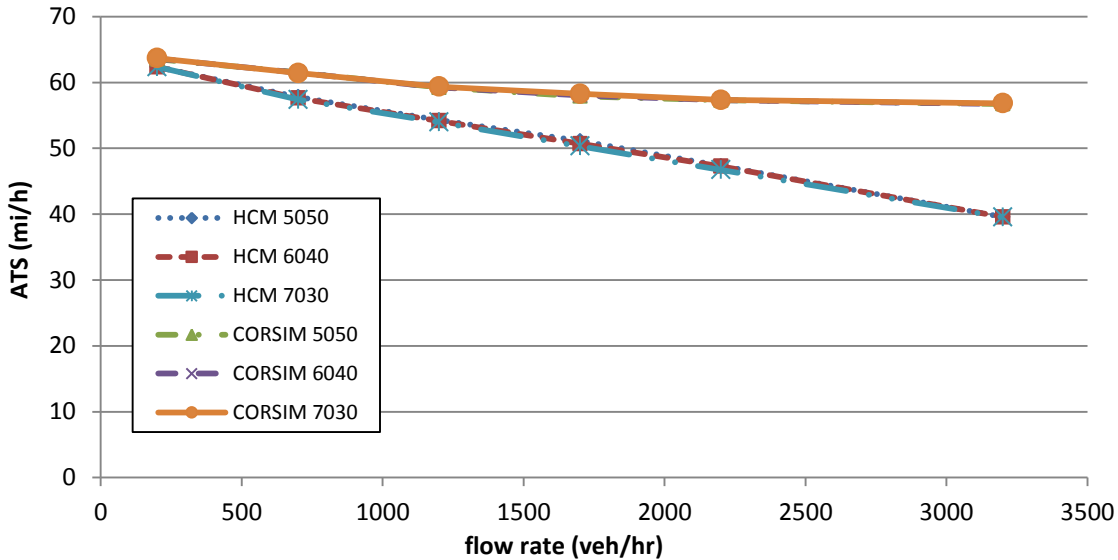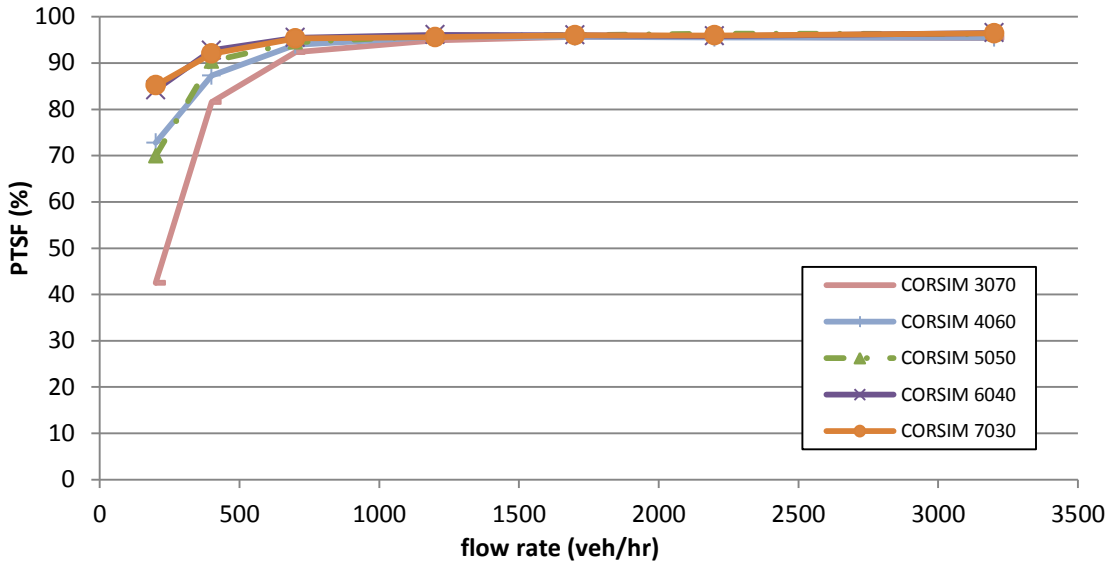
Figure 4-5. *ATS* vs. two-way flow rate - 0% grade, 0%NPZ, 0%HV, no passing lane
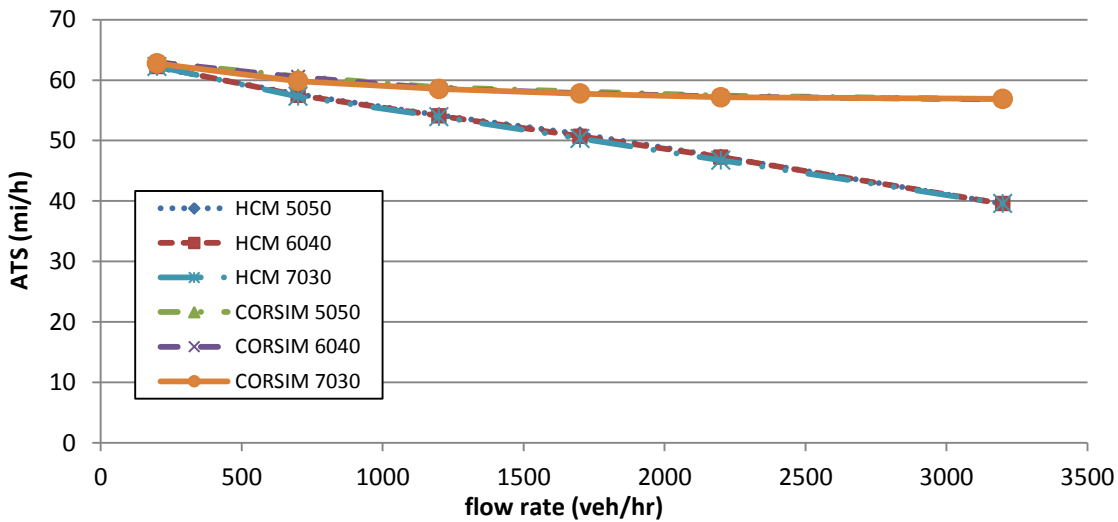
Figure 4-6. *ATS* vs. two-way flow rate - 0% grade, 0%NPZ, 10%HV, no passing lane

4.2.2   0% no-passing zone with 6% grade test results

For 0% heavy vehicles, CORSIM showed higher *PTSF* results than the HCM for flow rates lower than 1700 veh/h. For flow rates higher than 1700 veh/h, the HCM showed higher values for *PTSF* than CORSIM. The trend for *PTSF* is similar between the two tools. For 10% heavy vehicles, the HCM shows the same general trend, but the values for *PTSF* are higher because grade has a large effect on truck speeds and slow-moving trucks cause platoons to form. The HCM *PTSF* value at 3200 veh/h for the 70/30 split is 99.9%, which is unrealistic. The reason may be that the HCM equations are not valid for two-lane highways after breakdown.

In CORSIM, the *PTSF* results at 200 veh/h are about 50% higher than the HCM results for a 50/50 directional split and about 60% higher than the HCM results for the 60/40 and 70/30 directional splits for 10% heavy vehicles. For flow rates higher than 700 veh/h, the *PTSF* is consistently near 95% for the CORSIM curve while the HCM trend is similar to the 0% heavy vehicle case. The values given by CORSIM seem unreasonably high, especially for the lower flow rates. The *PTSF* values are high because long platoons form on the upgrade and the vehicles travel at high speeds on the opposing-direction downgrade. Therefore, there are few gaps for passing opportunities because the opposing vehicles arrive frequently. The *PTSF* plots are shown in Figure 4-7 and Figure 4-8.
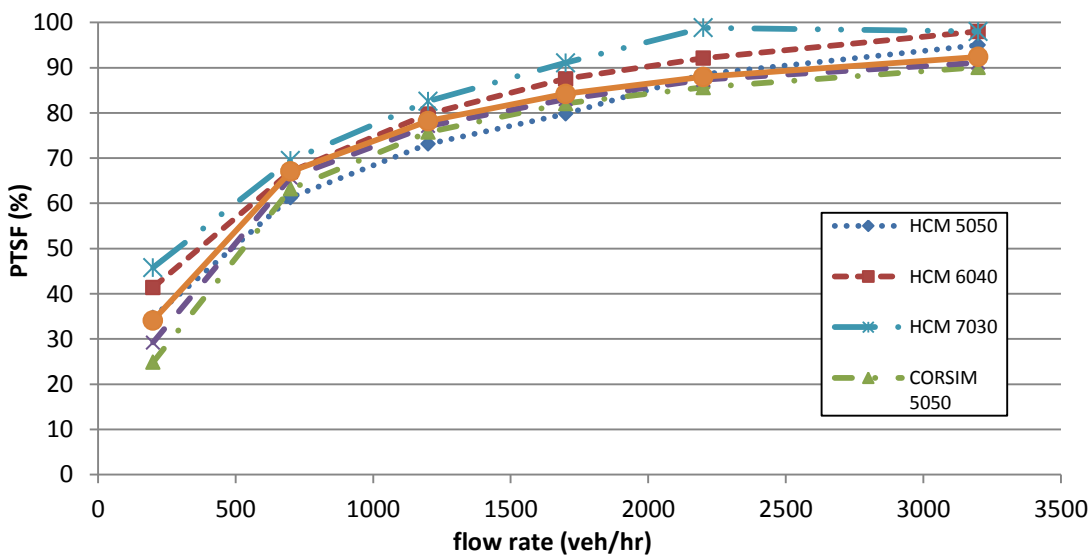

Figure 4-7. *PTSF* vs. two-way flow rate - 6% grade, 0%NPZ, 0%HV, no passing lane
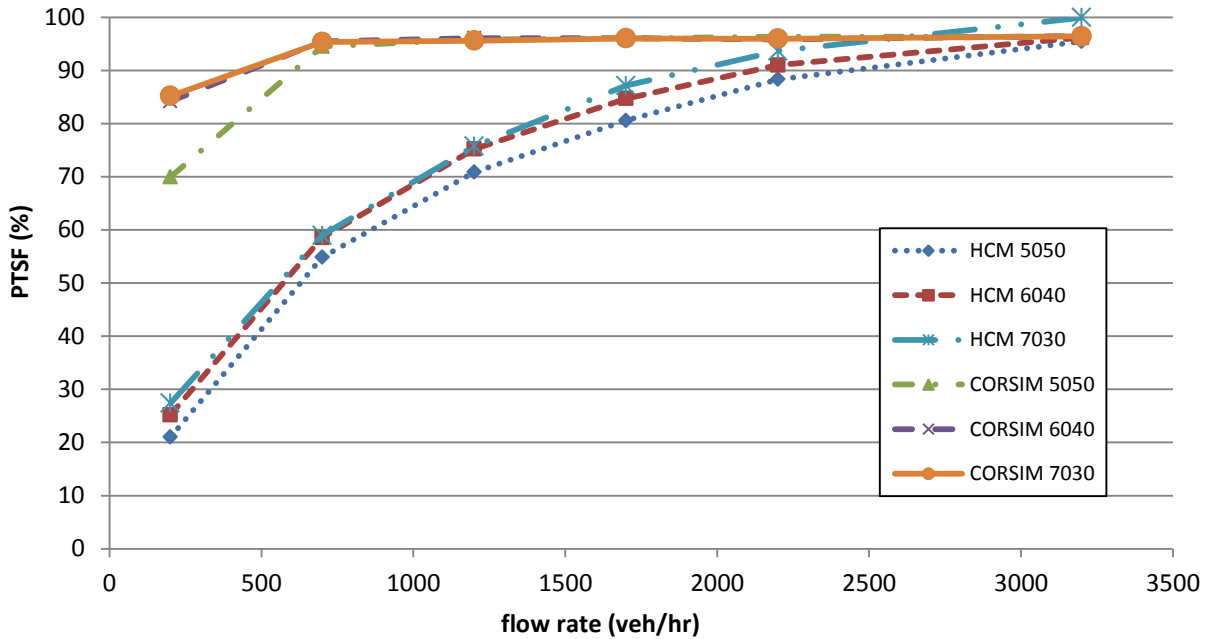
Figure 4-8. *PTSF* vs. two-way flow rate - 6% grade, 0%NPZ, 10%HV, no passing lane

