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EFFECTS OF MOVING BOTTLENECKS ON TRAFFIC OPERATIONS ON FOUR-LANE LEVEL FREEWAY SEGMENTS

by

Jianan Zhou

A DISSERTATION

Presented to the Faculty of
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Under the Supervision of Professors Laurence R. Rilett and Elizabeth G. Jones

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EFFECTS OF MOVING BOTTLENECKS ON TRAFFIC OPERATIONS

ON FOUR-LANE LEVEL FREEWAY SEGMENTS

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University of Nebraska, 2018

Advisor: Laurence R. Rilett (Chair), Elizabeth G. Jones (Co-chair)

The Highway Capacity Manual (HCM) was developed to provide capacity and level of service analyses for roadway facilities. Trucks may adversely affect the quality of traffic flow on a roadway. In HCM, the passenger car equivalent (PCE) of a truck, which represents the number of passenger cars that have an equivalent effect on traffic flow, is used to account for the impacts of trucks.

However, in the past ten years rural freeways in the western rural U.S. have experienced conditions that lie outside the standard HCM conditions. Also, the current HCM truck PCEs may not be appropriate for the western rural U.S. This is because, the interstates in the western rural U.S. consistently experience truck percentages in an excess of 25 percent, but the highest truck percentage published in current HCM is 25 percent. Additionally, there are large free-flow speed differences between heavy trucks and passenger cars in western rural U.S., however, the current HCM estimates the PCEs under the assumption that trucks maintain the same speed as passenger cars on level terrain. Compounding the above two issues, trucks passing other trucks at low speed differentials may cause moving bottlenecks.

This dissertation proposed a definition, developed identification methods for the moving bottlenecks on four-lane freeway segments, and developed metrics for measuring their effects. Then, this dissertation calculated PCEs under western rural U.S. traffic flow

conditions with localized congestion caused by moving bottlenecks, by equal-density and equal-capacity method. Finally, this dissertation explored the impacts of changes in speed limits, truck passing restriction and data aggregation interval on PCEs.

The results demonstrate moving bottlenecks have an adverse effect on vehicles on the freeway. It was found that the PCE values in the HCM 2010 and HCM 2016 underestimate the effect of heavy trucks on level terrain freeways that experience high truck percentage, and where different vehicle types have large differences in average free-flow speeds. The results also show that speed limits, percentage of truck passing restriction, and data aggregation interval significantly affect the PCEs. The results will be helpful in understanding how trucks affect passenger cars in moving bottlenecks.

DEDICATION

To my beloved families and friends

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CHAPTER 1 INTRODUCTION

1.1 Problems Introduction

Trucks may adversely affect the quality of traffic flow on a roadway due to the fact that

1) The average space occupied by a truck is greater than that of a passenger car;
2) The vehicle performance (e.g. acceleration, deceleration, maneuverability, operating speed, etc.) of trucks are typically lower than that of passenger cars.

In the Highway Capacity Manual (HCM), the passenger car equivalent (PCE) of a truck, which represents the number of passenger cars that would have an equivalent effect on the traffic flow as a given truck type (1), is used to account for the presence of trucks in a traffic flow. PCEs allow a heterogeneous mix of vehicles in a traffic stream to be expressed as a standardized, homogenous traffic stream of passenger cars.

However, the HCM truck PCE values may not be appropriate for the western rural U.S. This is because

1) The interstates in western rural U.S. consistently experience truck percentages in an excess of 25 percent, but the highest truck percentage value published in either HCM 2010 or HCM 2016 is 25 percent;
2) There are large free-flow speed differences between heavy trucks and passenger cars in western rural U.S., however, the HCM 2010 and HCM 2016 estimate the PCEs under the assumption that trucks maintain the same speed as passenger cars on level terrain (2)(3);

3) Compounding the above two issues, trucks passing other trucks at low speed differentials (e.g., a 67 mph truck passing a 66 mph truck) may cause moving bottlenecks.

Thus, this dissertation aims at estimating new PCEs under the western rural U.S. traffic conditions with localized congestion caused by moving bottlenecks.

1.2 Research Objective and Tasks

Because of the problems above, the main objectives in this dissertation include

1) Proposing a definition, developing identification methods for the moving bottlenecks on four-lane freeway segments, and developing metrics for measuring their effects;
2) Calculating PCEs under western rural U.S. traffic flow conditions with localized congestion caused by the moving bottlenecks. The results should be compared and evaluated with the values in corresponding original research;
3) Exploring the impacts of changes in speed limits, truck passing restriction and data aggregation interval on PCEs.

To achieve the research objectives above, there are eight tasks in this dissertation:

Task 1: Perform a literature review and analyze current issues (Chapter 2);

Task 2: Propose hypotheses (Chapter 3);

Task 3: Define study area, collect field data, and preliminary analysis (Chapter 4);

Task 4: HCM-standard operating analysis with empirical data (Chapter 5);

Task 5: Moving bottlenecks analysis with empirical data (Chapter 6);

Task 6: Developing simulation model (Chapter 7 and Chapter 8);

Task 7: Calculating PCEs based on existing methods under western rural U.S. conditions and making comparison with the original research (Chapter 7 and Chapter 8);

Task 8: Exploring the impacts of speed limits, truck passing restriction and data aggregation interval on PCEs (Chapter 9).

The results will be helpful in understanding how trucks affect passenger cars and how moving bottlenecks affect traffic flow on four-lane level freeway segments.

1.3 Definition of "Western Rural U.S." Freeway Segments

This research focuses on freeway segments in the western rural U.S. The term "western rural U.S." freeway segment is defined according to the four criteria outlined in Table 1.1.

Table 1.1 Criteria for western rural U.S. freeway segments

No.	Criteria
1	Passenger car and/or truck speed limit equal to or higher than 75 mph.
2	Desired free flow speeds for commercial trucks (e.g. FHWA classification 5 to 13) (4) that are lower (e.g. 5 mph) than the passenger car speed limit.
3	Truck percentages higher than 25 percent.
4	U.S. interstate, or highways designed to U.S. interstate standards (5), with two lanes per direction (divided).

There are a number of important points to note about the criteria in Table 1.1:

1) Note that interstate style roadways with two lanes in each direction (e.g. criteria 4) and with speed limits of 75 mph or higher (e.g. criteria 1) are mainly found in the western U.S. (6) as shown in Figure 1.1 (6-28).

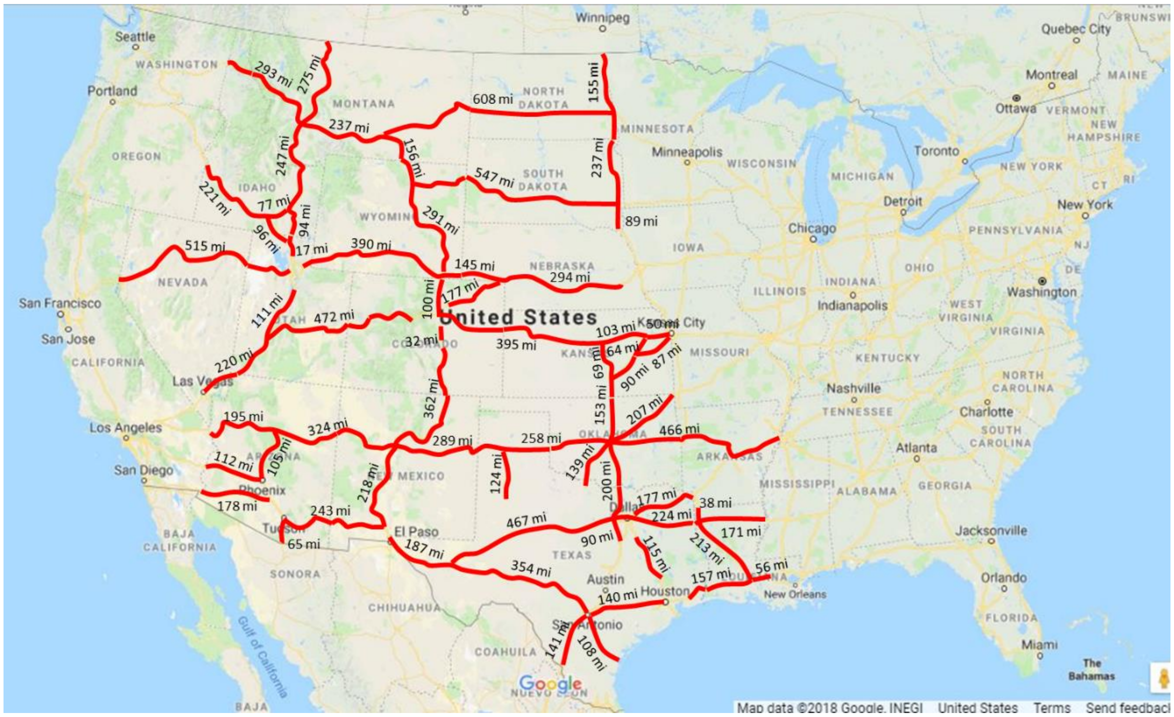


Figure 1.1 Interstate with two lanes per direction and speed limit of 75 mph or higher in western U.S.