For 0% heavy vehicles, CORSIM and the HCM had the same trends and values for *ATS* as with the 0% grade. For 10% heavy vehicles, the HCM *ATS* values generally followed a linear decreasing path. The directional splits showed slight differences in *ATS* for values before the flow rate of 1700 veh/h. At a flow rate of 3200 veh/h, the speed drops to an unreasonably low value of 4.7 mi/h. The *ATS* plots are shown in Figure 4-9 and Figure 4-10.
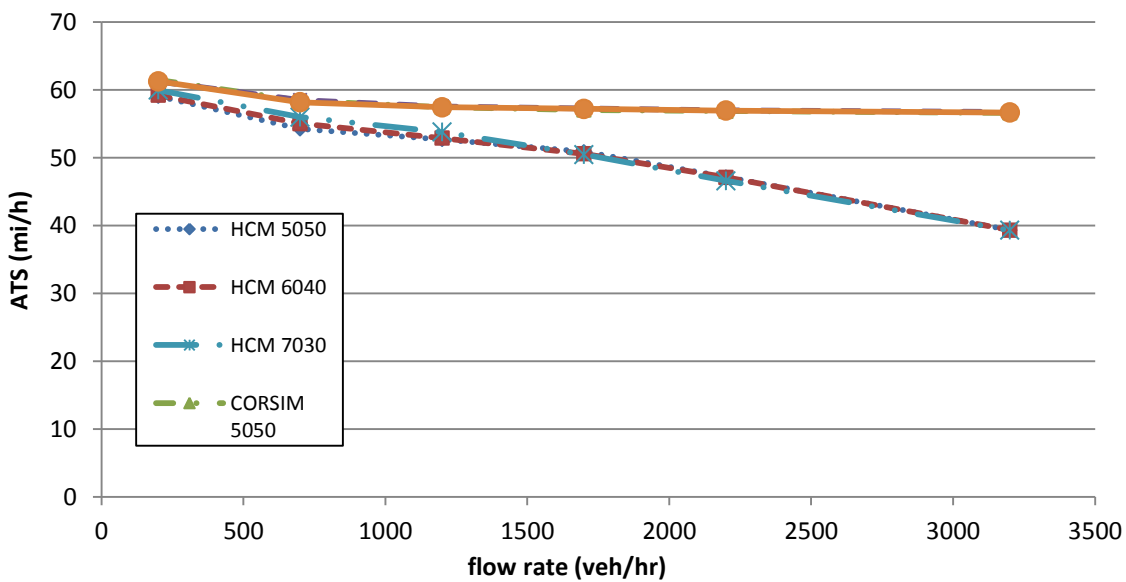
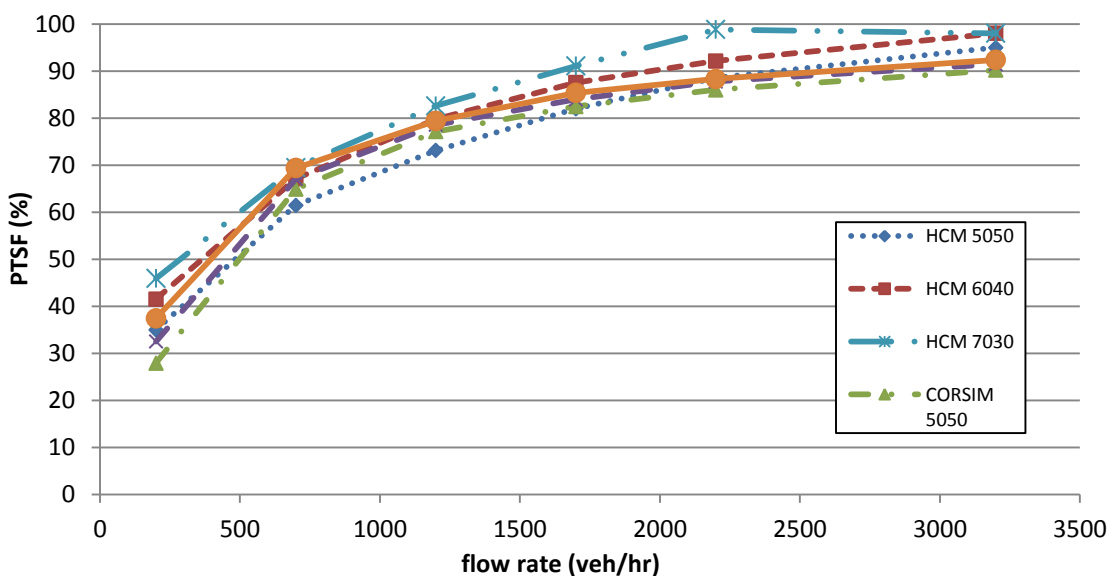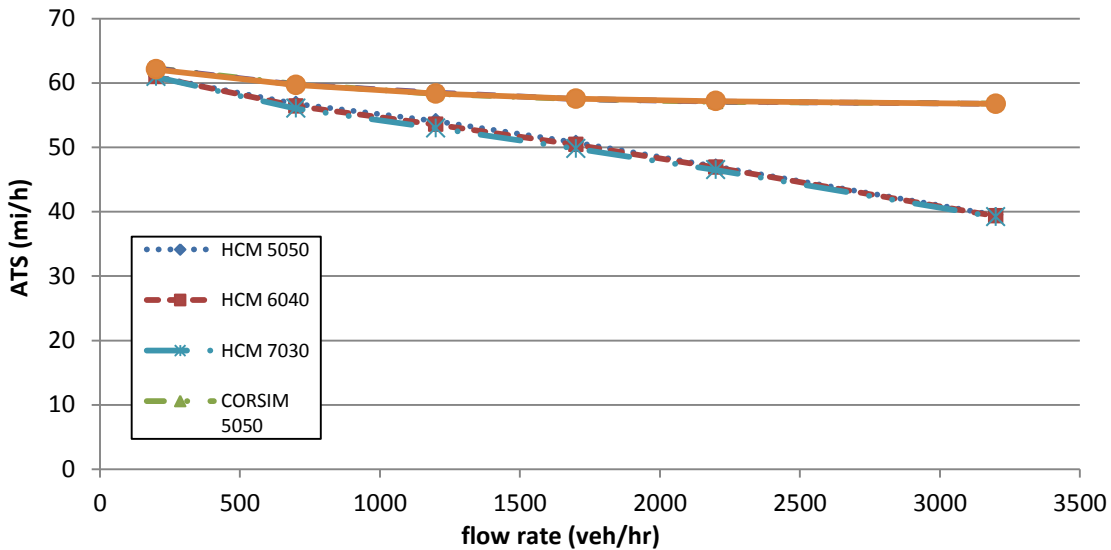
Figure 4-9. *ATS* vs. two-way flow rate - 6% grade, 0%NPZ, 0%HV, no passing lane
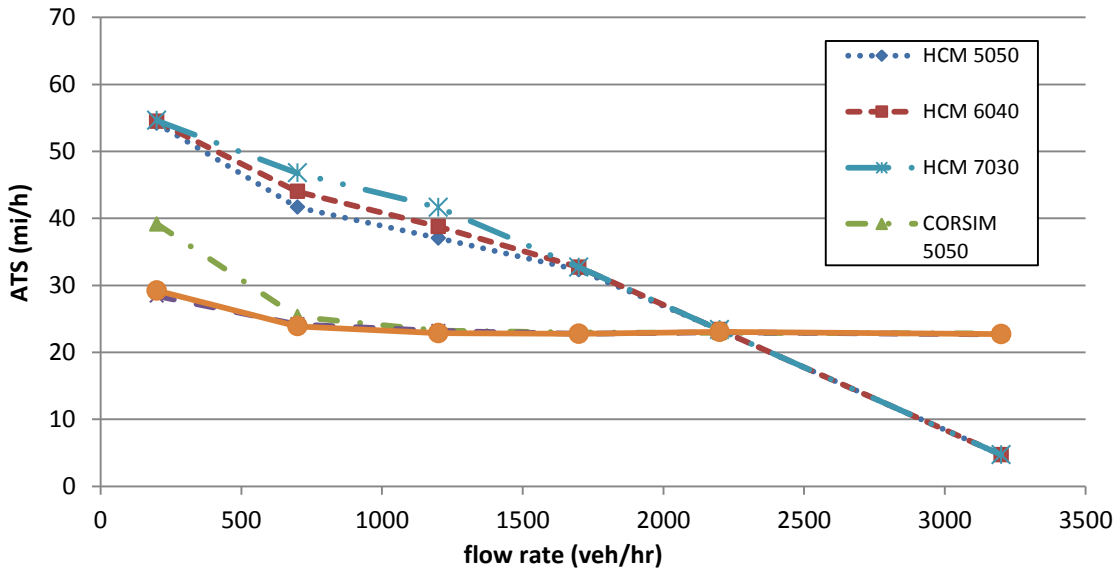
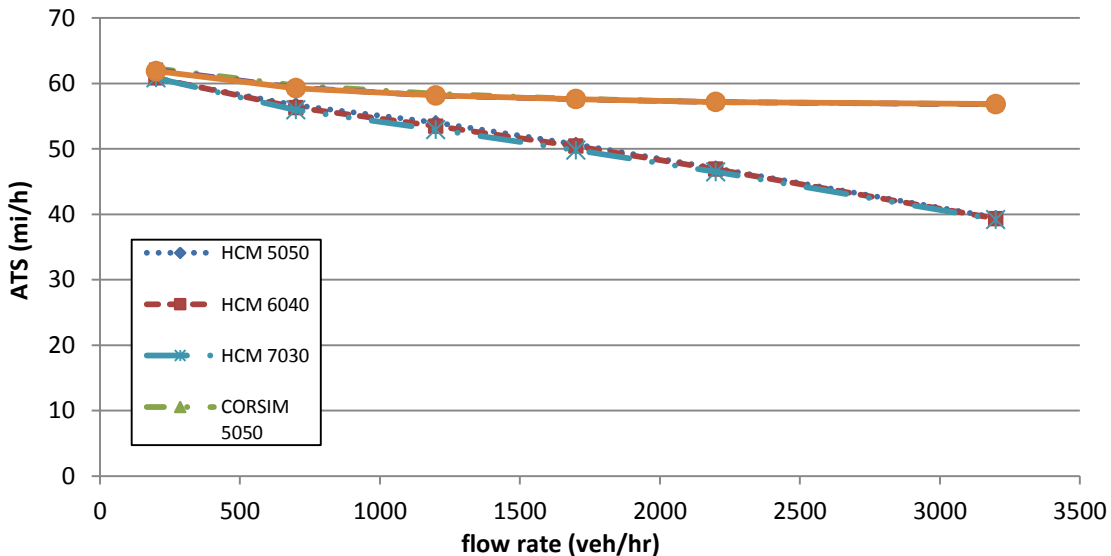Figure 4-10. *ATS* vs. two-way flow rate - 6% grade, 0%NPZ, 10%HV, no passing lane

The CORSIM results for *ATS* for 10% heavy vehicles followed the same trend as the 0% grade scenario, but the values for *ATS* were much lower. These low *ATS* values are caused by slow-moving trucks in the traffic stream due to the grade. Even for low traffic volumes, there are few opportunities to pass because the vehicles in the opposing direction are on a 6% downgrade. They are traveling at high speeds so vehicles cannot be in the opposing lane very long before the oncoming vehicle approaches and, as a result, vehicles in the major direction have inadequate time to pass. For a flow rate of 200 veh/h, the CORSIM *ATS* for a 50/50 split was higher than the CORSIM *ATS* for a 60/40 split and a 70/30 split. In order to further investigate the CORSIM *PTSF* and *ATS* results for 10% heavy vehicles on a 6% grade, two additional splits, 40/60 and 30/70, were analyzed. The results are shown in Figure 4-11 and Figure 4-12.
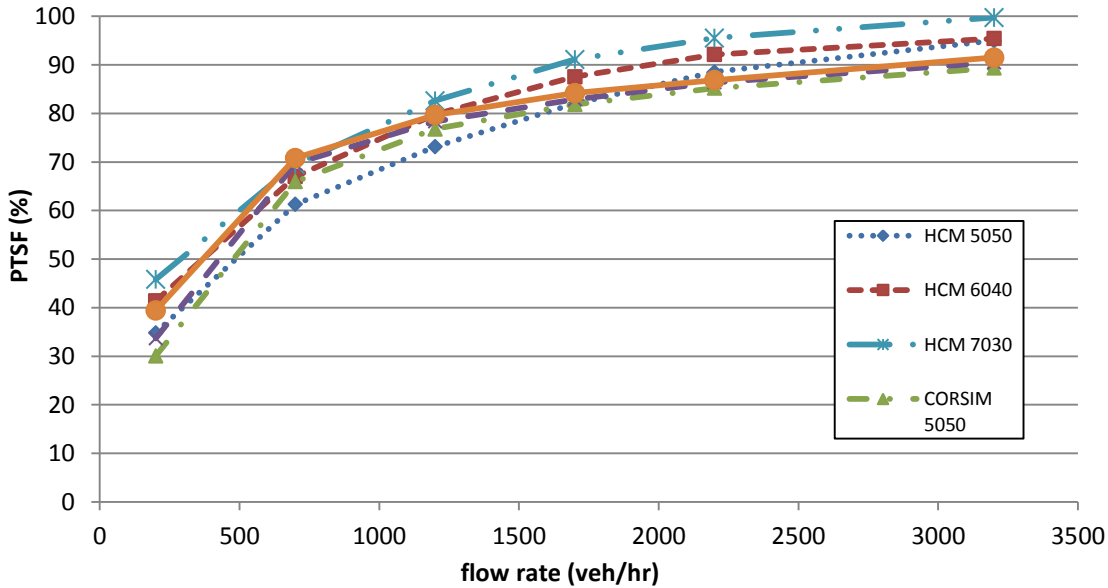
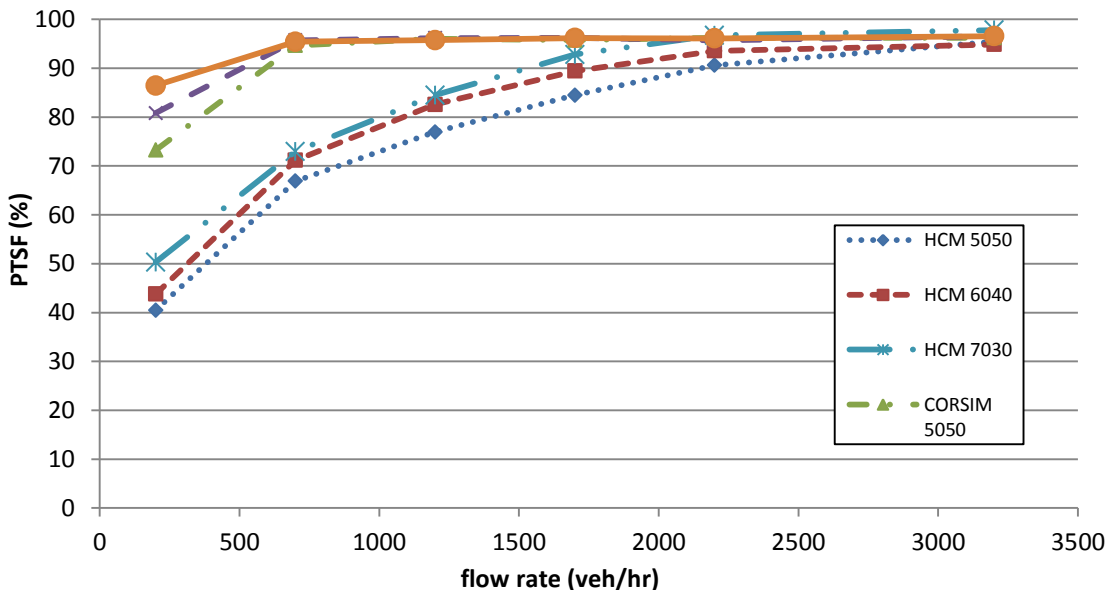Figure 4-11. *PTSF* vs. two-way flow rate - 6% grade, 0% NPZ, 10% HV, no passing lane
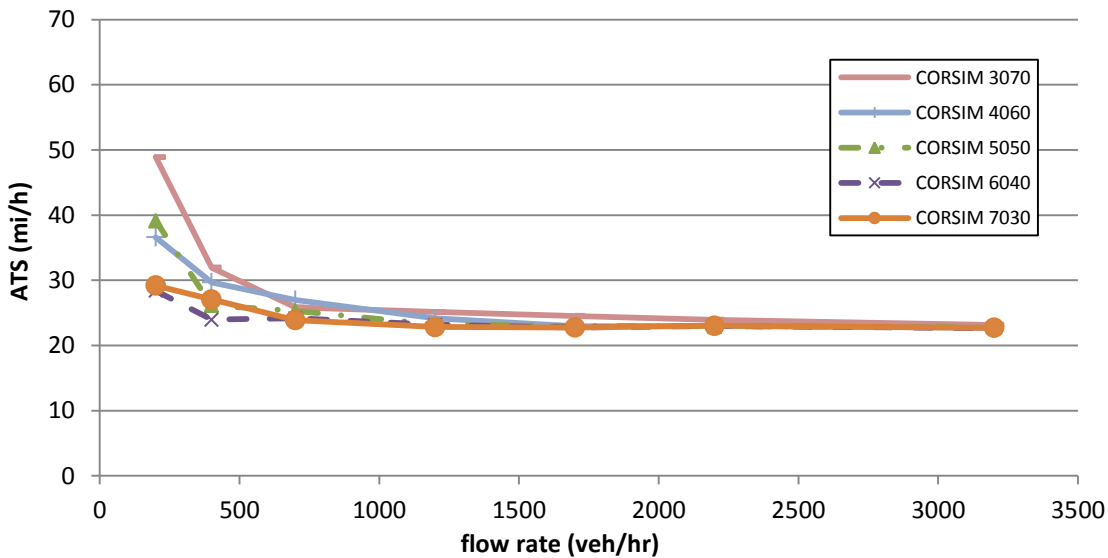
Figure 4-12. *ATS* vs. two-way flow rate - 6% grade, 0% NPZ, 10% HV, no passing lane

There was a greater difference between the splits for the flow rates of 200 and 700 veh/h. Then, the curves for all splits converged to about 96% for *PTSF* and about 23 mi/h for *ATS*. The performance measures are more sensitive to splits for lower flow rates because the number of vehicles in the traffic stream is major factor for vehicle interactions, which affects platooning. Vehicles are generated more frequently for a 70/30 split than for a 30/70 split. One slow truck in the 70/30 split traffic stream impedes all subsequent vehicles. There are fewer total vehicles to be affected by a slow truck in the 30/70 split.

4.2.3   50% no-passing zone with 0% grade test results

The 2010 HCM and CORSIM followed the same trend for 50% no-passing zones as for the corresponding 0% no-passing zone plots for *PTSF* for both 0% and 10% heavy vehicles as shown in Figure 4-13 and Figure 4-14. However, the HCM 70/30 split has more separation from the other splits for CORSIM and the HCM than in the 0% no-passing zone case. This is especially true at the flow rate of 2200 veh/h. The HCM and CORSIM followed the same trend for 50% no-passing zones as for the corresponding 0% no-passing zone plots for *ATS* for both 0% and 10% heavy vehicles as shown in Figure 4-15 and Figure 4-16.

Figure 4-13. *PTSF* vs. two-way flow rate - 0% grade, 50%NPZ, 0%HV, no passing lane

Figure 4-14. *PTSF* vs. two-way flow rate - 0% grade, 50%NPZ, 10%HV, no passing lane
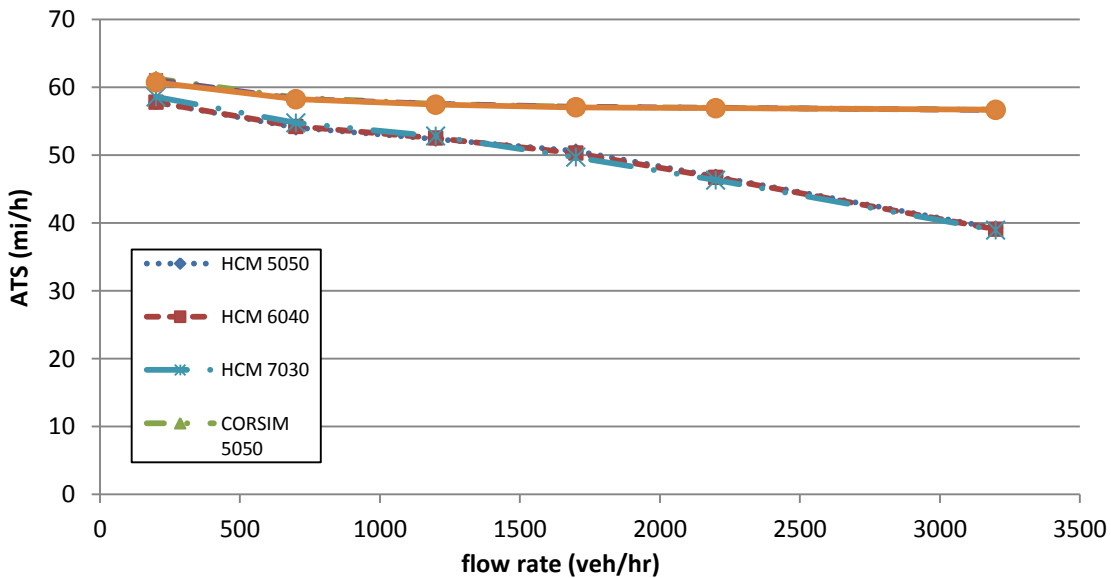

Figure 4-15. *ATS* vs. two-way flow rate - 0% grade, 50%NPZ, 0%HV, no passing lane
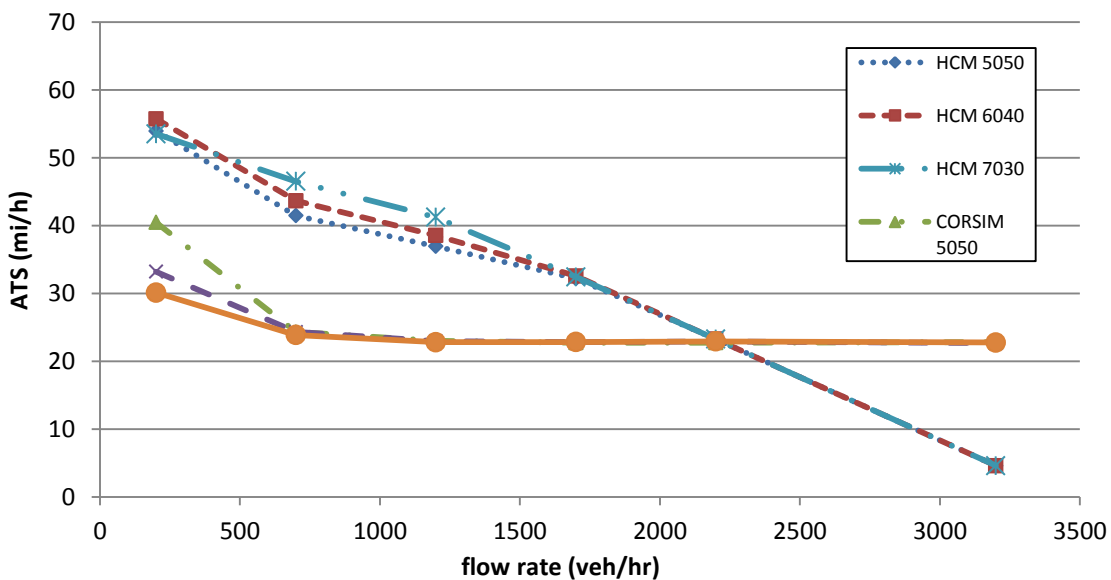

Figure 4-16. *ATS* vs. two-way flow rate - 0% grade, 50%NPZ, 10%HV, no passing lane

4.2.4   50% no-passing zone with 6% grade test results

The HCM and CORSIM *PTSF* curves have a more similar slope for 0% heavy vehicles than for the 0% no-passing zone case. The HCM curve for the 70/30 directional split shows the greatest values for *PTSF* and is slightly separated from the other curves. Overall, the *PTSF* values are higher for 50% no-passing zones than the *PTSF* values for 0% no-passing zones because there are fewer passing opportunities in the 50% no-passing zone case.

For 10% heavy vehicles, the HCM and CORSIM followed the same trend for 50% no-passing zones as for the corresponding 0% no-passing zone plots. However, the HCM curves have more separation between themselves and the CORSIM 60/40 split is more separated from the CORSIM 70/30 split at the flow rate of 200 veh/h. The CORSIM *PTSF* values for the 50/50 and 70/30 split are higher than for the 0% no-passing zones case and the 60/40 split is lower for the flow rate of 200 veh/h. The *PTSF* plots are shown in Figure 4-17 and Figure 4-18.
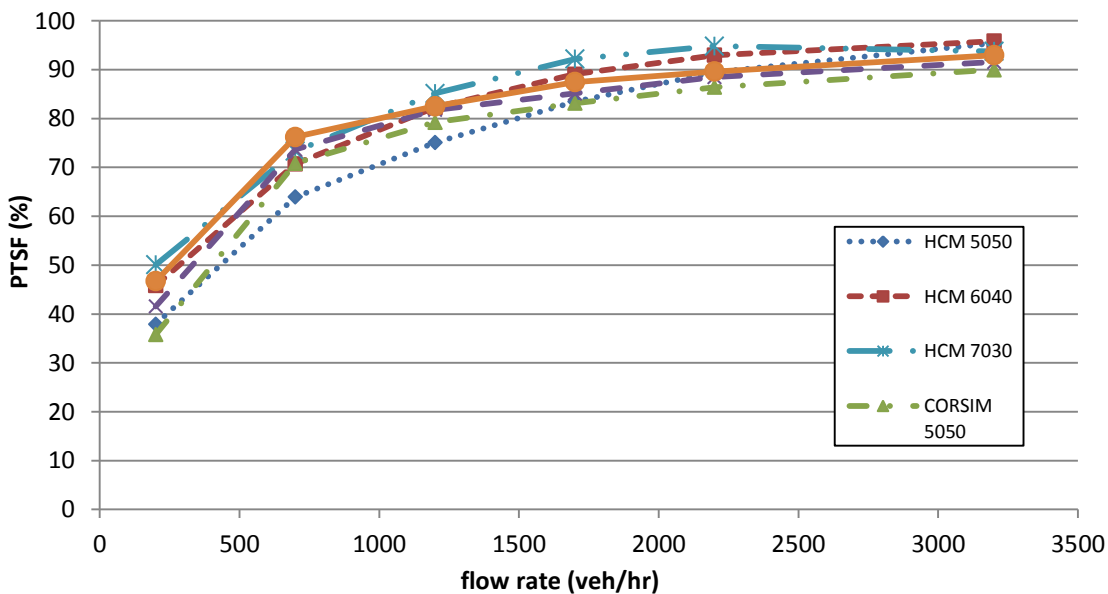

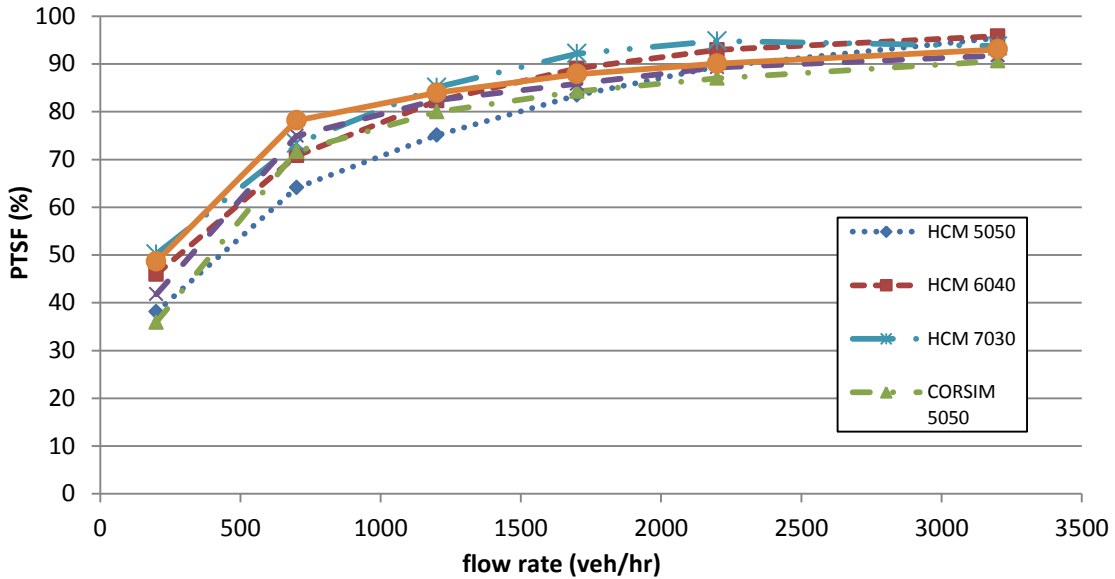Figure 4-17. *PTSF* vs. two-way flow rate - 6% grade, 50%NPZ, 0%HV, no passing lane

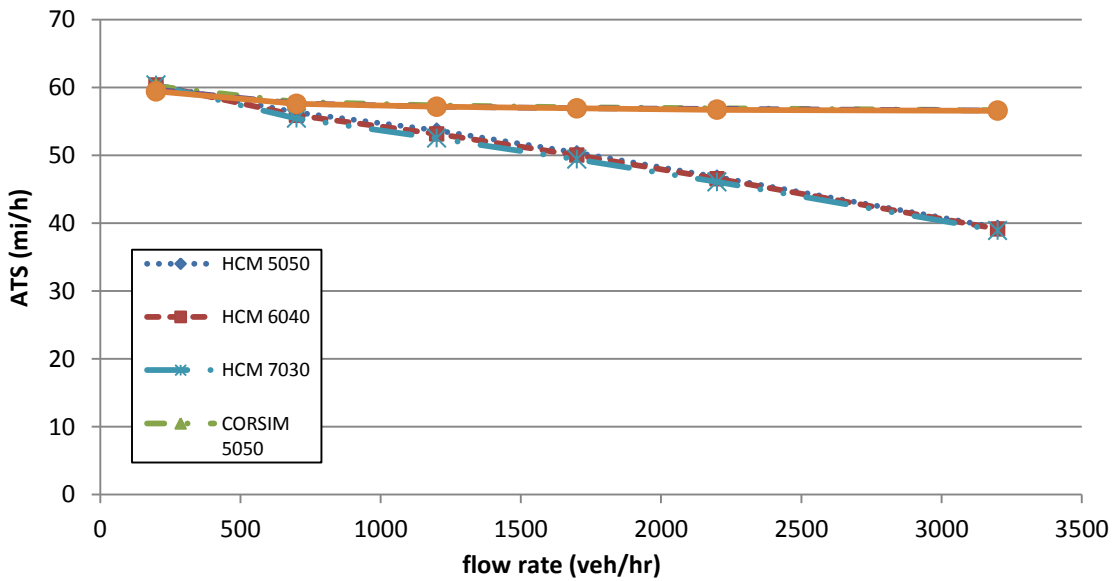Figure 4-18. *PTSF* vs. two-way flow rate - 6% grade, 50%NPZ, 10%HV, no passing lane

Figure 4-19. *ATS* vs. two-way flow rate - 6% grade, 50%NPZ, 0%HV, no passing lane
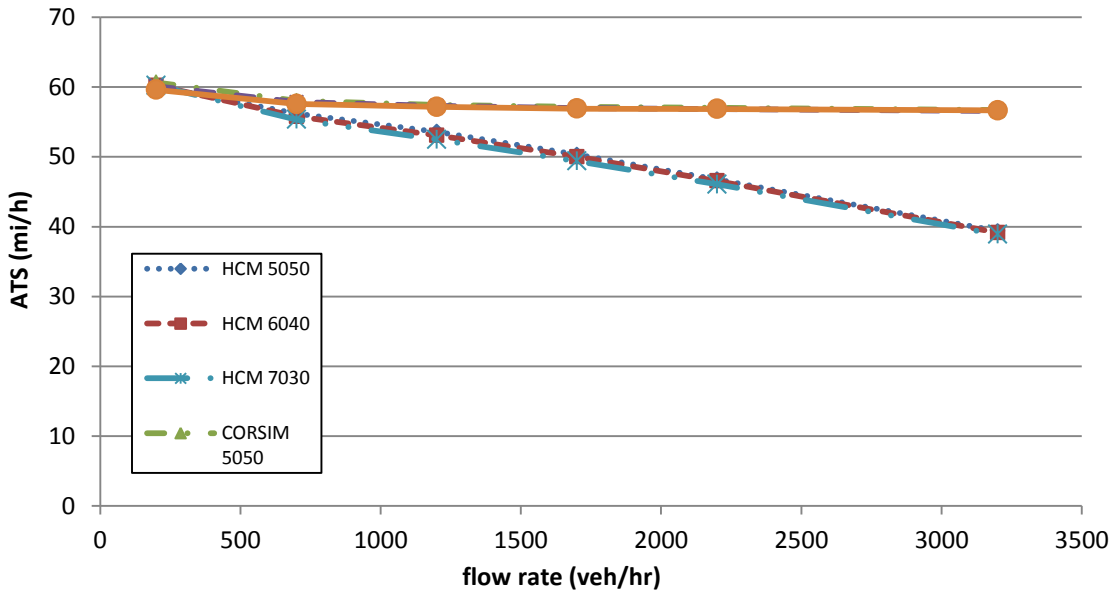

Figure 4-20. *ATS* vs. two-way flow rate - 6% grade, 50%NPZ, 10%HV, no passing lane

4.2.5 100% no-passing zone with 0% grade test results

The HCM and CORSIM *PTSF* curves are much closer together for 100% no-passing zones for both 0% and 10% heavy vehicles than for the 50% no-passing zone case. The HCM 50/50 split shows the highest separation from the other curves. The HCM curves level off at lower *PTSF* values than the other two no-passing zone scenarios. The highest *PTSF* value reached for both heavy vehicle percentages is near 95% rather than 100%. *PTSF* shows improvement for 100% no-passing zones because the value for the no-passing zone adjustment factor must be extrapolated from Exhibit 15-21 in the 2010 HCM when the two-way flow rate is greater than the highest value in the table. This could potentially lead to a negative no-passing zone adjustment factor, which would cause the *PTSF* to improve. The *PTSF* plots are shown in Figure 4-21 and Figure 4-22. The HCM and CORSIM followed the same trend for 100% no-passing zones as for the 0% and 50% no-passing zone cases for *ATS* for both 0% and 10% heavy vehicles The *ATS* plots are shown in Figure 4-23 and Figure 4-24.
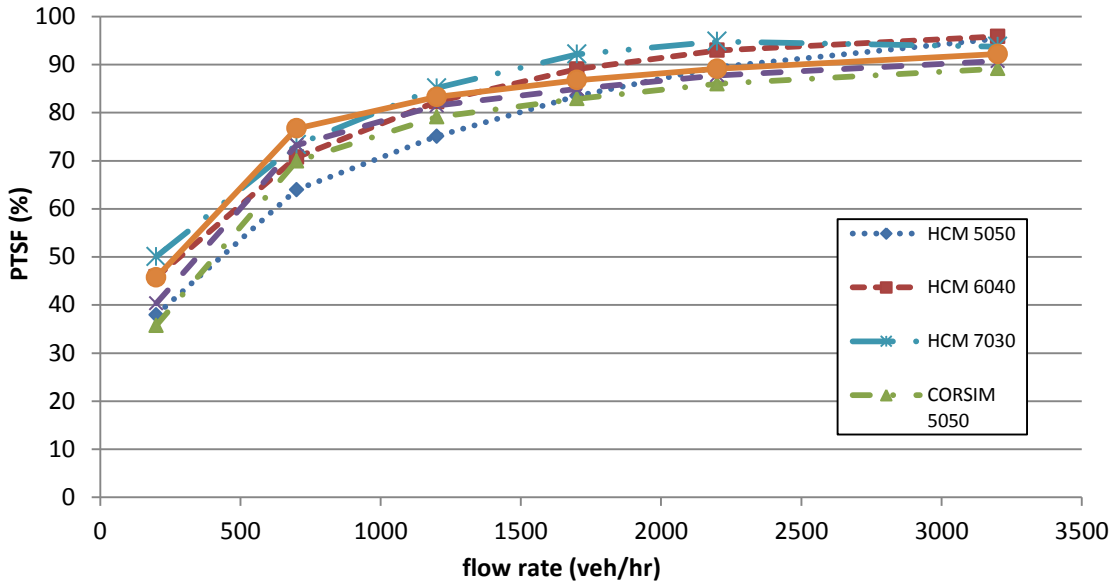