2) Criteria 2 may occur as a result of a) speed limiters implemented by the truck owners, and/or b) speed limit for trucks lower than the speed limit for passenger cars.

3) Truck percentages higher than 25 percent (e.g. criteria 3) are relatively rare in areas of the U.S. where the population density is relatively high. However, in many locations in the western U.S., particularly where population density is low,

the high truck percentages can be relatively high. The highways shown in red in Figure 1.2 indicate the locations on the interstate system where truck percentages may exceed 25 percent on a regular basis (7-28). High truck percentages rarely, if ever, occur in urban areas. Regardless, freeway segments that experience low passenger car volumes and relatively high truck volumes, where the trucks travel as considerably lower speeds, are susceptible to the creation of moving bottlenecks as will be discussed in detail in Chapter 6.

Figure 1.2 Interstate with truck percentage higher than 25 percent in western U.S.

Based on the criteria in Table 1.1 the “western rural U.S. freeway” segments that are the focus of this dissertation are shown in red in Figure 1.3. Complete details regarding these segments may be found in the appendix. Note that this figure is based on available information (7-28) and may not be comprehensive. Freeways that met the four criteria listed in Table 1.1 comprise 11,826 miles of the U.S. Interstate Highway System.

Figure 1.3 Western Rural U.S. freeway segments based on criteria in Table 1.1

It should be noted this research is based on the traffic flow data collected from I-80 between Lincoln and North Platte in Nebraska, which is a typical example of the “western rural U.S.” freeway segment. It is hypothesized that the methodologies and concluding remarks in this research may be generalized to the other “western rural U.S.” freeway segments shown in Figure 1.3. Note that this is no different than the hypothesis implicit in the HCM where the PCE values that were developed for three lane freeways and modeled using a traffic simulation model calibrated to conditions found on the east coast of the U.S. are assumed to apply across the U.S. The hypothesis in this dissertation is that the PCE values developed in this research will be better estimates of the PCE values on the western rural U.S. freeways than the 2016 HCM values. If there are any doubts about which PCE values to use the methodology used in this dissertation may be readily used by traffic agencies to develop PCE values for their local conditions.

Lastly, the methodology discussed in this dissertation was developed from empirical data obtained from level terrain as defined in the 2016 HCM (3). For rolling and mountainous terrain the methodology can be replicated relatively easily. Note that Figure 1.3 does not differentiate among level, rolling and mountainous terrain.

CHAPTER 2 BACKGROUND, LITERATURE REVIEW AND ISSUE STATEMENTS

2.1 Trucks Definitions and Characteristics

The FHWA 13-Category Rule Set standardized vehicle classification system, developed by FHWA in the mid-1980s, currently serves as the basis for state vehicle classification counting efforts (4). The FHWA classification is shown in Figure 2.1. Vehicles are classified into the following 6 groups: motorcycles (class 1); passenger cars, pickups, vans, and vehicles with trailers (classes 2 and 3); buses (class 4); single-unit trucks (classes 5, 6, and 7); recreational vehicles (class 5); and heavy trucks (classes 8 to 13).

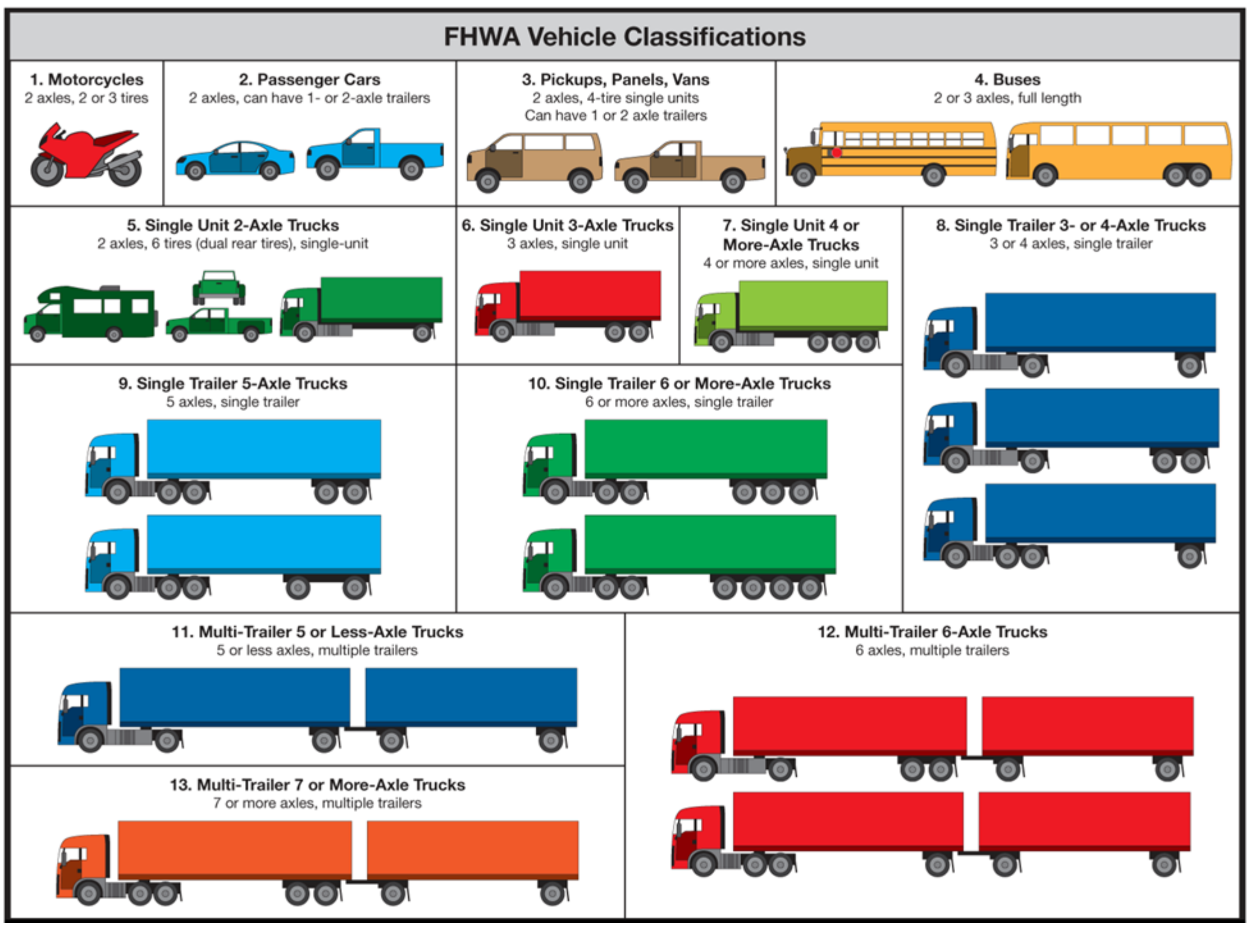


Figure 2.1 FHWA 13-category vehicle classifications

2.2 Characteristics of Traffic Flow on Interstate 80 in Nebraska

In Nebraska, Interstate 80 (I-80) is 456 miles in length. This research focuses on the section between the milepost 177 near to North Platte and the milepost 399 near to Lincoln. According to Statewide Traffic Flow Map for Nebraska in 2014 (7), this section the AADT (average annual day traffic volume) is between 15,265 veh/d and 25,930 veh/d. The AADT for trucks is between 6,960 veh/d and 8,490 veh/d. The truck percentage based on AADT ranges between 31% and 49%. The Statewide Traffic Flow Map for Nebraska in 2014 shows that

1) The traffic volume decreases from east to west, and the decrease rate of passenger car volumes are higher than truck volumes;
2) The truck percentage increases while the passenger car percentage decreases from east to west, and the truck percentage is higher than 25% for the entire section;
3) The higher truck percentage usually appears with lower traffic volume and vice versa.

2.3 Research on Moving Bottlenecks Identification

Moving bottlenecks are defined as the queuing caused by a slow-moving vehicle during periods of moderate to heavy demand (29). Such queuing can occur not only on a single lane of traffic, but also on multilane highways, even though only one of the lanes appears to be obstructed by the slow moving vehicle. In general, a moving bottleneck is caused by the fast vehicles catching up with the slower vehicles and not being able to pass. Vehicles that do not catch up with other vehicles or are not impeded by other vehicles are defined as “free vehicles.” One accepted definition of a free vehicle is that its speed is not influenced by the speed of the vehicle traveling ahead of it (30 and 31). In this research, a moving bottleneck is defined as a group of vehicles traveling on either the median or the shoulder lane in the same direction, where one influences the speed of the other.

The identification for moving bottlenecks on two-lane highway, referred to as platoons, has been widely researched. Platoons are usually identified by headways between the leading and following vehicles. In order to be considered as a vehicle in platoons, the headways should not be greater than a specific threshold (32 and 33). The threshold for platoon identification is defined as the “critical headway”. The values of critical headway vary among researchers. A detailed explanation of critical headway may be found in Table 2.1. The previous research shows that the critical headway varies among different road type and traffic conditions.

Table 2.1 Critical headway values in previous research

Literature	**Critical Headway Value**
HCM 2010 and HCM 2016(2)(3)	3s (two-lane highway)
Miller 1961(87)	8s (two-lane highway)
Edie et al 1963(88)	4s to 5s (two-lane highway)
Keller 1976(89)	2s (two-lane highway)
Al-Kaisy and Karjala 2010(31)	6s (two-lane rural highway)
Fitzpatrick et al 2004(90)	5s for leading headway, 3s for lagging headway (two-lane highway)

2.4 Research on Metrics for Level-of-Service of Uninterrupted Traffic Flow

In HCM 2010 and HCM 2016, for freeway and multilane highway segments, the density is used as the measure of effectiveness for level-of-service (LOS). For two-lane highway segments, the LOS measures of effectiveness include average travel speed (ATS), percent time spent following (PTSF), and percentage of free-flow speed (PFFS). In previous research other popular LOS metrics include the average travel speed of passenger cars (ATSPC), the percentage of free-flow speed of passenger cars (PFFSPC), travel time, travel delay, platoon length, platoon flow, percent followers, and follower density (34,35, and 36). The ATSPC and the PFFSPC have been identified as better metrics than ATS and PFFS because the passenger cars are more affected by high traffic volumes than heavy vehicles and therefore more accurately describe the speed reduction of passenger cars (37). The explanations for each metric are summarized in Table 2.2.

Table 2.2 Explanation of metrics for level-of-service of uninterrupted traffic flow

Metric	Explanation
Density	The number of vehicles occupying a given length of a lane or roadway at a particular instant and reflects the degree of congestion.
Average travel speed (ATS)	The highway segment length divided by the average travel time taken by vehicles to traverse it during a designated time interval, reflecting the mobility on two-lane highway.
Percent time spent following (PTSF)	The average percentage of time that vehicle must travel in platoons behind slower vehicles due to the inability to pass, representing the approximate percentage of vehicles traveling in platoons and the freedom to maneuver and the comfort and convenience of travel.
Percentage of free-flow speed (PFFS)	The ability of vehicles to travel at or near the posted speed limit.
Travel time	The average travel time for vehicles passing two specific locations.
Travel delay	The difference in travel time between travel with free-flow speed and actual speed.
Platoon length	The number of vehicles in per platoon.
Platoon flow	The number of vehicles in platoons in traffic flow.
Percent followers	The percentage of vehicles with short headway in the traffic stream.
Follower density	The number of followers per mile per lane, represents the feeling of congestions experienced by impeded vehicles suffered in platoons.

2.5 Research on Passenger Car Equivalents

2.5.1 Overview

The concept of passenger car equivalents (PCE) was first proposed in the 1950 HCM. PCE was first used for multilane highways. The PCE values were updated and expanded to other facilities in each of the following HCM editions. According to the literature, widely used methods for PCE determination on highway, freeway, and urban roads are:

1) Equal-density method, used in HCM 2010 PCE determination;
2) Equal-capacity method, used in HCM 2016 PCE determination;
3) Equal-impedance method (e.g. equal-speed, equal volume-capacity ratio, etc.);
4) Overtaking method;
5) Headway-based method;
6) Delay-based method;
7) Platoon-based method;
8) Speed-area-based method;
9) Speed-based method;
10) Travel-time-based method;
11) Equal-flow method;
12) Queue-discharge-flow method.

The theory and logic behind these methods will be discussed in the following sections. The places where these methodologies have been used (e.g., two-lane highway, multilane highway, etc.) are summarized in Table 2.3.

Table 2.3 Summary of places that PCE methodologies used

Method	**Two-Lane Highway**	**Multilane Highway**	**Freeway**	**Urban Road**
Equal Density			Webster and Elefteriadou, 1999	
Equal Capacity			Yang 2013; Dowling et al 2014	
Equal Impedance	Huber, 1982	Okura and Sthapit, 1995; Elefteriadou et al, 1997; Torbic et al, 1997; Webster and Elefteriadou, 1999	Okura and Sthapit, 1995; Elefteriadou et al, 1997; Torbic et al, 1997; Webster and Elefteriadou, 1999	Sumner et al, 1984
Overtaking	Werner, 1976; Cunagin and Messer, 1982			
Headway	Werner and Morrall, 1976	Krammes and Crowley, 1986	Krammes and Crowley, 1986	Molina, 1987(91)
Delay		Chitturi and Benekohal, 2007	Chitturi and Benekohal, 2007	Benekohal and Zhao, 2000
Speed Area				Chandra and Sikdar, 2000
Travel Time				Keller and Saklas, 1984
Platoon	Van Aerde and Yagar, 1984			
Speed	Van Aerde and Yagar, 1984			
Equal Flow		Fan 1990; Alecsandru et al, 2012; Yeung et al 2015(92)	Fan 1990; Alecsandru et al, 2012; Yeung et al 2015(92)	
Queue Discharge Flow		Al-Kaisy et al, 2002	Al-Kaisy et al, 2002	

2.5.2 Equal-Density (HCM 2010) Method

The HCM 2010 recommends all truck PCE values for trucks as 1.5 at freeway segments with level terrain (with grade no greater than 2%) for any length and truck percentage conditions (2). Simulation data from FRESIM, which is part of CORSIM, were used for calculating these PCEs based on the "equal-density" method. The basic idea behind the equal-density method is that the PCE is determined by comparing the volume for a given mixed traffic flow to a base flow (e.g. passenger-car-only) that has the same density (1). The basic approach was augmented by Sumner in 1984 by adding the concept of subjected flow. Subjected flow means a certain number of passenger cars in the mixed-traffic flow are replaced by an equal number of subjected vehicles, which are defined as the vehicles for which a PCE will be estimated. The replacement proportion is a decision variable, usually set to 5% (38). Thus, the PCE is a function of the base, mixed and subjected vehicle traffic flows that give the same density and the replacement proportion. Figure 2.2 shows the process and key parameters.

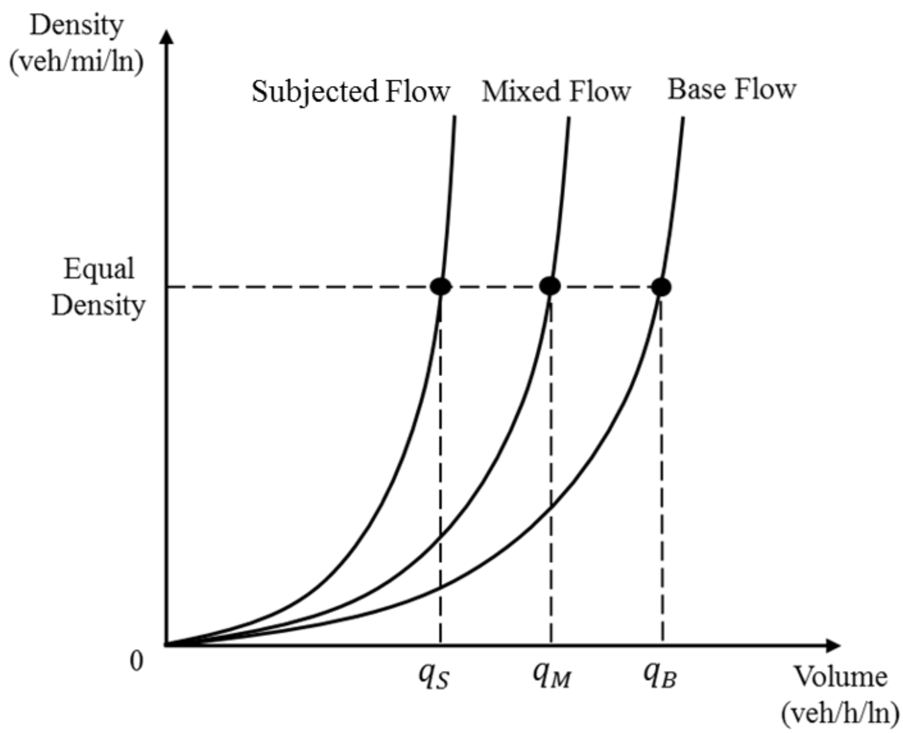


Figure 2.2 Volume-density curves for estimating PCEs using equal-density method

The equation for equal-density PCE is shown in Equation 2.1. It has been found that PCEs calculated using this method increases with grade, length of grade, traffic volume, and decreases with truck percentage.

$$ED\_PCE_{q_s,p_t} = \frac{1}{\Delta p}\left(\frac{q_B}{q_S} - \frac{q_B}{q_M}\right) + 1 \qquad (2.1)$$

$ED\_PCE_{q_s,p_t}$	Equal-density passenger car equivalents for trucks for given traffic flow volume q_S and truck percentage p_t
Δp	Percent of subjected vehicles (e.g., trucks) that replace an equivalent percentage of passenger cars in subject traffic flow (5%).

q_B Base (e.g. passenger car only) flow volume that results in same density as given traffic flow (veh/h/ln)

q_M Mixed (e.g. p_t percent trucks and (1-p_t) percent cars) flow volume that results in same density as given traffic flow (veh/h/ln)

q_S Subjected (e.g.(p_t +5) percent trucks and (1-p_t-5) percent cars) traffic flow volume (veh/h/ln)