Figure 4-21. *PTSF* vs. two-way flow rate - 0% grade, 100% NPZ, 0% HV, no passing lane

Figure 4-22. *PTSF* vs. two-way flow rate - 0% grade, 100%NPZ, 10%HV, no passing lane
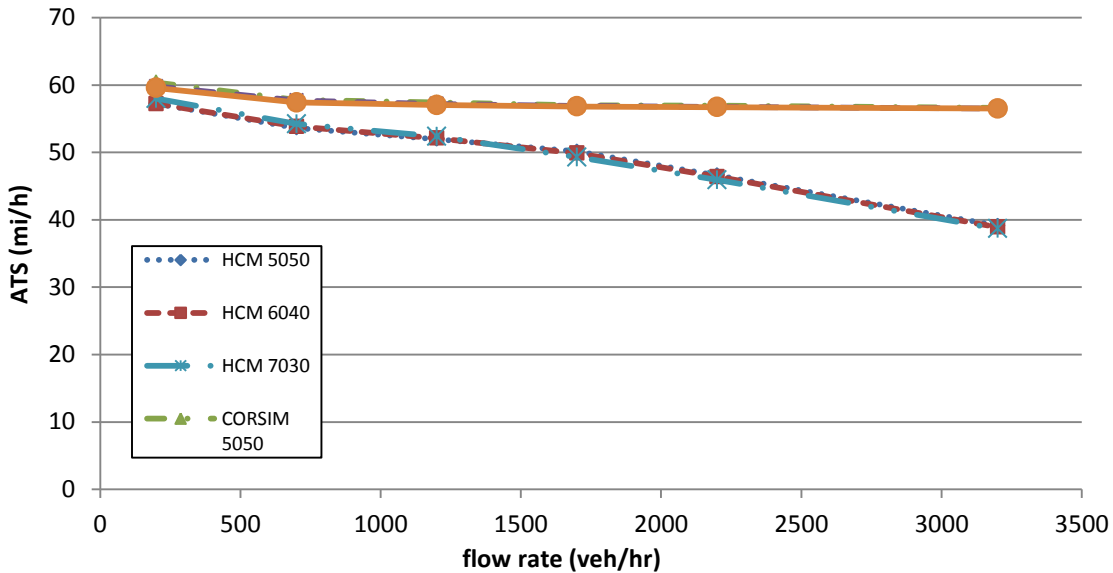

Figure 4-23. *ATS* vs. two-way flow rate - 0% grade, 100%NPZ, 0%HV, no passing lane

Figure 4-24. *ATS* vs. two-way flow rate - 0% grade, 100%NPZ, 10%HV, no passing lane

4.2.6   100% no-passing zone with 6% grade test results

The HCM and CORSIM *PTSF* curves are closer together for the 100% no-passing zone case for 0% heavy vehicles than for the 0% and 50% no-passing zone cases. Overall, the *PTSF* values are higher, but the maximum values reached for any curve are lower than the values reached by curves in the other two no-passing zone cases. The plot is shown in Figure 4-25. For the 10% heavy vehicle case, The HCM 50/50 directional split curve has more separation from the other two HCM curves than in the 0% and 50% no-passing zone cases. Also, the CORSIM 60/40 curve overlaps the CORSIM 70/30 split curve from the flow rate of 200 veh/h to the flow rate of 700 veh/h as in the 0% no-passing zone case. For the lower flow rates, the *PTSF* values were all slightly higher than the values given for the other two no-passing zone cases. The plot is shown in Figure 4-26.

Figure 4-25. *PTSF* vs. two-way flow rate - 6% grade, 100% NPZ, 0% HV, no passing lane
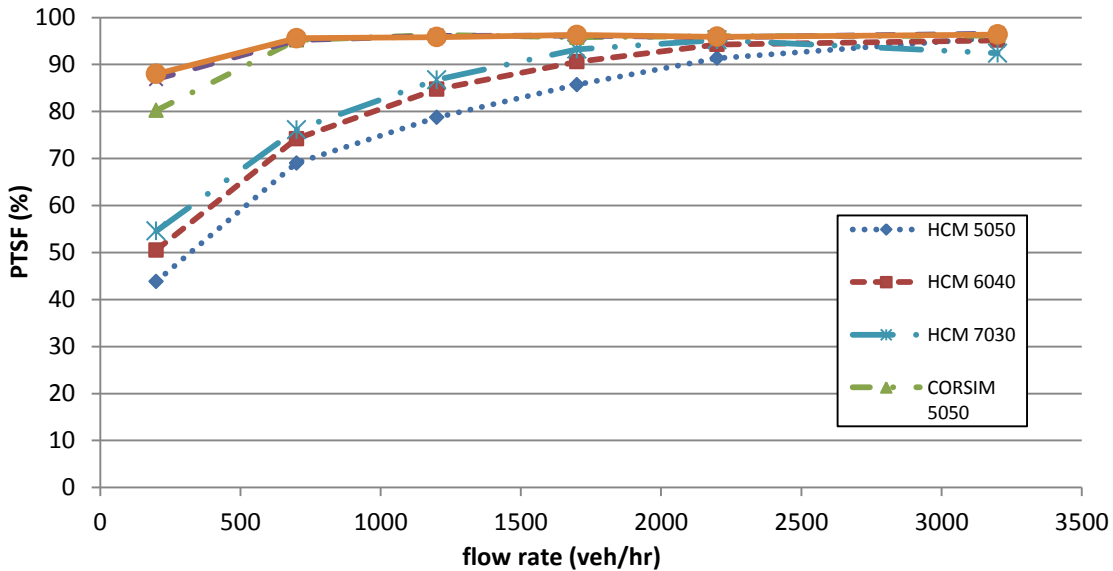
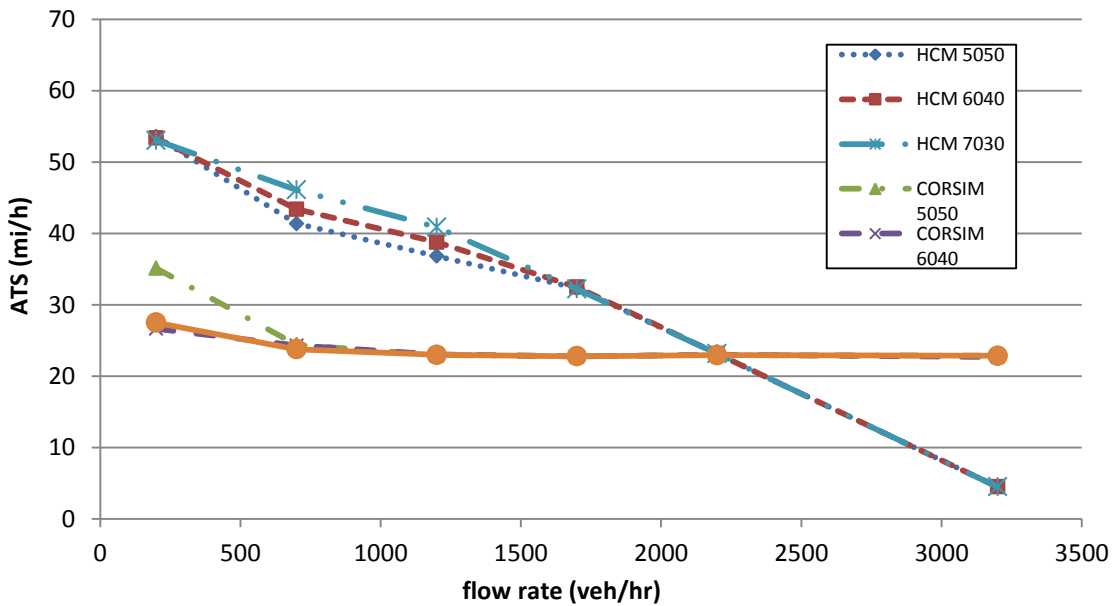Figure 4-26. *PTSF* vs. two-way flow rate - 6% grade, 100% NPZ, 10% HV, no passing lane

The *ATS* curves for CORSIM and the HCM have trends and values that are similar to the other no-passing zone cases for both percentages of heavy vehicles. However, the CORSIM 60/40 split overlaps the CORSIM 70/30 split curve from the flow rate of 200 veh/h to the flow rate of 700 veh/h as in the 0% no-passing zone case. The plots are shown in Figure 4-27 and Figure 4-28.

Figure 4-27. *ATS* vs. two-way flow rate - 6% grade, 100%NPZ, 0%HV, no passing lane

Figure 4-28. *ATS* vs. two-way flow rate - 6% grade, 100%NPZ, 10%HV, no passing lane

4.2.7   Passing lane test results

The *ATS* average for all passing lane scenarios is slightly higher than the corresponding no passing lane scenarios in both CORSIM and the HCM, but within about one mi/h. The *PTSF* values were affected more than *ATS* by the passing lane in both tools. CORSIM showed

improvements as high as 17%, which were higher than the HCM's greatest improvements at 13%. The CORSIM results showed that the total *PTSF* improvements increased as the percentage of no-passing zones increased and that the improvements were greater for lower flow rates.

The performance measures for the passing lane scenarios were analyzed using CORSIM based on the position along the highway in order to evaluate how a passing lane affects *PTSF* and *ATS*. Since the no-passing-lane scenarios did not show much variation between splits, only the 60/40 split scenarios were plotted. This split was chosen to be plotted because it is the median between the perfectly even distribution of 50/50 and the biased distribution of 70/30.

The *ATS* values decreased at the passing lane link for all no-passing zone cases for the higher volumes. This is counterintuitive, but there is a logical explanation. For the higher volumes such as 2200 veh/h and 3200 veh/h, most of the vehicles are in platoons throughout the entire facility because of a slower truck. Most of the vehicles in the platoons are passenger cars, which are capable of traveling at much higher speeds than trucks, especially when there is a 6% grade. As soon as the platoon reaches the passing lane section, most of the slow trucks move to the outside lane and the cars that have been following are able to travel through at their desired speeds. Eventually, the trucks reach the end of the passing lane section and have to merge back into the original lane. When a truck merges back in from the passing lane and continues onto the next link, the cars are trapped again in a platoon behind the truck as shown in Figure 4-29. The cars are almost at a complete stop and back up onto the passing lane link. The link directly after the passing lane link acts as a bottleneck. The plots for each no-passing zone case are discussed in the following sections.
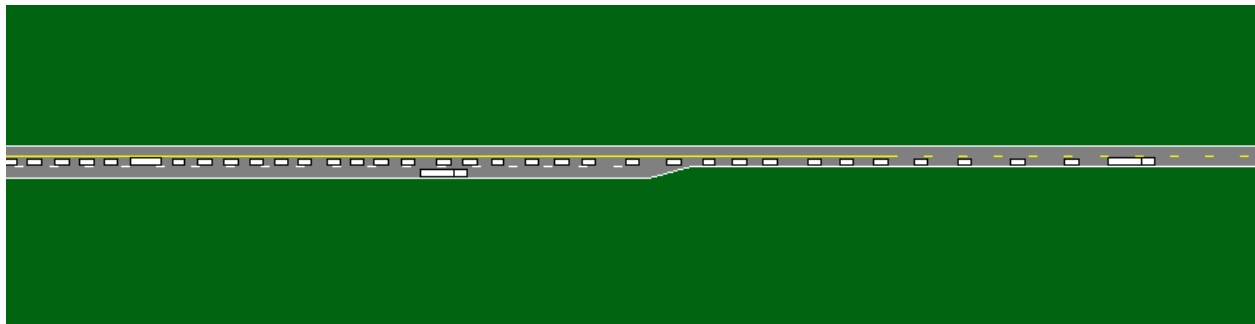

Figure 4-29. Passing lane bottleneck

1. 0% no-passing zones
    For 0% grade, the *PTSF* values increased as flow rate increased for both heavy vehicle cases. The separation between the curves decreased as flow rate increased with the 200 veh/h curve having the greatest separation from the others. There was a large *PTSF* drop at the passing lane location. This happens because the slow trucks move into the right-hand lane and the cars are able to travel through at their desired speeds. The effects of the *PTSF* reduction lasted further downstream of the passing lane location for the lower flow rates. Overall, the *PTSF* values were higher for the 10% heavy vehicles case. The plots are shown in Figure 4-30 and Figure 4-31.
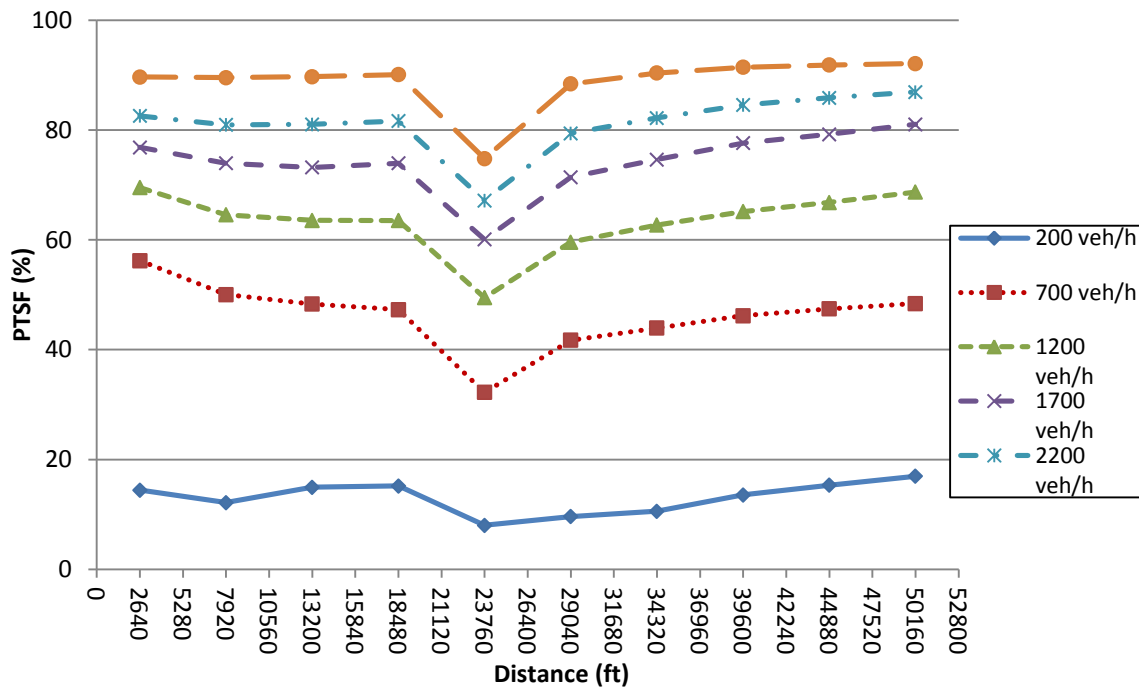
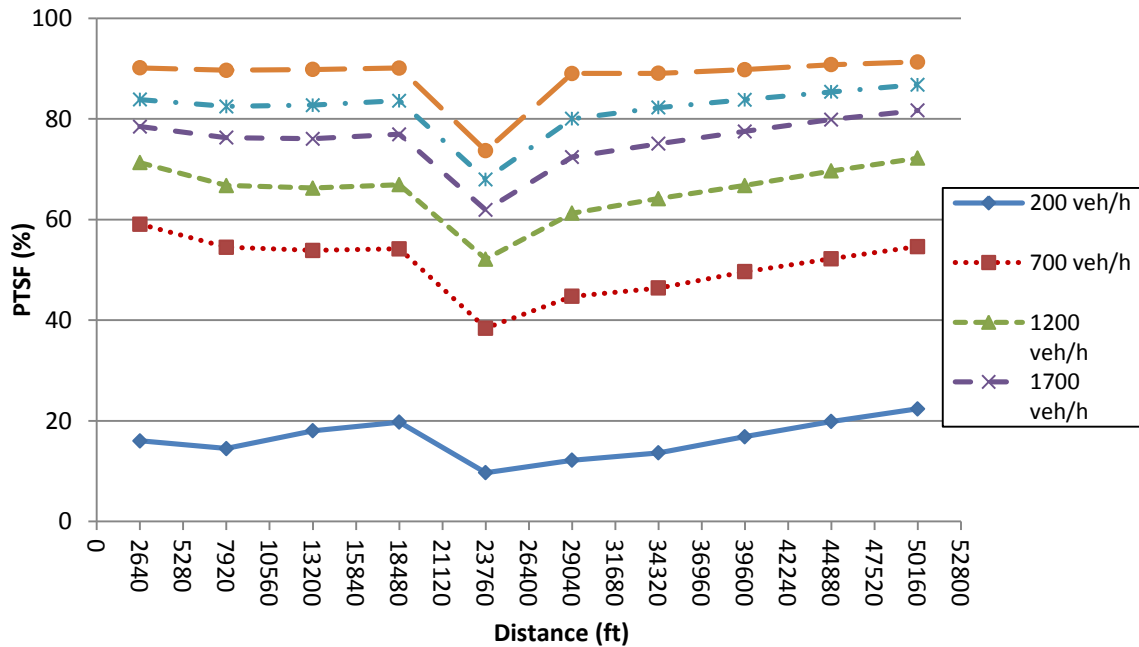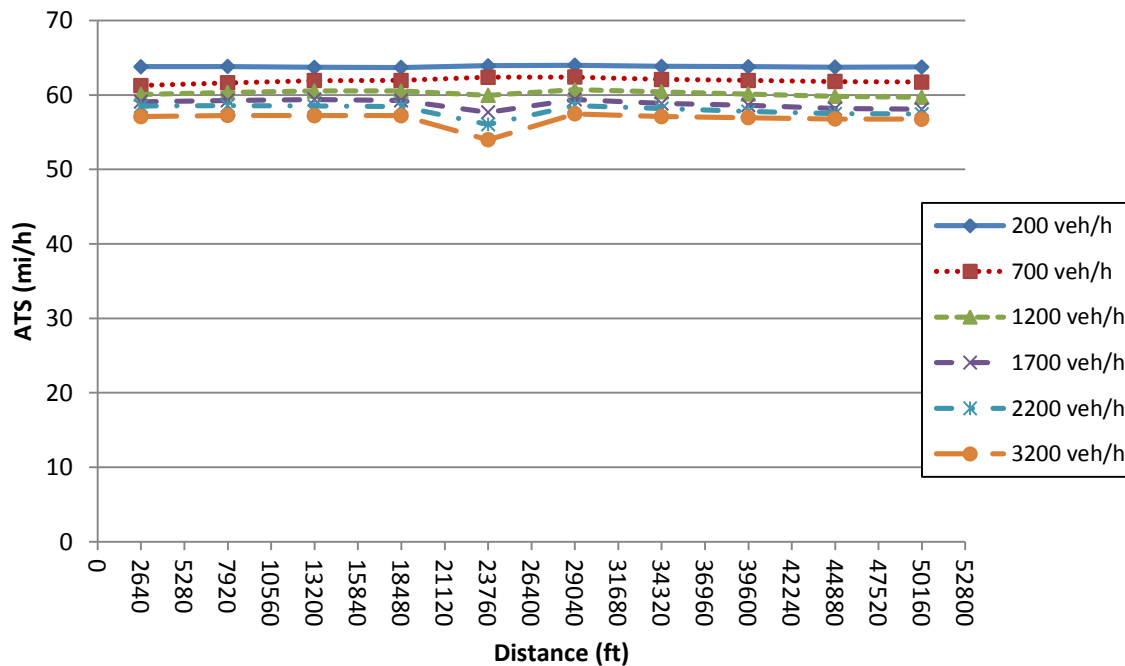Figure 4-30. *PTSF* vs. distance - 0% grade, 0%NPZ, 0% HV, passing lane