2.5.3 Equal-Capacity (HCM 2016) Method

In HCM 2016, the PCE values are estimated based on VISSIM simulation data at one minute intervals along three lanes in each direction, fifteen mile (eight mile level plus six miles graded and one mile level) section of a freeway (3). The methodology employed an equivalency method where a PCE was identified that would make the capacity of mixed flow equal to the capacity of auto-only flow, which means the approach is designed to estimate PCEs under "equal-capacity" condition (39 and 40). The equations for equal-capacity PCE are presented in Equations 2.2 and 2.3:

$$C_{ao,g,d} = C_{mix,p,tc,g,d} * p * EC\_PCE_{p,tc,g,d} + C_{mix,p,tc,g,d} * (1-p) \qquad (2.2)$$

$$EC\_PCE_{p,tc,g,d} = \frac{1-(1-p)*CAF_{mix,p,tc,g,d}}{p*CAF_{mix,p,tc,g,d}} \qquad (2.3)$$

$EC\_PCE_{p,tc,g,d}$ EC_PCE for the mixed flow at truck percentage p, grade g, distance d, and tc truck composition

$CAF_{mix,p,tc,g,d}$	Capacity adjustment factor for the mixed flow at truck percentage p, grade g, and distance d, and truck composition tc
p	Truck percentage (between 0 to 1)
g	Grade (between -1 to 1)
d	Distance of grade (mile)
tc	Truck composition (percentage of single-unit trucks and heavy trucks)

In Equation 2.4, the capacity adjustment factor is defined by the ratio of the capacity of the mixed flow at specific truck percentage, truck composition, grade and distance, to the capacity of auto-only flow at corresponding grade and distance, as the following equation:

$$CAF_{mix,p,tc,g,d} = C_{mix,p,tc,g,d}/C_{ao,g,d} \qquad (2.4)$$

$CAF_{mix,p,tc,g,d}$	Capacity adjustment factor for the mixed flow at truck percentage p, grade g, distance d, and truck composition tc
$C_{mix,p,tc,g,d}$	Capacity adjustment factor for the mixed flow at truck percentage p, grade g, distance d, and truck composition tc
$C_{ao,g,d}$	Capacity adjustment factor for the auto-only flow at grade g and distance d
p	Truck percentage (between 0 to 1)
g	Grade (between -1 to 1)
d	Distance of grade (mile)
tc	Truck composition (percentage of single-unit trucks and heavy trucks)

The capacity adjustment factor for different simulation scenarios can be calculated by the equation above using simulation data. The capacity adjustment factor for the auto-only flow is always 1 according to the definition. In the PCEs calculation procedure in HCM 2016, the capacity adjustment factors for different conditions are estimated by a series of non-linear regression models developed based on the simulation data. The models are developed with dependent variable $CAF_{mix,p,tc,g,d}$ at different simulation scenarios, and with independent variables truck percentage p, grade g, distance d, and truck composition tc. The details for the capacity adjustment factor estimation models are discussed in Chapter 8.2 and 8.3.

In HCM 2016, a value of 2.0 was recommended for general level terrain. The recommended PCEs for level freeway segments with zero grade are disaggregated based on the truck percentage and truck composition, ranging from 1.83 to 2.62 (3). For all grades, the PCEs are recommended increasing with the percentage of heavy trucks, the grade and the grade length, and decreasing with the total truck percentage.

2.5.4 Equal-Impedance Method

The basic idea behind the equal-impedance method is that the PCE is determined by comparing the volume for a given mixed traffic flow and subjected flow to a base flow (e.g. passenger-car-only) that has the same impedance. Note that any impedance metric could be used and previous research has examined speed, density, volume-capacity ratio, vehicle-hour, travel time, and passenger car travel time (44, 45, 46, and 47). Similar to Chapter 2.5.2, the subjected flow means a certain number of passenger cars in the mixed-traffic flow are replaced by an equal number of subject vehicles, which are defined as the vehicles for which a PCE will be estimated. The PCE is estimated using Equation 2.5. It is a function of the replacement proportion, Δp, and the base, mixed, and subjected vehicle traffic flows that give the same impedance value c.

$$E\_PCE = \frac{1}{\Delta p}\left(\frac{q_{Bc}}{q_{Sc}} - \frac{q_{Bc}}{q_{Mc}}\right) + 1 \qquad (2.5)$$

E_PCE	Equal-impedance passenger car equivalents for subject vehicles
Δp	Proportion of subject vehicles adding to the mixed flow and subtracted passenger cars from the mixed flow
q_{Bc}	Flow rate at impedance c for base traffic flow
q_{Mc}	Flow rate at impedance c for mixed traffic flow
q_{Sc}	Flow rate at impedance c for subjected vehicle traffic flow
Δp	Proportion of subjected vehicles adding to the mixed flow and subtracted passenger cars from the mixed flow

This methodology is best illustrated by an example. Consider Figure 2.3 where the y axis represents impedance and the x axis represents flow in terms of vehicles/hour.

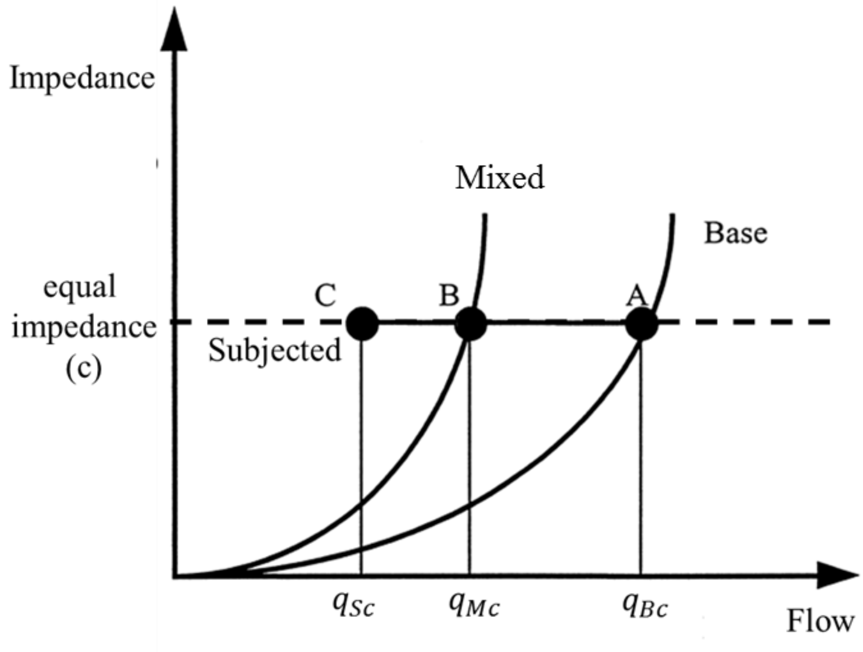


Figure 2.3 Impedance-flow relationships for equal impedance method

There are two impedance-flow curves shown in the figure. One curve (e.g. labeled "Base") represents the basic flow with only passenger cars. The other curve (e.g. labeled "Mixed") shows the mixed traffic flow for the condition of interest (e.g., 90% passenger cars and 10% trucks). As would be expected, the impedance is higher for the mixed traffic flow than the base flow for a given vehicle flow rate (e.g., equal number of vehicles). Consider the situation where there is equal impedance as shown by the horizontal line. For the base case, this is represented as point A with a flow of q_{Bc}. For the mixed flow, this is point B with a flow of q_{Mc}. This can also been seen in point C, which is a subjected flow of q_{Sc} that has same impedance as q_{Bc} and q_{Mc}. These are the values used in Equation 2.5.

This method is used for estimating PCEs using simulation data. For example, data from TWOPAS and NETSIM have been used for estimating PCE values using the equal-speed method (46 and 47). The simulation data from FRESIM has been used for calculating

PCEs based on equal-density method (1). In general, it has been found that PCEs calculated using this method increase with grade, length of grade, traffic volume, and decrease with truck percentage.

2.5.5 Overtaking Method

The overtaking method was initially proposed for calculating PCEs on two-lane highways. It was first used to estimate PCEs for two-lane highways in the 1965 HCM. In the overtaking method, traffic data is collected from a series of representative roadway sections. The data is collected at a single point and includes vehicle type and vehicle speed. The passenger cars are categorized into several cohorts based on speed. The number of cohorts can vary from 8 to 15, although 10 is recommended. The cohorts are ordered from the slowest cohort to the fastest. Trucks are categorized as belonging to a separate and single group. Note that a given vehicle is classified as either a truck or a passenger car (48 and 49). It is assumed that only passenger car cohorts that have average speeds higher than the truck cohorts will produce overtaking maneuvers. It is further assumed that only the passenger cars in the faster cohorts can overtake the passenger cars in the slower cohorts.

The PCE is defined using Equation 2.6. There are two parts to the equation. The first is the ratio of the frequency of passenger cars f_p to the frequency of trucks f_T. The second term is also a ratio. The numerator is an estimate of the number of passenger cars passing trucks. The denominator represents the estimate of the number of faster passenger cars passing slower passenger cars. The PCE is the product of the two ratios.

$$O\_PCE = \left(\frac{f_p}{f_T}\right) * \frac{\sum_{j=T+1}^{N} f_T f_j \left[\left(\frac{1}{v_T}\right) - \left(\frac{1}{v_j}\right)\right]}{\sum_{i=1}^{N} \sum_{j=i+1}^{N} f_i f_j \left[\left(\frac{1}{v_j}\right) - \left(\frac{1}{v_i}\right)\right]} \tag{2.6}$$

O_PCE	Overtaking-based passenger car equivalents for trucks
f_p, f_T	Frequency of passenger cars and trucks
f_i, f_j	Frequency of passenger cars in cohort *i* (slower vehicle) and in cohort *j* (faster vehicle)
v_T	Average speed of trucks
v_i, v_j	Average speed of passenger cars in cohort *i* (slower vehicle) and in cohort *j* . Note that by definition the cohort *j* is traveling faster than the cohort *i*

2.5.6 Headway-Based Method

Headway-based PCEs are based on the relationship between the spacing maintained by passenger car drivers in the proximity of trucks and the spacing maintained by passenger drivers in the proximity of passenger cars. It is hypothesized that these should be equivalent when considering the driver's perception of proximity to other vehicles and the freedom to maneuver. This concept is referred to as the driver's perception of equivalent densities. The assumption is that headways between passenger cars in base flow are equal to headways between passenger cars in mixed flow (50). Equation 2.7 is used for the headway-based method. The denominator, h_{pp}, represents the mean headway of passenger cars following passenger cars. The numerator is comprised of two terms. The first term is the product of the percentage of non-trucks in the traffic stream and the sum of the mean headways for passenger cars following trucks, the mean headway of trucks following passenger cars, and the negative of the mean headway of passenger cars following passenger cars. The second term is the product of the percentage of trucks in the traffic stream and the mean headways or trucks following trucks.

$$H\_PCE = \frac{(1-p)(h_{pt}+h_{tp}-h_{pp})+p*h_{tt}}{h_{pp}} \tag{2.7}$$

H_PCE	Headway-based passenger car equivalents for trucks
p	Percentage of trucks at a mixed traffic stream
h_{pp}	Mean headway for passenger cars following passenger cars (seconds)
h_{pt}	Mean headway for passenger cars following trucks (seconds)
h_{tp}	Mean headway for trucks following passenger cars (seconds)
h_{tt}	Mean headway for trucks following trucks (seconds)

If h_{tp}, h_{pt}, and h_{tt} are the same as the h_{pp}, then the PCE will be one. The more the heavy vehicle affects the headway of the following passenger car, the higher the PCE. This method is used to calculate PCE for one lane of a highway, urban road, or freeway (50). Note that either leading or lagging headways can be used in Equation 2.7. The leading headway includes the length of the vehicle and the inter-vehicle space behind the vehicle. The lagging headway includes the length of the vehicle, and the inter-vehicle space precedes the vehicle. In Krammes' research, the lagging headway is used (50).

It is assumed that headways are for vehicles that are interacting with each other. For example, vehicles that are following each other but are five minutes apart would not be used. This means that a critical headway, which is the threshold for vehicle interaction, must be defined as a *priori* knowledge.

2.5.7 Delay-Based Method

The delay-based PCE method is shown in Equation 2.8. The PCE is a ratio of the amount of delay caused by a given amount of trucks in a given flow to the delay resulting from the same flow, which consists of all passenger cars (51). In essence, the PCE represents how many passenger cars could replace a given truck and result in the same amount of delay to all vehicles.

$$D\_PCE = 1 + \frac{\Delta d_t}{d_0} \tag{2.8}$$

D_PCE	Delay-based passenger car equivalents for trucks
Δd_t	Additional delay caused per truck (seconds)
d_0	Average delay per vehicle of passenger car when truck percentage is 0% (base delay) (seconds)

This equation was initially proposed for PCE determination at signalized interactions (52). It was extended for estimating PCE on work zone areas on the highway (53). This method can only be used where this is a strict lane-following discipline (e.g., no passing).

2.5.8 Speed-Area-Based Method

Speed-area-based method is based on a ratio, as shown in Equation 2.9. The numerator is the ratio of the mean speed for passenger cars to the mean speed of trucks. The denominator is the ratio of projected rectangular areas (e.g., product of length and width) on the road for passenger cars to the projected rectangular areas on the road for trucks (54). As might be suspected from the equation, this model is used where lane discipline is not maintained and where the number of vehicles on a cross-section may be greater than the number of lanes. This occurs in many developing countries where small cars, auto rickshaws, and motorcycles have significant market penetration. This method attempts to capture the lateral and longitudinal space usage of different vehicle types.

$$SA\_PCE = \frac{V_c/V_i}{A_c/A_i} \tag{2.9}$$

SA_PCE	Speed-area-passenger car equivalents for trucks
V_c	Mean speed for passenger cars (mph)
V_i	Mean speed for vehicle type *i* (mph)
A_c	Projected rectangular areas on the road for passenger cars (ft^2)
A_i	Projected rectangular areas on the road for vehicle type *i* (ft^2)

The speed area-based method was initially proposed for estimating PCEs on Indian urban road conditions. It has subsequently been widely used in developing countries where lane-discipline is not followed and where there is a high degree of mixed traffic volume (e.g., non-motorized vehicles, two-wheeler vehicles, and three-wheeler vehicles). It is hypothesized that this method may not be appropriate for traffic conditions in the U.S. where lane discipline is universally maintained and a given vehicle will occupy the entire lane regardless of its size.

2.5.9 Travel-Time-Based Method

The travel-time-based PCE method is shown in Equation 2.10. The PCE is defined as the ratio total travel time of a given vehicle type over a section of roadway to the total travel time of the base vehicle (e.g., passenger car) over the same section (55). Note that the "section" could consist of the entire network.

$$T\_PCE_i = \frac{TT_i}{TT_b} \tag{2.10}$$

T_PCE_i	Travel-time-based passenger car equivalents for vehicle type *i*
TT_i	Total travel time of vehicle type *i* over the network (seconds)
TT_b	Total travel time of base vehicle (passenger car) over the network (seconds)

This method was proposed for PCEs on highway and urban roads. In this method, the travel time can include two parts: the travel time for the link (road midway), and the travel time for traveling through intersections (including stop delay). Because this approach is very data intensive, it has historically been used with simulated data (55). For example, TRANSYT was used to calculate PCEs for intersections (56).

2.5.10 Platoon-Based Method

The platoon-based PCE is calculated using equations 2.11 and 2.12. The PCE is the ratio of the number of followers caused by a given vehicle type (e.g., truck, bus, motorcycle) to the number of followers caused by a passenger car (57). In essence, the approach attempts to identify the number of passenger cars that would replace a given vehicle type in the traffic stream and result in the same amount of followers.

$$V_f = a + \sum_{i=0}^{n} b_i V_i \tag{2.11}$$

$$P\_PCE_i = \frac{b_i}{b_0} \tag{2.12}$$

P_PCE_i	Platoon-based passenger car equivalents for vehicle type *i*
V_i	Traffic volume of vehicle type i; for passenger car, $i = 0$.
V_f	Average number of followers in platoons

a, b_i	Regression coefficients for vehicles of type i. Note that for passenger car, i is equal to 0

The PCEs are based on an average number of followers, which is modeled by a linear regression equation. In essence, the modeler must collect data on a number of platoons that are led by vehicles of varying types. This data is used to estimate the parameters. The platoon-based method was initially proposed for PCE determination on two-lane highways, where passing lead vehicles may be difficult if there is a considerable amount of on-coming vehicles and/or many locations of restricted sight lines.

2.5.11 Speed-Based Method

The speed-based PCE approach is shown in equations 2.13 and 2.14. The PCE is the ratio of the amount of speed reduction in a traffic stream caused by a given vehicle type to the amount of speed reduction caused by a passenger car (57). In essence, this ratio represents the number of passenger cars that would replace a given vehicle type and result in the same amount of speed reduction.

$$S_{percentile} = S_F + \sum_{i=0}^{n} c_i V_i \tag{2.13}$$

$$S\_PCE_i = \frac{b_i}{b_0} \tag{2.14}$$

S_PCE_i	Speed-based passenger car equivalents for vehicle type i
V_i	Traffic volume of vehicle type i; for passenger car, $i = 0$.
$S_{percentile}$	Percentile speed (mph)
S_F	Free flow speed (mph)

c_i Regression coefficient; for passenger car, $i = 0$

The PCEs are based on estimated speeds as modeled by a linear regression equation. The estimated speed may be average, 50th percentile (median), 90th percentile, 95th percentile, etc. In essence, the modeler must collect data on vehicle speed and traffic composition (e.g. number of vehicles of each type). This data is used to estimate the parameters. This approach does not rely on defining a platoon. This method was initially proposed for PCE determination on two-lane highways.