Figure 4-31. *PTSF* vs. distance - 0% grade, 0%NPZ, 10% HV, passing lane

The *ATS* curves are bunched close together for 0% grade. The passing lane does not have a large impact on speed. There was a slight increase in the *ATS* values after the passing lane for the flow rate of 700 veh/h. The *ATS* values were basically unchanged after the passing lane for the flow rate of 200 veh/h. All other flow rates had an *ATS* drop at the passing lane location. This happens at the higher flow rates because, when trucks merge back into the original lane, the cars are forced to slow down and become part of a slow-moving platoon again. Since the passenger cars are able to travel fast on the passing lane link, they arrive quickly and frequently at the end of that link, but they get backed up because of the slow-moving trucks at the beginning of the next link. The *ATS* plots are shown in Figure 4-32 and Figure 4-33.

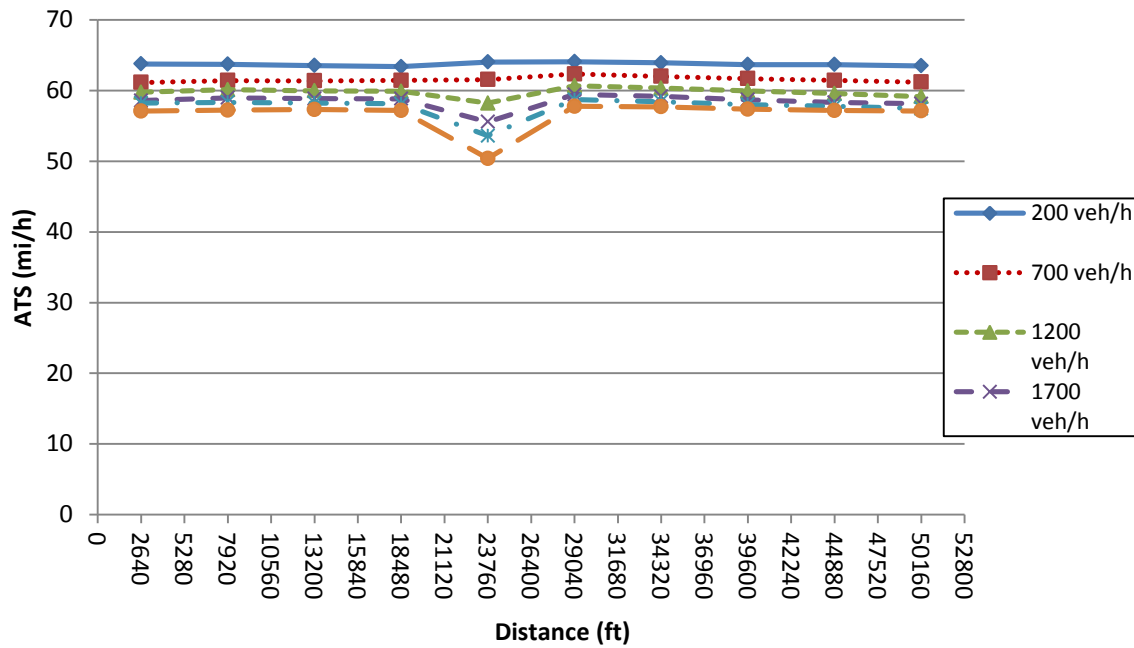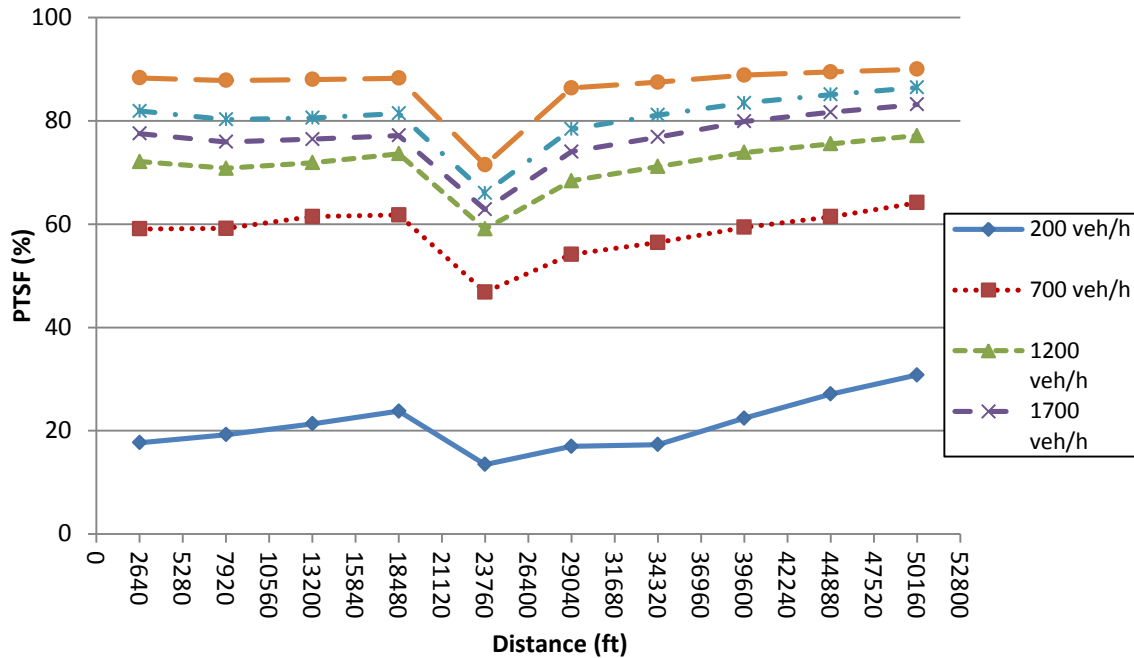Figure 4-32. *ATS* vs. distance - 0% grade, 0% NPZ, 0% HV, passing lane

Figure 4-33. *ATS* vs. distance - 0% grade, 0% NPZ, 10% HV, passing lane

For a 6% grade, the *PTSF* trend is similar to the 0% grade trend. The curves are closer together and the values are slightly higher. The 200 veh/h curve still has the greatest separation from the other curves. The *PTSF* drop is larger for the higher flow rates, but the downstream effects last longer for the lower flow rates. For 10% heavy vehicles, the flow rate curves are all close together except for the 200 veh/h curve. The *PTSF* has a large drop at the passing lane location for the flow rate of 3200 veh/h. The plots are shown in Figure 4-34 and Figure 4-35.

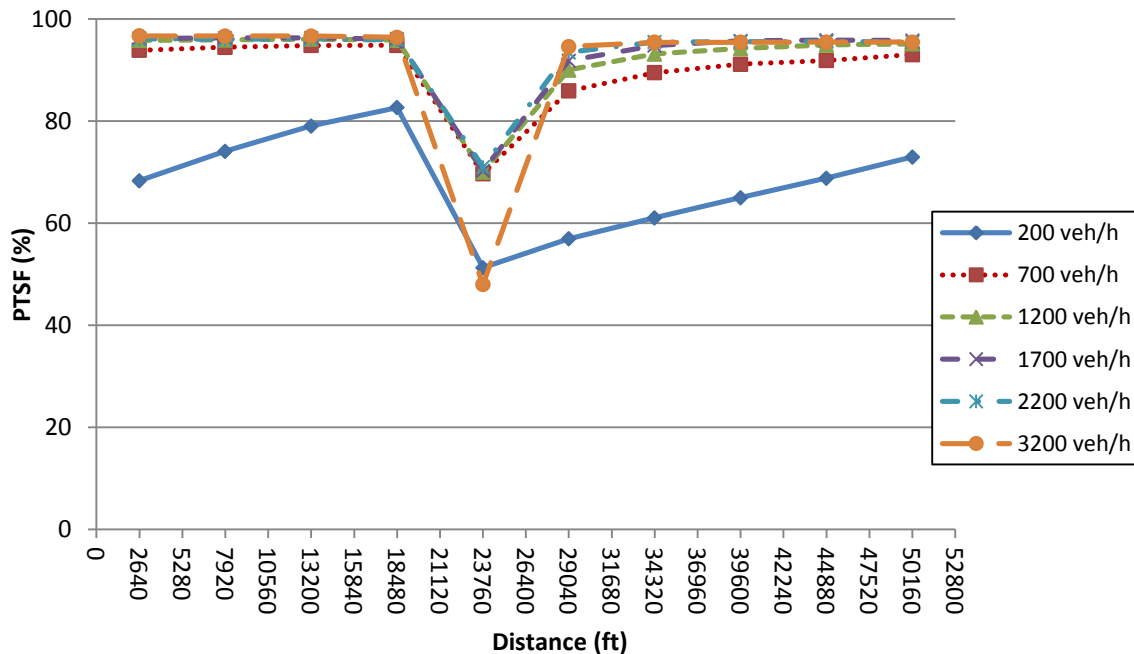Figure 4-34. *PTSF* vs. distance - 6% grade, 0%NPZ, 0% HV, passing lane

Figure 4-35. *PTSF* vs. distance - 6% grade, 0%NPZ, 10% HV, passing lane

The *ATS* values are slightly lower on the 6% grade for the 0% heavy vehicles scenario compared to the 0% grade scenario. For 10% heavy vehicles, the speeds jump at the passing lane location and decrease again for all flow rates except 2200 veh/h and 3200 veh/h. For the flow rates of 2200 veh/h and 3200 veh/h, the *ATS* jumped at the link before the passing lane link. This is because the slow trucks leading the platoons move to the right-hand lane in the passing lane

section, which creates a shockwave upstream. The passenger cars are suddenly free to travel at their desired speeds. Passing lanes are a more effective highway improvement method for low flow rates. The plots are shown in Figure 4-36 and Figure 4-37.
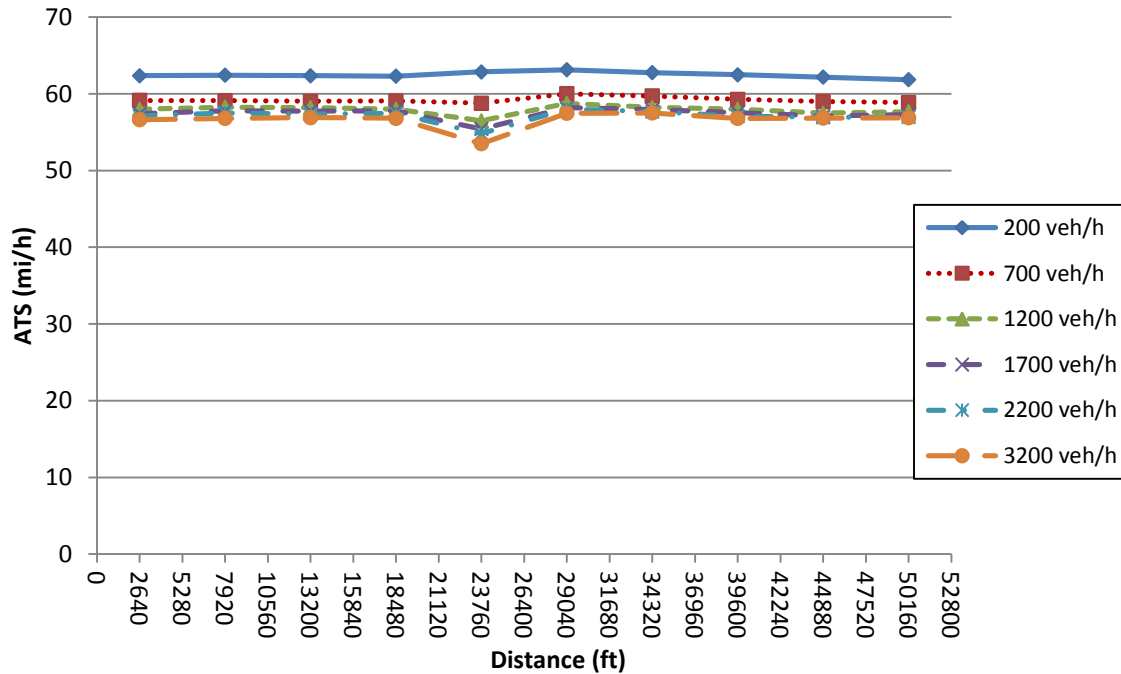