2.5.12 Equal-Flow Method

The equal-flow method is shown in Equation 2.15. In essence the PCE represents the number of passenger cars that would replace a truck in a given mixed traffic stream. It is assumed that the mixed stream would produce the same traffic conditions (e.g., travel time, speed, density, etc.) as a passenger-car-only traffic stream that was developed based on the PCE (58). In this instance, the goal is to have the PCE-based passenger-car-only stream replicate the mixed traffic stream for all traffic flow conditions. Note that in other approaches the researchers are only concerned with traffic flow at capacity (e.g., LOS E) (59). The generalized form of the conversion from the mixed traffic flow to the passenger-car-only flow is provided in Equation 2.16.

$$MSF_i * N \ = \ E_{car} * P_{car} * F + \textstyle\sum_j E_j * P_j * F \qquad (2.15)$$

$$EF\_PCE_i \ = \ \frac{E_j}{E_{car}} \qquad (2.16)$$

EF_PCE_i Equal-flow passenger car equivalents for vehicle type i

MSF_i Maximum service flow rate at LOS i under ideal conditions (capacity for

LOS *i*) in passenger cars per hour per lane

N	Number of lanes
F	Observed traffic volume (veh/h)
E_{car}, E_j	Regression coefficients for passenger cars and vehicle type *j*
P_{car}, P_j	Percentage of passenger cars and vehicle type j

The product of MSF_i and N is the total passenger car throughput for the base case scenario (passenger-car-only flow) at the road segment clearance time (or at intersection discharging time).

2.5.13 Queue Discharge Flow Method

In the queue discharge flow (QDF) method, the QDF capacity is considered to be the equivalent criterion because it governs the operation of the freeway after the onset of congestion. This means that if trucks in the mixed stream are converted to passenger cars based on a QDF-based PCE, the converted QDF capacity is expected to have minimal variation (60 and 61). The objective is the minimum of variation for PCE-based converted QDF capacity. The design variable is the PCE, with constraints between the lowest and highest values of QDF capacities and PCEs. The goal is to find the optimal value that minimizes the variation for the converted QDF capacity. The QDF method is shown as a mathematical program in Equation 2.17.

$$\text{Objective Function: } Minimize\ Z(C^*) \tag{2.17}$$

$$\text{Design Variable: } QDF\_PCE$$

$$\text{Constrains: } X1 \le C^* \le X2, X3 \le PCE \le X4$$

QDF_PCE	queue-discharge-flow-based passenger car equivalents
C^*	Queue discharge flow capacity
$Z(C^*)$	coefficient of variation for converted QDF capacity
$X1, X2$	lower and upper limit of QDF capacity
$X3, X4$	lower and upper limit of passenger car equivalents

Based on the definition, this method is only appropriate for PCE determination under the congestion condition (e.g. work zone area or bottleneck on freeways and highways).

2.6 Research on Development and Calibration of Microscopic Traffic Simulation Models

In this research, the high fidelity CORSIM and VISSIM simulation models are used. Models with high fidelity, meaning they use high level of detail and accuracy to simulate the vehicles, can provide more information and better approximate real world situations (62).

CORSIM is a microscopic traffic simulation software package for signal, highway, and freeway systems developed under the direction of FHWA. The simulator models the movements of individual vehicles, which include the influences of geometric conditions, control conditions, and driver behavior. CORSIM consists of an integrated set of two microscopic simulation models that represent the entire traffic environment. NETSIM represents traffic on urban streets. FRESIM represents traffic on highways and freeways. The reasons for choosing CORSIM are

1) The HCM 2010 PCEs are calculated based on data simulated by FRESIM;

2) It is convenient to adjust the parameters values for different traffic conditions.

VISSIM is a microscopic, time-step, behavior-based multipurpose traffic simulation developed by the company PTV. The traffic simulator simulates the movement of vehicles and records the corresponding output. The traffic simulator is a microscopic traffic flow simulation model including the car following and lane change logic, incorporating Wiedemann's psycho-physical car following model for longitudinal movement and a rule-based algorithm for lateral movements. The Wiedemann's 99 car following model is adopted in this research, since the Wiedemann's 99 model has been shown to be suitable for interurban or freeway traffic flow (63). The VISSIM outputs include the order number of the data collection point, detector entering and leaving time, order number of vehicle, vehicle type, length, speed, travel time, and delay.

The parameters needing to be calibrated in CORSIM include vehicle performance parameters (e.g. mean queue discharge headway) and driver behavior parameters (e.g. mandatory lane change gap acceptance parameter). The driver behavior related parameters needing to be calibrated in VISSIM include car following parameters (e.g. headway time) and lane change parameters (e.g. speed reduction rate). The literature indicates that it is necessary to calibrate and validate the parameters before the simulation model is used for analysis (64). Many methods have been proposed for model calibration, including simulated annealing, neural networks, Latin hypercube experimental design algorithm, sequential simplex algorithm, non-parametric statistical technique and genetic algorithm (GA) (65,66, and 67). The GA is widely used since it does not require gradient information for the evaluation of an objective function, and is rather robust and can solve the problem of the combinatorial explosion of model parameters (68). For model calibration at basic freeway segments, the travel time and travel speed are recommended as traffic flow metrics in evaluating the accuracy of calibration results (64). There are several commonly used objective functions for evaluation and validation of the calibration, include the root mean square error, the mean absolute error ratio, the mean absolute percentage error, and the GEH statistics (69 and 70).

2.7 Research on Influencing Factor Analysis for Passenger Car Equivalents

A number of researchers have studied the impacts of different free-flow speed on PCEs. Huber (44) found PCEs decrease with the free-flow speed when the PCEs were estimated by equal-travel-time or equal-total-travel-time methods. It was found that PCEs estimated using the equal-density approach, which was used in the 2010 HCM, tend to increase with free-flow speed (1). A research on heavy vehicle effects on Florida freeways conducted by FDOT (71) also showed that the equal-density-based PCEs increase as free-flow speed increases. Research on the M25 and M24 freeways in the United Kingdom (72) found that headway-based PCEs decrease with the free-flow speed. All the above research studies assumed that all vehicle types had the same desired free-flow speed.

The impacts of speed limits on PCEs, when the speed limits were set separately for passenger cars and trucks have also been explored in previous research. It was found that, the equal-density-PCEs estimated when truck speed limits were 15 mph lower than the passenger car speed limits were, on average, 32% higher than when the speed limits were the same (73 and 74). Chitturi et al (53) found the delay-based-PCEs also increase when the free flow speed difference between the trucks and passenger cars increases.

A number of studies examined the impacts of trucks lane restrictions on PCEs. Al-Kaisy et al (75) examined the effects of truck lane-use restrictions on PCEs under four scenarios for a six-lane freeway segment (e.g. three lanes per direction). It was found that if all heavy vehicles were restricted from using the left-lane and larger heavy vehicles were restricted from using the middle-lane, the PCEs were lower compared to the no lane restriction scenario. Under this lane restriction scenario traffic operations improved overall and the lane restrictions mitigated the negative impact of trucks on traffic flows (75). A simulation-based study using data from a four-lane rural freeway segment in Louisiana also found that lane restrictions had a significant effect on the PCEs estimated by the equal-traffic-flow method (58 and 76).

There was no consensus in the literature on what data aggregation size should be used when estimating PCE values. For example, a 1-minute interval was used in both an equal-speed PCE study and the 2016 HCM equal-capacity PCE estimation procedure (39, 40, 77). A 5-minute interval was used for estimating PCE values based on queue-discharging-flow and capacity variability (60 and 78). The 15-minute interval has been widely adopted in previous research particularly in procedures where capacity values need to be identified (59). It is hypothesized that a 15 minute disaggregation level was chosen because in the HCM the capacity in the flow-speed curves represents the maximum hourly flow rate for a 15-min interval (2, 3). Interestingly, the 2016 HCM used a 1 minute data aggregation interval when identifying the capacity values required for estimating the PCEs.

2.8 Issue Statements

2.8.1 Issues in PCE at Level Freeway Segments in HCM 2010 and HCM 2016

One issue in the HCM 2010 and HCM 2016 is that the highest truck percentage value is 25 percent. While these PCE values may be appropriate for urban areas and large parts of the eastern U.S., they may not be appropriate for the western rural U.S., which consistently experiences truck percentages in an excess of 25 percent. For example:

1) Data from Nebraska Department of Transportation show that Interstate 80 in western Nebraska (west of Lincoln) experiences truck percentages in the range of 25% to 60% (7);
2) Data from Kansas Department of Transportation show that Interstate 70 in western Kansas (west of Salina) experiences truck percentages in the range of 25% to 50% (8);
3) Data from Missouri Department of Transportation show that Interstate 29 in northwestern Missouri (between Rock Port and St Joseph) experiences truck percentages in the range of 25% to 40% (9).

Given that it is estimated that truck freight tonnage will increase by 42 percent by 2040, this problem will continue into the future (79).

Another issue is that both the HCM 2010 and HCM 2016 PCE procedures for level freeway segment assume that trucks attempt to travel at the same free-flow speeds as passenger cars (2 and 3). In both situations the free flow speed is assumed to be the speed limit. However, many heavy trucks in the western rural U.S., and indeed the entire U.S., are governed through the use of speed limiters to travel in the range 60 to 70 mph (80). This is done to optimize transportation costs by limiting fuel usage. Recently, the U.S. government has recommended that speed limiters be applied to all heavy trucks to increase safety and reduce fuel usage (81). Compounding this issue is the speed limits on rural freeways in most of the western rural U.S. are higher than that in eastern U.S. (82). The differences in free-flow speeds between heavy trucks and passenger cars in western rural U.S. range from 8 to 15 mph.

The important point with respect to PCE is that the trucks passing other trucks at low speed differentials (e.g., a 67 mph truck passing a 66 mph truck) may cause moving bottlenecks. It is hypothesized that these events increase as truck percentage increases and speed differential decreases. It may take a substantial amount of time for the slower moving trucks to pass each other, and while this moving bottleneck is in existence, the passenger cars and other trucks that do not have speed limiters who, all else being equal, wish to travel faster, will be delayed by the localized congestion. Research on quantifying the effects of moving bottlenecks on multilane divided freeways is relatively sparse.

There are issues related to the truck types and truck fleet composition used in the simulation models that derived the PCE values in the HCM 2010 and HCM 2016. In HCM 2010, the truck fleet composition used to calculate the PCEs are not discussed. In Nebraska nearly 90% of trucks are heavy trucks, it is hypothesized that the PCEs calculated based on Nebraska data might be different. In HCM 2016, only two FHWA truck types (Class 5 and Class 9) were used in the simulation model. The characteristics of these two types might not be representative of the truck fleets in western states.

There is also an issue related to the number of lanes used in simulation in HCM 2016. The HCM 2016 uses three lanes per direction in simulation model, which might not be appropriate for freeways with two lanes per direction that widely exist in western states (e.g. Nebraska, Wyoming, etc.). It is hypothesized that the bottlenecks will not be as severe in freeways with three lanes per direction as compared to freeways with two lanes per direction, all else being equal.

2.8.2 Issues in Other PCE Methodologies

Although the PCEs have been estimated by a variety of techniques in previous research, there is very little research on PCE estimation under moving bottlenecks conditions on four-lane level freeway segments.

The platoon-based method is the only one that directly considers the effects of moving bottlenecks, referred to as platoons, in PCE estimation. In this method, a linear regression model is established between the number of followers and the volume of truck and passenger cars. The PCE is determined by the ratio of coefficients for trucks to passenger cars (57). However, this method is initially proposed to estimate PCEs on two-lane freeways. Other problems are

1) The critical headways for vehicles in platoons are empirically determined by observers;
2) There is potential high stand error for coefficients, since both the volume of trucks and passenger cars are used as independent variables, and there is high correlation between the volume of trucks and passenger cars (83).

The headway-based PCEs in previous research also have similar problems stated above (50), where

1) Only the headways between vehicles on the same lane are concerned;
2) The thresholds for vehicles interacting with each other are empirically determined;

3) The assumption that the average headway for car following car in passenger-car-only traffic flow is equal to that in mixed traffic flow;
4) The trucks are assumed to follow the same speed distribution as passenger cars.

The delay-based method is designed for PCE determination at signalized intersections or work zone (51 and 52), but this method has not been used on four-lane freeways with moving bottlenecks.

2.8.3 Issues in Moving Bottlenecks Identification

The current moving bottlenecks identification methodologies have a number of issues, including

1) Definition of moving bottlenecks on four-lane freeways. Until now the moving bottlenecks have been defined under two-lane highway conditions and assumed to occur only on one lane. In other words vehicle performance is assumed to not be affected by vehicles on adjacent lanes. However, when a vehicle (e.g. heavy truck) would like to change to adjacent lane to pass another vehicle (e.g. heavy truck), and both of these vehicles are traveling at relatively low speed, all the followers on two lanes who wish to travel at faster speeds will be blocked.
2) Methodology for critical headway determination. A key determinist on whether a given vehicle may be blocked is headway. Typically researchers use a critical headway to identify vehicles that may be blocked. Until now most of the values are empirically determined by observers, lacking a general methodology to quantify at what conditions the performance of vehicles could be considered interacted with others.

3) Leading and lagging critical headways values. Until now in many of the research the leading and lagging critical headways use the same values. However, due to different lengths among vehicle types, the leading and lagging headways for one vehicle pair may be different. For example, for the passenger cars following trucks, the leading critical headways may be longer than the lagging critical headways since the trucks are typically longer than the passenger cars.
4) Critical headways for vehicle pairs with different vehicle types. Until now, much research on vehicle pairs with different vehicle types used the same critical headways. However, previous research suggests that inter-vehicular spaces are affected by the types of vehicles that delimit the spacing (84), indicating that the critical headways need to be determined respectively for different vehicle pairs.

2.8.4 Issues in Metrics for Moving Bottlenecks Effects

The issues for moving bottlenecks metrics are:

1) How to use the existing metrics to quantify moving bottlenecks effects on four-lane freeway segments;
2) Metrics for impacts of moving bottlenecks existence on impeded vehicles. Until now, existing metrics are proposed for vehicles passing a specific location, not for measuring effects of the whole process of moving bottlenecks formation and dispersion (e.g. how long the moving bottlenecks actually exist, and the delay generated on the impeded vehicles).
3) Impacts of different moving bottlenecks types on impeded vehicles. Until now, the difference effects among various moving bottleneck types due to the different classifications and occupied lane of the leader vehicles are not

abundantly discussed. For example, the vehicles impeded by two trucks usually experience higher speed reduction than those impeded by two cars.

2.8.5 Other Issues

Currently there is no PCE estimation methodology for conditions with localized congestion caused by the moving bottlenecks on four-lane level freeway segments. This may be problematic because if the congestion is measured by density, the average density over several consecutive intervals will not be equal to the average density for these consecutive intervals, unless the vehicles are “uniformly” distributed in these consecutive intervals. Because in HCM 2010 and 2016 the level of service is measured by the density, the existence of localized congestion may affect the results of level of service analysis.

Lastly, many agencies plan to take new traffic management options for four-lane freeways, including the changes in speed limit and truck passing restriction (85 and 86). There is a need for a method for estimating the impacts of these measures.

CHAPTER 3 HYPOTHESIS STATEMENT

Based on field observation, the following hypotheses are proposed and need to be tested.

Hypothesis 1:

Vehicles impeded in moving bottlenecks are slower than free-flow vehicles.

Null hypothesis: $H_0: \bar{v}_1 = \bar{v}_0$

Alternative hypothesis: $H_a: \bar{v}_1 < \bar{v}_0$

$\bar{v}_m$	Average speed of vehicles with indicator m
m	Indicator for vehicle impeded in moving bottlenecks: 1 = Impeded vehicle; 0 = Free-flow vehicle (out of moving bottlenecks).

Hypothesis 2:

The truck-leading moving bottlenecks will cause more delay on impeded vehicles than passenger-car-leading moving bottlenecks.

Null hypothesis: $H_0: \bar{D}_{M_1} = \bar{D}_{M_0}$

Alternative hypothesis: $H_a: \bar{D}_{M_1} > \bar{D}_{M_0}$

$\bar{D}_{M_l}$	Average moving-bottlenecks-delay caused by moving bottlenecks with indicator l
l	Indicator for truck-leading moving bottlenecks: 1 = Truck-leading moving bottlenecks; 0 = Passenger-car-leading moving bottlenecks.

Hypothesis 3:

The PCEs calculated using 1) equal-density (HCM 2010) 2) equal-capacity (HCM 2016) methods may be different when Nebraska data are used. It is hypothesized that the higher truck percentage and speed difference will affect the PCE values.

Null hypothesis: $H_0: PCE_{i,1} = PCE_{i,0}$

Alternative hypothesis: $H_a: PCE_{i,1} \neq PCE_{i,0}$

$PCE_{i,j}$	PCE estimated by method i at j condition
i	Indicator for PCE estimation method: 1 = equal-density; 2 = equal-capacity
j	Indicator for PCE estimation condition: 1 = Western rural U.S. condition; 0 = Original research

Hypothesis 4:

The PCEs estimated under western rural U.S. conditions will better capture the level of service under moving bottlenecks conditions. In other words, compared with the empirical speed-flow plots based on HCM 2010 and 2016 PCE, the empirical speed-flow plots based on the PCE estimated under western rural U.S. conditions may better match the “speed-flow curve” provided in HCM 2010 and 2016, which means the empirical speed-flow plots based on the PCE estimated under western rural U.S. conditions may be “closer” to the “speed-flow curve” provided in HCM 2010 and 2016.Mathmatically, when the PCE estimated under western rural U.S. conditions is used, the sum of squared error (SSE) between the empirical speed and the estimated speed by the “speed-flow curve” provided in HCM 2010/2016, is different from the SSE when the HCM 2010/2016 PCEs are used.

Null hypothesis: $H_0: \sum_{t=1}^{n_t} \left(\bar{v}_t - \hat{v}_{t,PCE_{i,1}}\right)^2 = \sum_{t=1}^{n_t} \left(\bar{v}_t - \hat{v}_{t,PCE_{i,0}}\right)^2$

Alternative hypothesis: $H_a: \sum_{t=1}^{n_t} \left(\bar{v}_t - \hat{v}_{t,PCE_{i,1}}\right)^2 \neq \sum_{t=1}^{n_t} \left(\bar{v}_t - \hat{v}_{t,PCE_{i,0}}\right)^2$

$\bar{v}_t$	Average empirical speed of vehicles in time interval t
$\hat{v}_{t,PCE_{i,j}}$	Estimation of average speed for time interval t from "speed-flow curve" provided in HCM 2010/2016, at the equivalent flow rate calculated based on PCE with method i under j condition
n_t	The number of time interval t in empirical data
$PCE_{i,j}$	PCE estimated by method i at j condition
i	Indicator for PCE estimation method: 1 = equal-density; 2 = equal-capacity
j	Indicator for PCE estimation condition: 1 = Western rural U.S. condition; 0 = Original research

CHAPTER 4 DATA COLLECTION AND PRELIMINARY ANALYSIS

4.1 Data Collection Sites

Data was collected on Interstate 80 at 13 locations between mileposts 177 and 399, as shown in Figure 4.1. This 222-mile section is located between Lincoln and North Platte. Interstate 80 is a divided four-lane freeway with a speed limit of 75 mph.