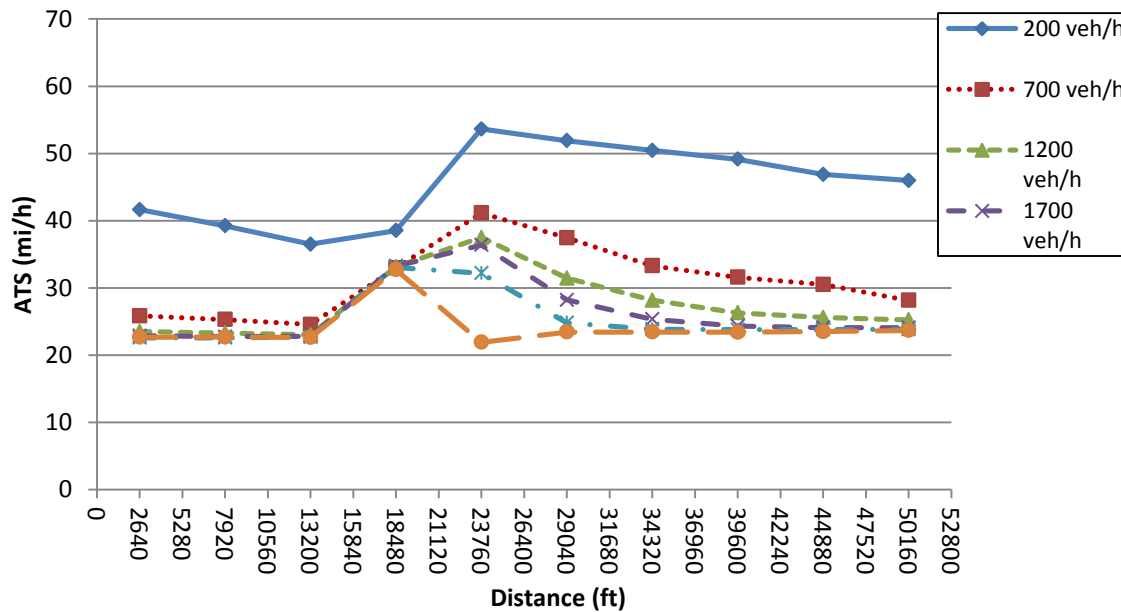Figure 4-36. *ATS* vs. distance - 6% grade, 0%NPZ, 0% HV, passing lane

Figure 4-37. *ATS* vs. distance - 6% grade, 0%NPZ, 10% HV, passing lane

2. 50% no-passing zones

For the 0% grade, the trends are almost the same as for 0% no-passing zones. However, the *PTSF* values increased for the first four links in the 50% no-passing zone case whereas the values decreased in the 0% no-passing zone case until the vehicles reached the passing lane. The values increase because the platoons grow larger as the vehicles move through the facility. In the 0% no-passing zone case, the values decreased before the passing lane because the opposing flow rate was low enough that the following vehicles could make use of the passing zones. For the 50% case, passing was not allowed on the links leading up to the passing lane section. Therefore the *PTSF* values grew larger and larger until the passing lane relieved the following vehicles. There was no major difference between the 0% and 10% heavy vehicles cases as shown in Figure 4-38 and Figure 4-39.
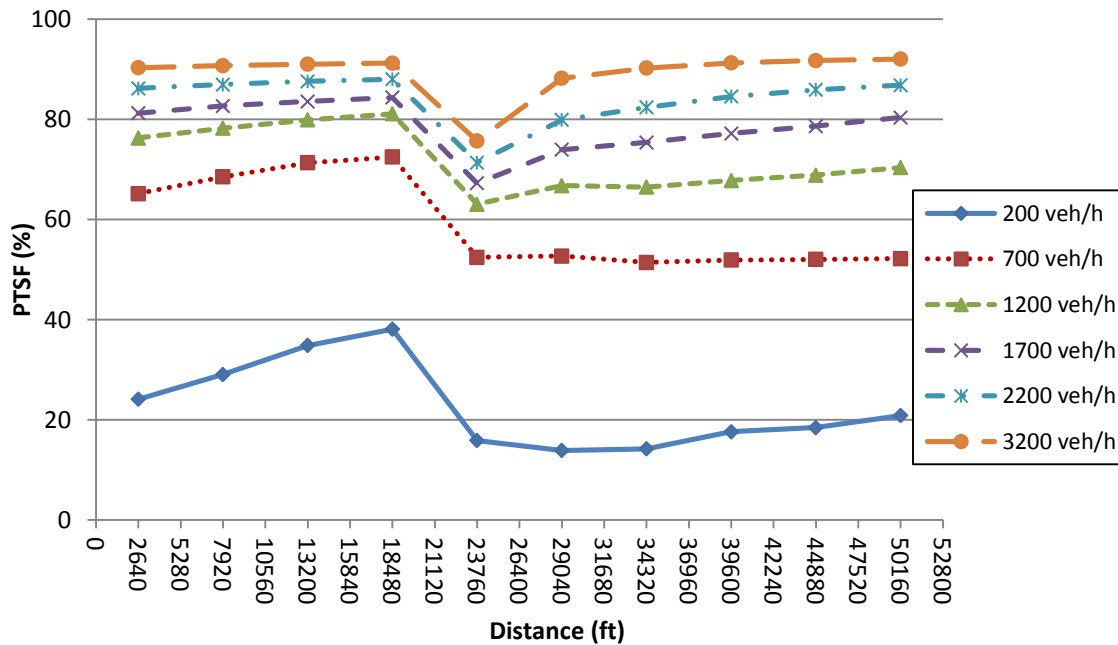

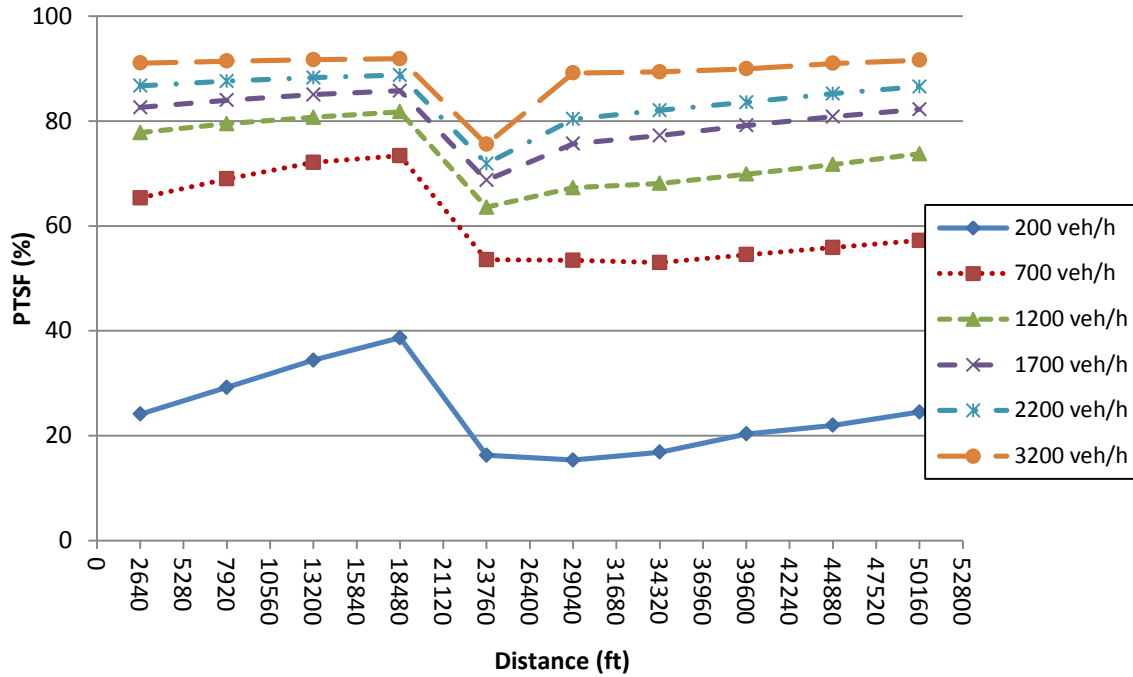Figure 4-38. *PTSF* vs. distance - 0% grade, 50% NPZ, 0% HV, passing lane

Figure 4-39. *PTSF* vs. distance - 0% grade, 50% NPZ, 10% HV, passing lane

For the 0% grade, the *ATS* values increased more sharply due to the effects of the passing lane for the flow rates of 200 veh/h and 700 veh/h than in the 0% no-passing zone case. This is because the links prior to the passing lane link were specified as no-passing zones. Slow trucks on those links caused all the following vehicles to travel at lower speeds until they were finally able to pass on the passing lane link. All flow rates showed a small improvement after the passing lane section for both heavy vehicle percentages. The plots are shown in Figure 4-40 and Figure 4-41.
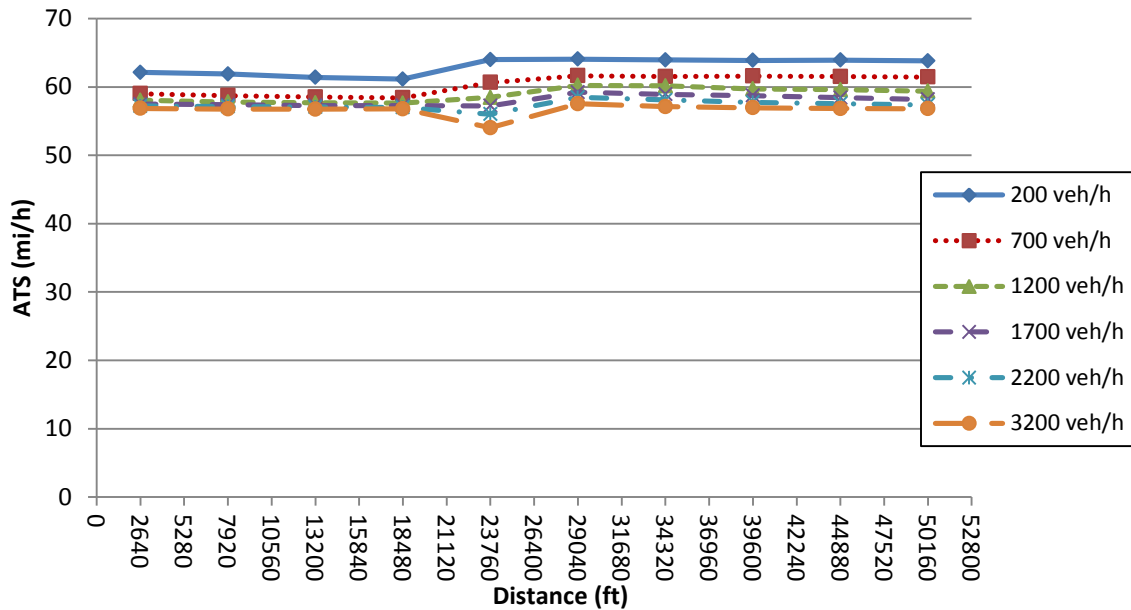
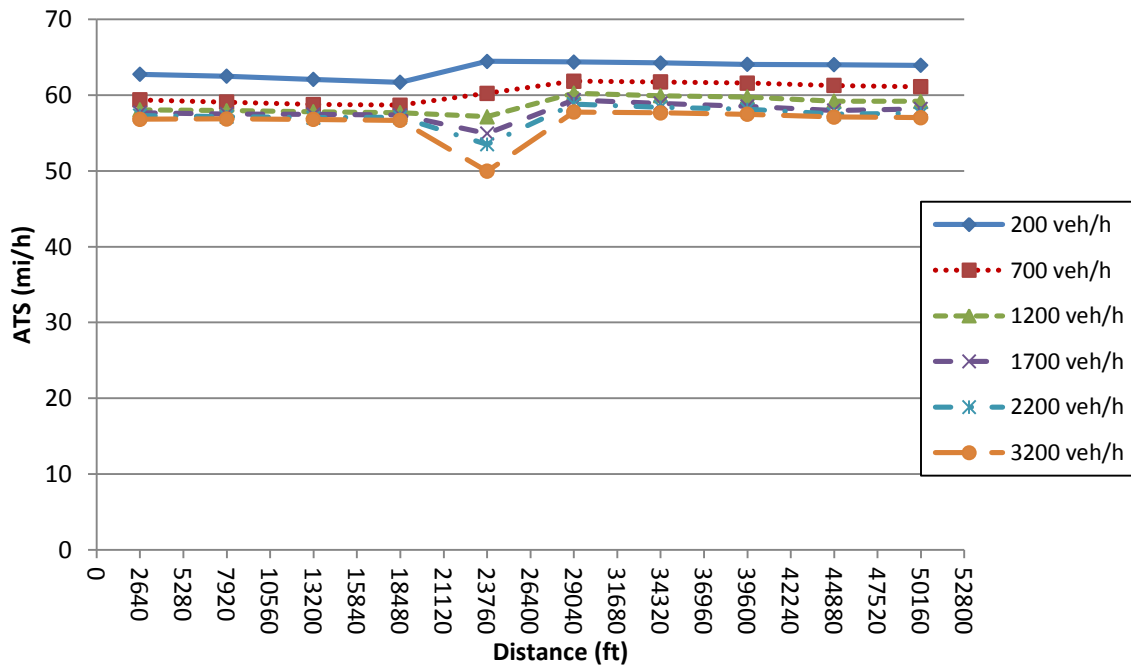Figure 4-40. *ATS* vs. distance - 0% grade, 50% NPZ, 0% HV, passing lane

Figure 4-41. *ATS* vs. distance - 0% grade, 50% NPZ, 10% HV, passing lane

For the 6% grade, the curves for the *PTSF* plot for 0% heavy vehicles are spaced out with the spacing between them getting smaller as the flow rate increases as shown in Figure 4-42. The *PTSF* values are much higher just before the passing lane. The 200 veh/h curve reached 36.2%, but in the 0% no-passing zone case it only reached 23.8%. The passing lane effects last longer for this no-passing zone case. The *PTSF* values did not return to the values they were at prior to the passing lane section as quickly. For 10% heavy vehicles, the values are the same as the

corresponding 0% no-passing zone case except the 200 veh/h curve only returns to 67.4% by the end of the facility rather than 73% as it did for 0% no-passing zones. The plot is shown in Figure 4-43.
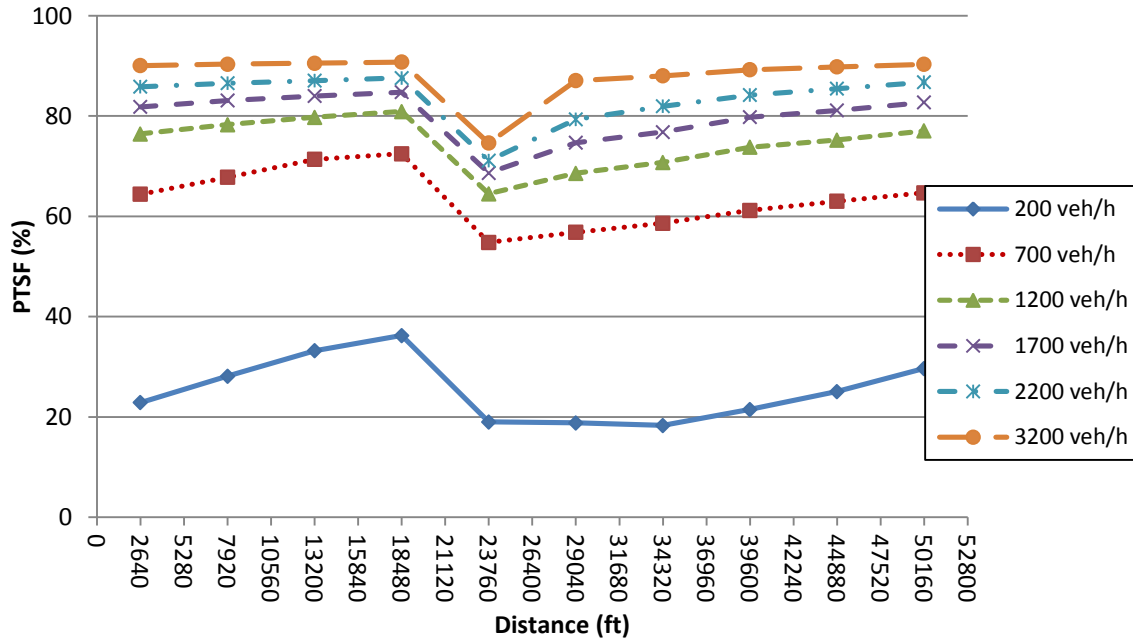

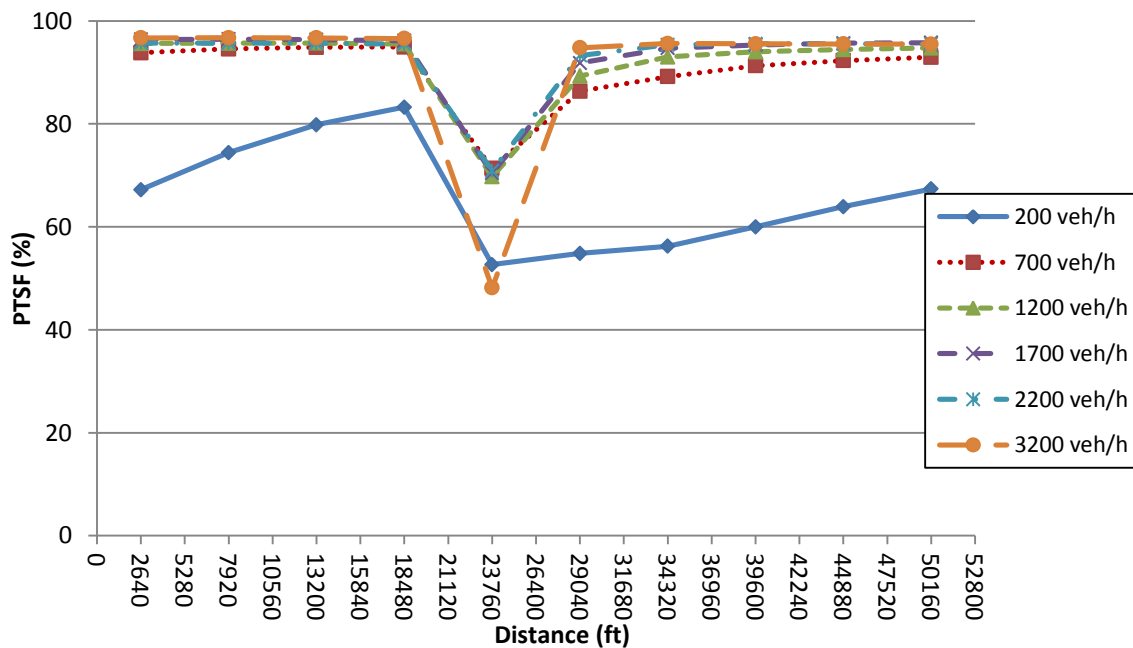Figure 4-42. *PTSF* vs. distance - 6% grade, 50% NPZ, 0% HV, passing lane

Figure 4-43. *PTSF* vs. distance - 6% grade, 50% NPZ, 10% HV, passing lane

For the 6% grade, the *ATS* plots have the same general trend as in the 0% no-passing zone case for 0% heavy vehicles. The curve for the flow rate of 3200 veh/h drops to a lower value at the passing lane than it did in the 0% no-passing zone case and the 200 veh/h flow curve increases after the passing lane as shown in Figure 4-44. For 10% heavy vehicles, the *ATS* values reached the highest point on the link after the passing lane for the flow rate curve for 200 veh/h. That link is a passing zone and the vehicles that come from the passing section have just gotten away from slow trucks. The combination of these two factors is the cause of the *ATS* highpoint happening on link 6. For the flow curves for 2200 veh/h and 3200 veh/h, the highest point occurs on link 4, which is the link just before the passing zone section. The traffic is extremely congested prior to the passing lane because of the few trucks in the traffic stream impeding all of the other vehicles. As soon as a slow truck reaches the passing lane, the other vehicles are unimpeded and begin traveling at high speeds. This creates a shockwave that propagates through the links prior to the passing lane and link 4 is affected as shown in Figure 4-45.
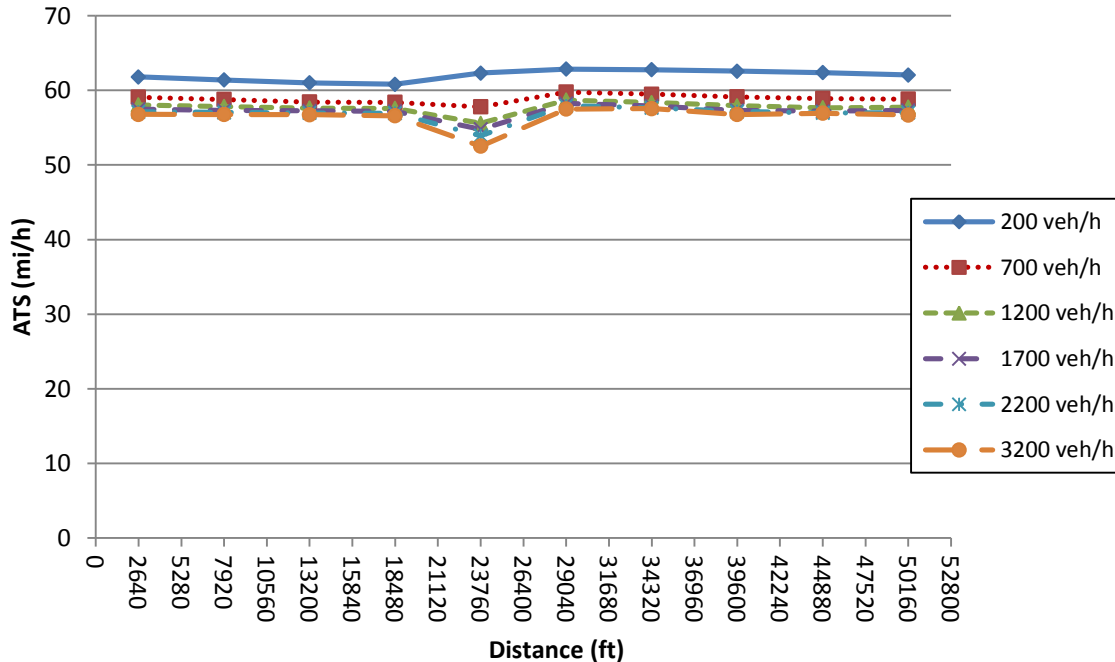

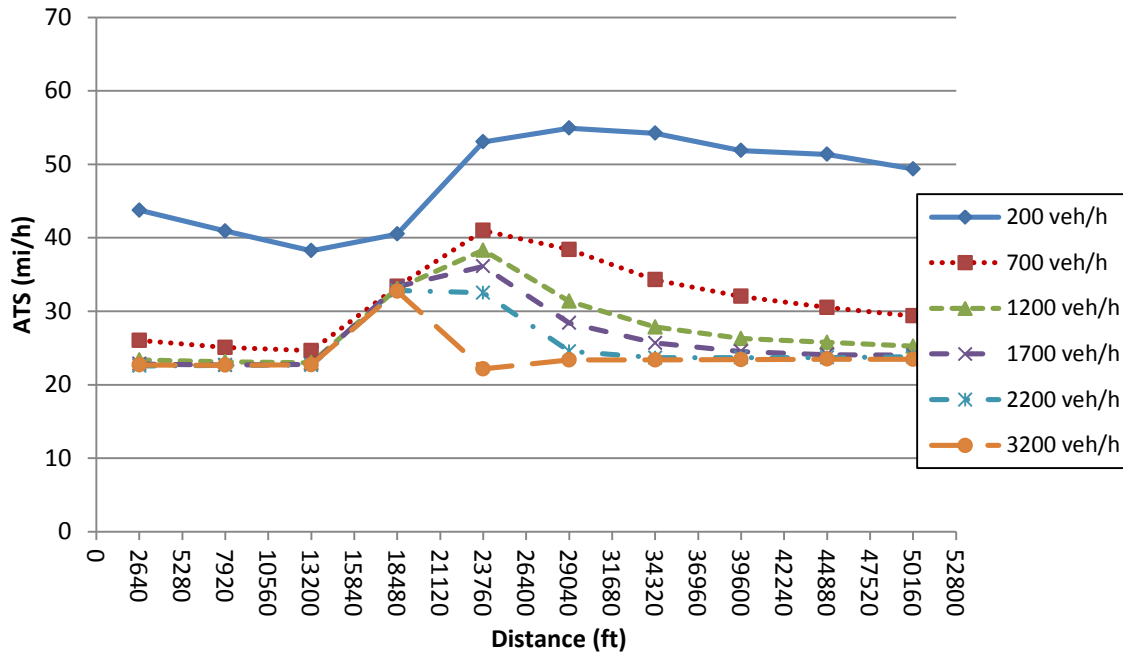Figure 4-44. *ATS* vs. distance - 6% grade, 50% NPZ, 0% HV, passing lane

Figure 4-45. *ATS* vs. distance - 6% grade, 50% NPZ, 10% HV, passing lane

3. 100% no-passing zones

For the 0% grade, the *PTSF* plots for both heavy vehicle percentages have the same trends and values as the 50% no-passing zone case until after the passing lane. The flow curves return more quickly to the values they were at prior to the passing lanes for this no-passing zone case as shown in Figure 4-46 and Figure 4-47. The passing lane effects do not carry on to the downstream links when passing is not allowed. The *ATS* trends are also similar to the 50% no-passing zone case for both percentages of heavy vehicles except the values return quickly to the values they were at just before the passing lane. The *ATS* plots are shown in Figure 4-48 and Figure 4-49.
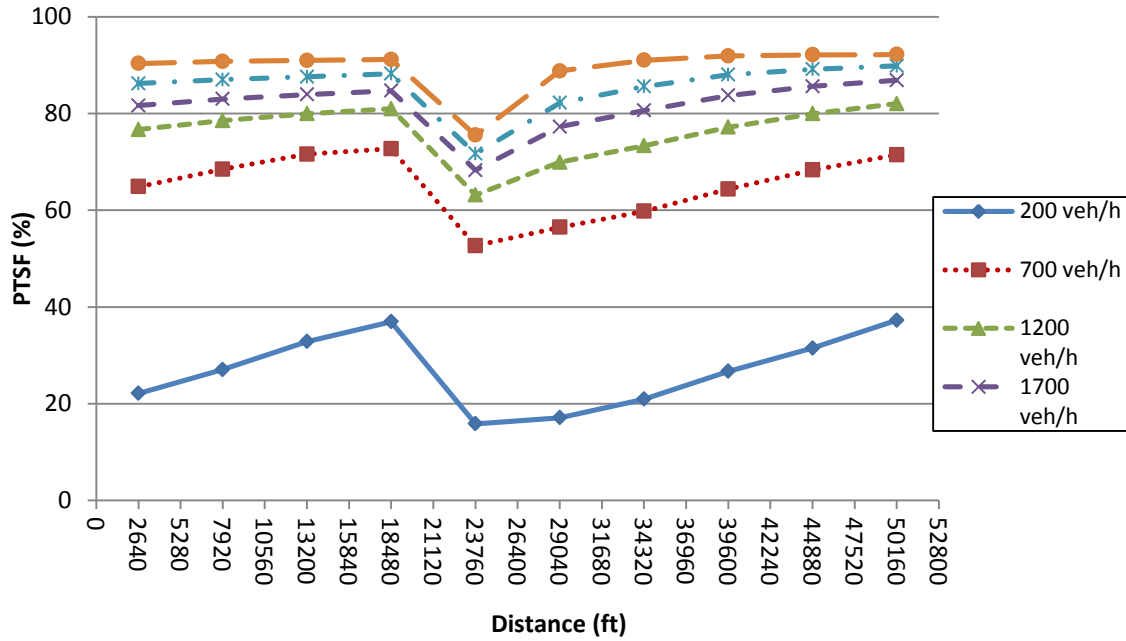
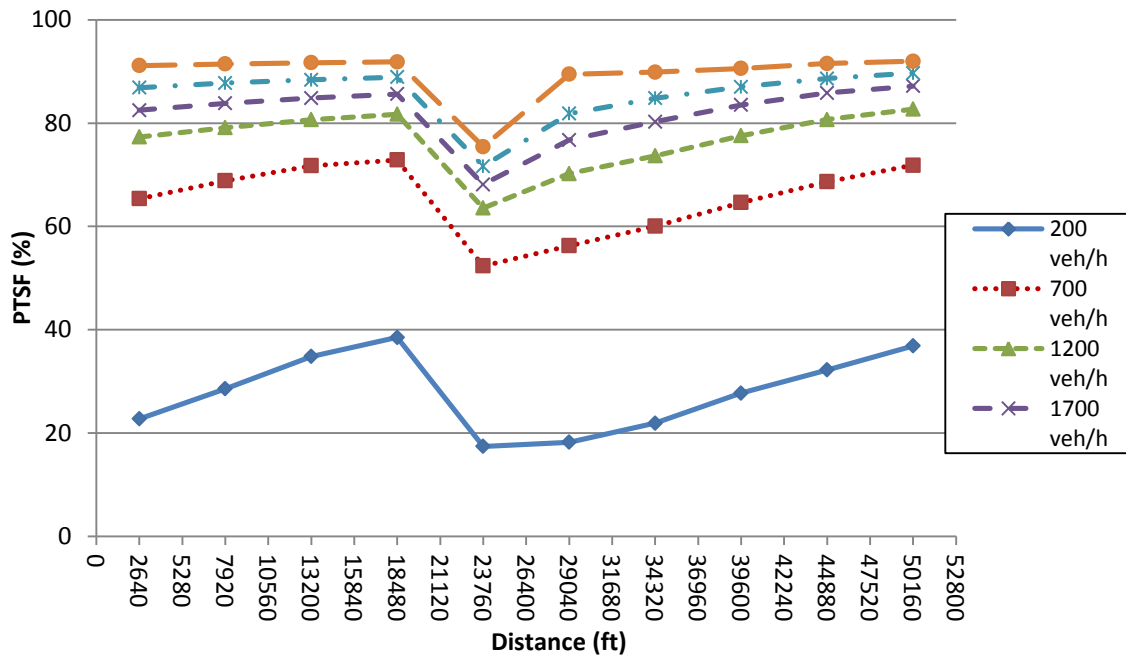Figure 4-46. *PTSF* vs. distance - 0% grade, 100%NPZ, 0% HV, passing lane

Figure 4-47. *PTSF* vs. distance - 0% grade, 100%NPZ, 10% HV, passing lane

Figure 4-48. *ATS* vs. distance - 0% grade, 100% NPZ, 0% HV, passing lane

Figure 4-49. *ATS* vs. distance - 0% grade, 100% NPZ, 10% HV, passing lane

For the 6% grade, the *PTSF* results have the same patterns as the 0% grade for 0% heavy vehicles. The 10% heavy vehicles case has the same trends as the 50% no-passing zone case, but for the 100% no-passing zone scenario, the *PTSF* values after the passing lane for the flow rate

of 200 veh/h return quickly to the values they were at before the passing lane section. The plots are shown in Figure 4-50 and Figure 4-51.

Figure 4-50. *PTSF* vs. distance - 6% grade, 100% NPZ, 0% HV, passing lane

Figure 4-51. *PTSF* vs. distance - 6% grade, 100% NPZ, 10% HV, passing lane
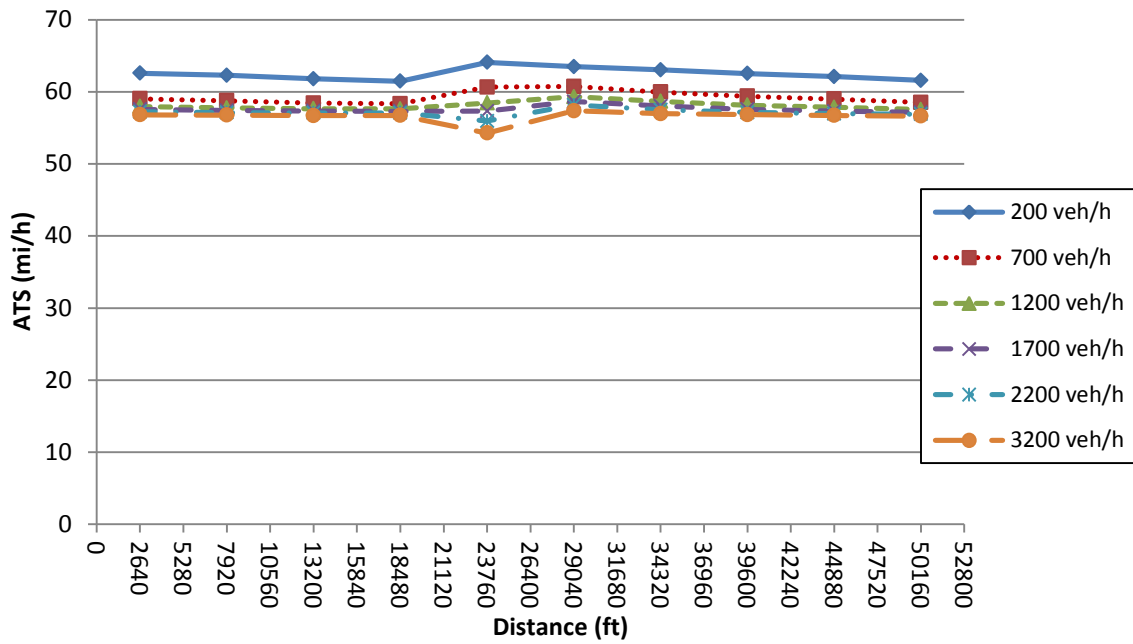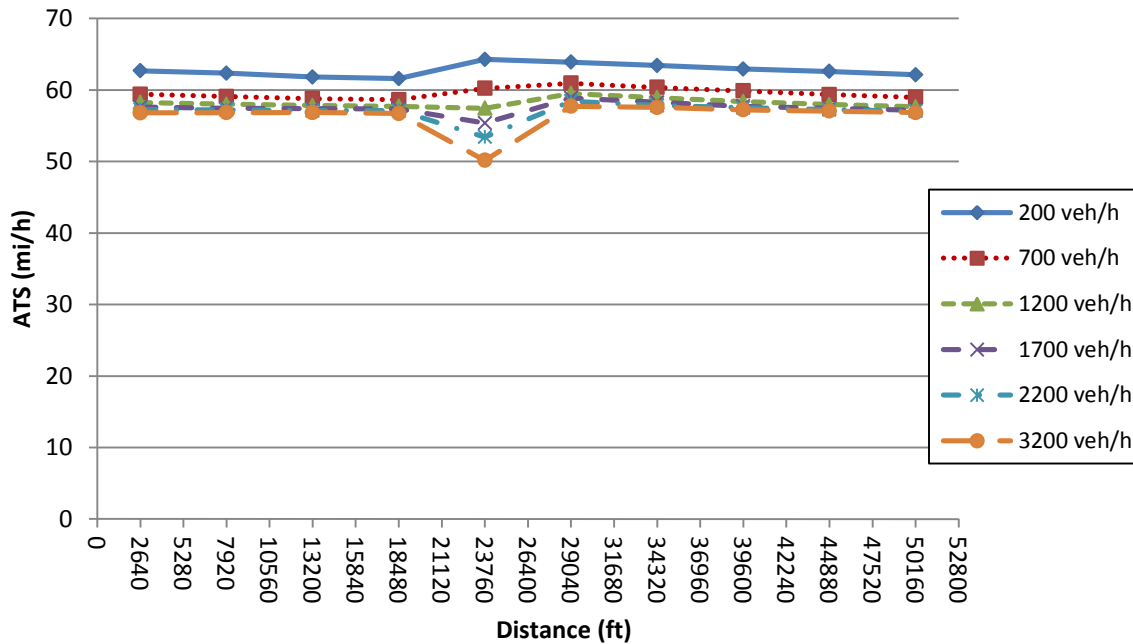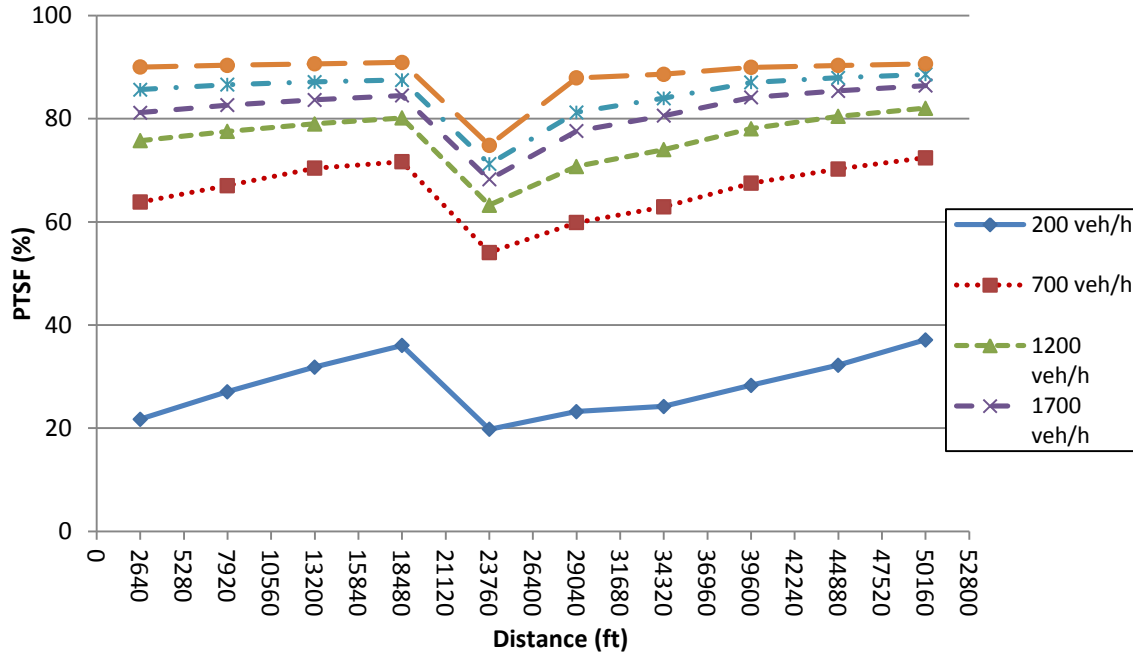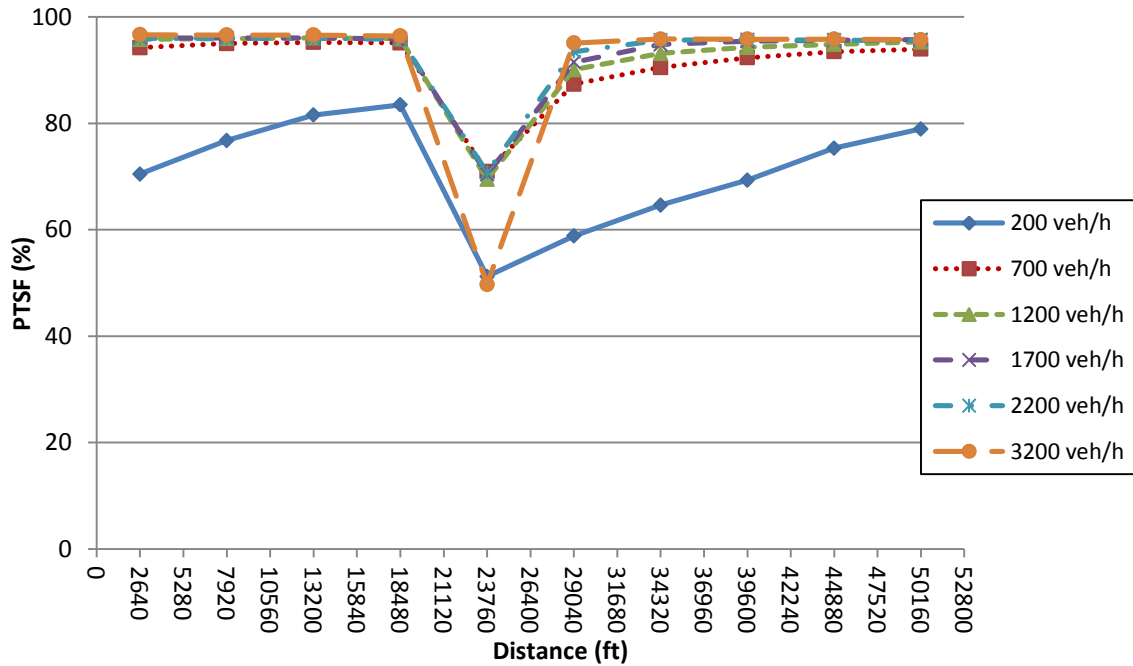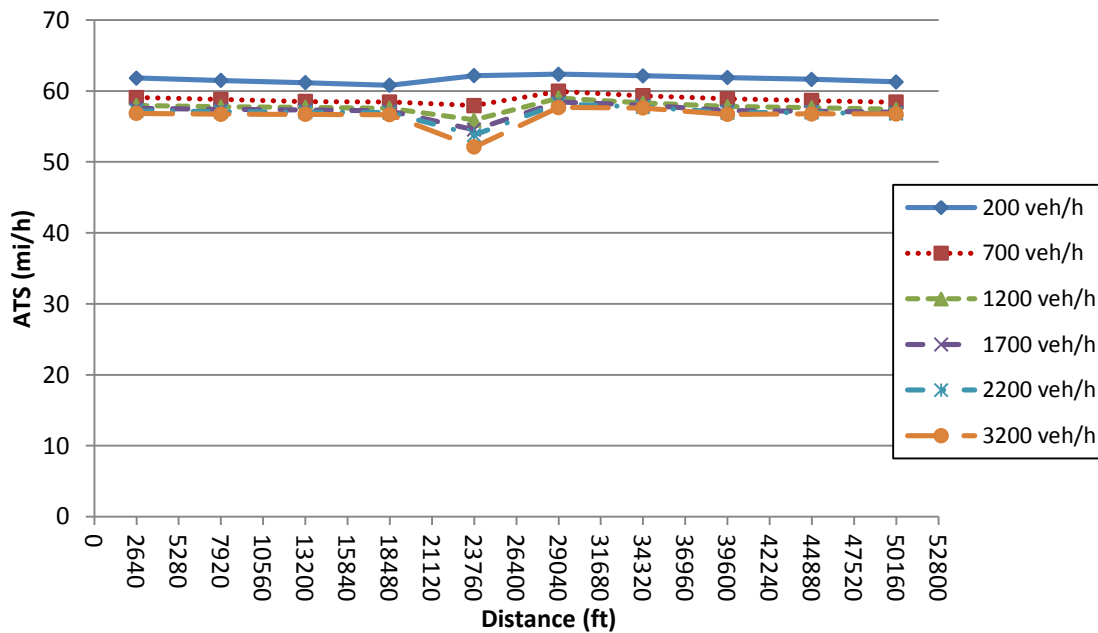
Figure 4-52. *ATS* vs. distance - 6% grade, 100% NPZ, 0% HV, passing lane
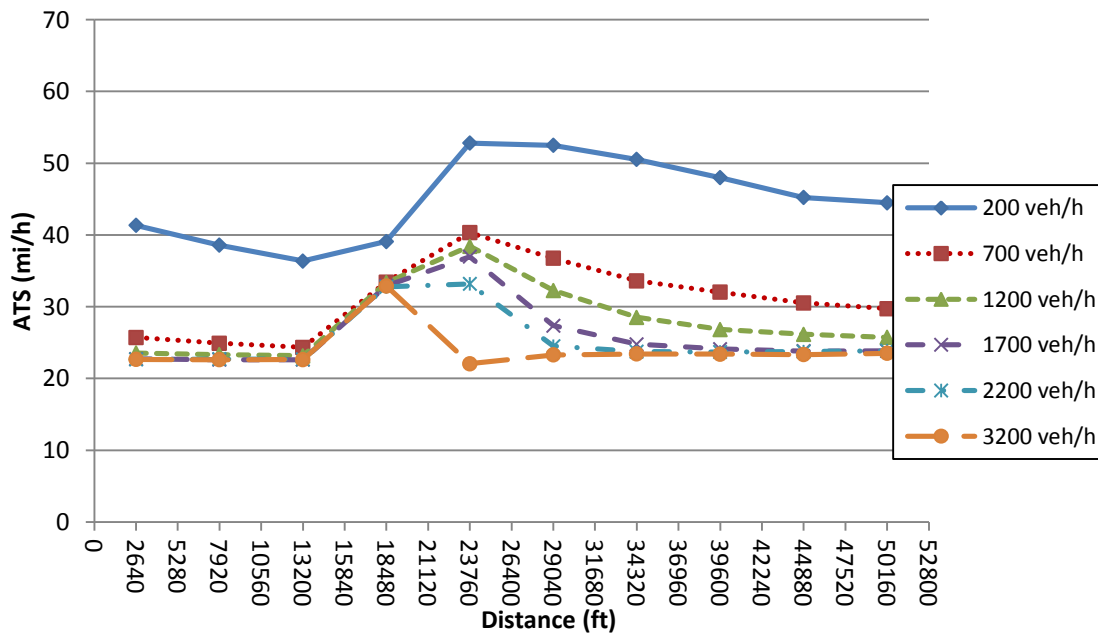
Figure 4-53. *ATS* vs. distance - 6% grade, 100%NPZ, 10% HV, passing lane

CHAPTER 5   SUMMARY AND RECOMMENDATIONS

5.1   Summary

In this project, based upon the new two-lane highway 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, which included 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 used to determine upstream and downstream intersection influence areas were developed based upon individual vehicle trajectories, 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 in determining the level of service of a two-lane highway facility including signalized intersections. A percent delay based LOS criteria derived from simulation results 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 simulation experiment results indicate that the relationship between follower density 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.

Furthermore, in this project, a number of experiments were executed with the new version of CORSIM to determine the basic relationship between "percent time-spent-following" and traffic flow rate and "average travel speed" and traffic flow rate, the two primary performance measures in the HCM 2010 two-lane highway analysis methodology. The experiment results show that the speed-flow relationship between the HCM and CORSIM did not match up well. Other sources (Luttinen 2000, Brilon and Weiser 2006) indicate that the speed-flow relationship is not linear, and CORSIM further supports those claims. Therefore, the HCM methodology should be modified so that the speeds level off as the flow rate increases. The difference between the *PTSF* results was found to be minimal, even though the two tools (the HCM methodology and CORSIM) have different procedures for calculating the *PTSF*. The HCM method uses a regression equation for finding *PTSF* that is based on TWOPAS simulation results. CORSIM provides a true estimate of *PTSF* in that it records every vehicle's follower status at every time step and finds the percentage of time that each vehicle was in a following state. Then, for the average facility *PTSF*, CORSIM takes the sum of the following time across all vehicles divided

by the sum of the time spent in the network by all vehicles. The TWOPAS simulation *PTSF* calculation is most likely similar to the one used in CORSIM, which is probably the reason for the small *PTSF* differences between the HCM and CORSIM. This general agreement in *PTSF* results does provide some support to the validity of the HCM estimation method.

For grades of 6%, the HCM and CORSIM comparison showed very different results for the no-passing lane cases with 10% heavy vehicles. CORSIM had much higher values for *PTSF* and much lower values for *ATS* than the HCM for this condition. This is largely a function of the truck passenger-car equivalency (PCE) values in the HCM 2010 not being as punitive to traffic operations as the treatment of truck performance is in CORSIM.

5.2   Recommendations

Certainly, a very desirable future study is to use field data to validate the new methodology for two-lane highway facilities with signalized intersections. Due to the 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 speed distribution, may affect the upstream/downstream effective length models in the methodology. Thus, adjustments might be necessary when applying the methodology in practice.

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 HCM and CORSIM provide very similar results for *PTSF*. However, research by Luttinen (2001) found that the HCM *PTSF* estimates generally overestimated the PTSF as determined from Finnish field data. Again, the HCM estimation equation is based on TWOPAS simulation results, and the results of this study are based strictly on CORSIM simulation results. And while some amount of field data was used to calibrate and validate TWOPAS, there is still clearly a need to do more analysis of *PTSF* results from field data. It is also possible that U.S. field data results may compare more favorably to the simulation results due to driver behavior differences with Finnish drivers.

The LOS criteria for two-lane highways could benefit from further 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, as 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. It is also recommended that the current CORSIM control delay algorithm be revised to be consistent with the signalized intersection upstream/downstream effective length algorithms, as they share the same conceptual definition.

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