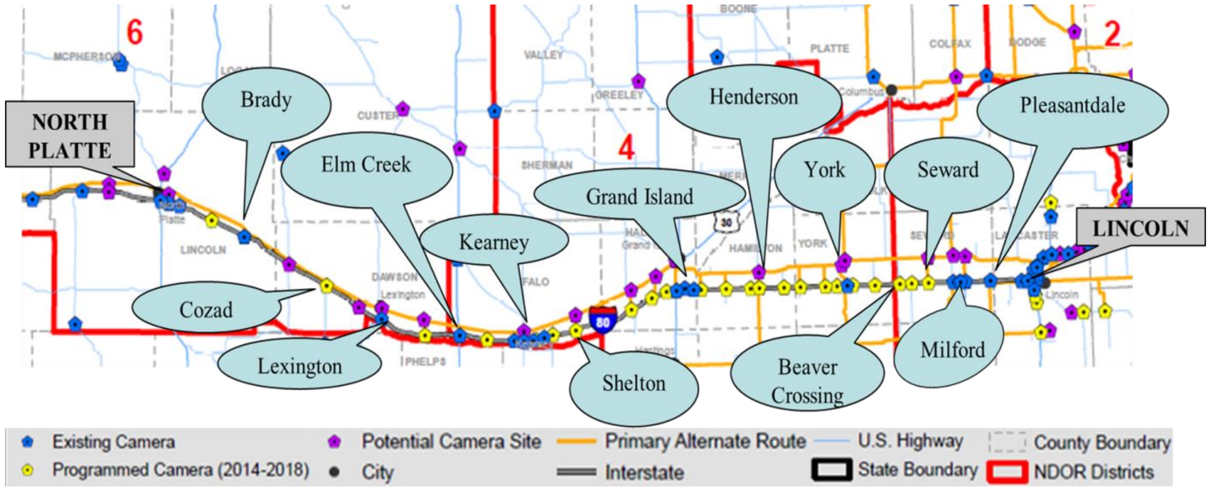


Figure 4.1 I-80 Data collection sites between Lincoln and North Platte

The goal was to find data collection sites that were similar to each other but experienced different volumes and truck percentages. Table 4.1 provides a description of attributes of the 13 sites.

Table 4.1 Data collection sites along I-80 on western part of Nebraska, between Lincoln and North Platte

Data Collection Site	Mile Marker	Camera Height (ft)*	Overpass Road	Direction*	Nearest Exit
Pleasantdale	388	47.0	County Hwy 154 / State Hwy 103	WB	-
Milford	382	47.0	County Hwy 238 / State Hwy 80H	WB	-
Seward	378	46.5	County Hwy 294	EB	Exit 379
Beaver Crossing	369	46.8	County Hwy 420 / State Hwy 80E	WB	-
York	354	47.5	Rd M	WB	Exit 353
Henderson	342	48.0	State Hwy 93A	WB	-
Grand Island	316	47.1	County Hwy 4	WB	Exit 318
Shelton	290	47.7	Willow Rd	EB	Exit 291
Kearney	280	47.0	M Ave	WB	Exit 279
Elm Creek	255	48.2	450 Rd	WB	Exit 257
Lexington	234	47.0	Rd 431	WB	Exit 237
Cozad	220	46.0	Rd 419	WB	Exit 222
Brady	199	46.5	56D / S Banner Rd.	WB	-

*Camera is located “Camera Height” feet above the roadway; “WB” means westbound, and “EB” means eastbound.

All of the test sites were straight (e.g., no horizontal curvature), having two 12-foot lanes in each direction, a 6-foot lateral clearance on the right lane, and level terrain (e.g., grades less than 1%). As discussed later, it was critical that the traffic information was collected from an overpass. Consequently, the data collection site was located on an overpass and information was collected from the traffic below. Five of the sites had entrance and exit ramps for Interstate 80, and eight did not. Data was collected for a single direction. A detailed description of each site is given in the appendix.

4.2 Data Collection Methodology

4.2.1 Equipment

Data were collected using the Nebraska Transportation Center's (NTC) mobile data collection equipment and the NTC ITS van. The van is equipped with two cameras mounted on a 42-foot telescope mast, as well as Autoscope, a video detection system. In the field, the van was parked on the overpass above I-80 with cameras directed straight down in order to obtain the best view of the two lanes. Figure 4.2 shows a picture of the van during data collection at the overpass in Milford.

Figure 4.2 NTC's ITS data collection van

The Autoscope system was used to collect speed data. Virtual speed detectors were directly set on the video, and these were located in the middle of each lane, as shown in Figure 4.3.

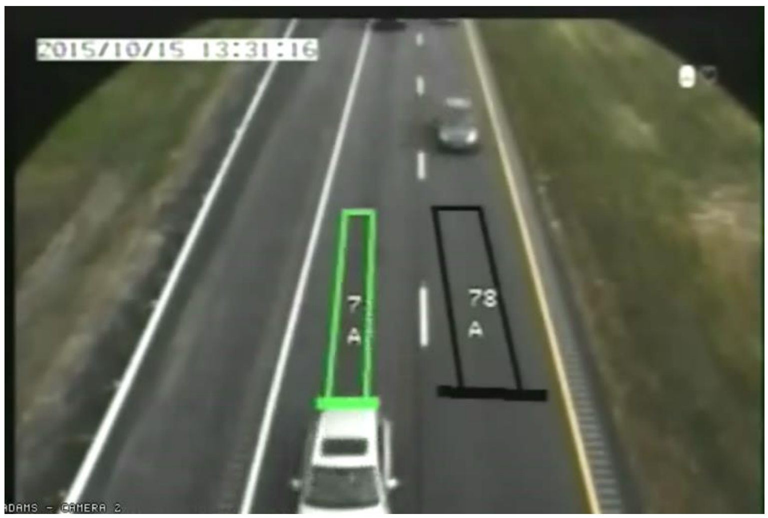


Figure 4.3 Layout of virtual speed detectors on video

When the front bumpers of the vehicles reached the front edge of virtual detectors, the detectors were activated until the rear bumpers were no longer in the detection zone. The detectors were used to measure instantaneous speed and occupancy. Autoscope uses the occupancy and speed information to estimate vehicle length and, based on vehicle length, vehicle type. The raw data included the vehicle count, the time at which vehicles entered and left detectors (milliseconds), and vehicle speed (mph). Vehicles were classified into five categories that corresponded to the FHWA 13-Category Rule Set (4): passenger cars (including normal passenger cars, pick-ups, panel vehicles, and vans as well as vehicles with one or two axle trailers), buses, single-unit trucks, heavy trucks, and recreational vehicles. Data collection results were automatically output into an ASCII.txt file by Autoscope.

4.2.2 Sensor Calibration

The sensors were calibrated prior to each day's data collection in order to ensure the most accurate results. In particular, the following Autoscope parameters were calibrated:

1) The critical length for vehicle type identification;
2) The minimum and maximum detected vehicle speeds;
3) The length, width, and position of virtual speed detectors.

Critical length is used for identifying differences in length among the five vehicle classifications. Vehicles with a length no longer than 25 feet were identified as passenger cars; vehicles with a length between 25 feet and 45 feet were identified as buses, single unit trucks, or recreational vehicles; and those with a length longer than 45 feet were identified as heavy trucks. Due to similarities in length among buses, single-unit trucks, and recreational vehicles, these three classifications were manually identified from the video recording through a two-step process. First, the time stamp for each vehicle over 45 feet was identified. Then, a viewer examined the tape and classified the vehicle accordingly.

As part of the calibration, minimum and maximum detected speeds need to be identified. Prior to data collection, the preliminary data was collected. It was found that the minimum and maximum speeds were between 45 mph and 50 mph, and between 95 mph and 100 mph, respectively. In order to effectively identify outliers in output data, the minimum and maximum detected speeds were set as 0 mph and 120 mph, respectively, so the complete range of speeds could be observed.

The length, width, and position of virtual speed detectors were calibrated according to Autoscope calibration protocol (93). First, an image of the data collection site was obtained, which is known in literature as “snapped on.” Critical data, including lane width, length of observing freeway segments, and height of the cameras, was measured in the field. A set of three horizontal and three vertical grid lines were placed on the image. The length and width were calibrated using the gird lines as references. The distance between two adjacent vertical lines was set to represent lane width. These values were set to 0 feet, 12 feet, and 24 feet from the left vertical grid line to the right grid line. These values were set from left to right so that the left grid line would represent the base line: the 0 foot position. The distance between two adjacent horizontal lines represents the length of the measured freeway segment. Prior to data collection, three markings were placed on the highways, located 30 feet apart to use as guides. The horizontal distances were set to 0 feet, 30 feet, and 60 feet. These markings were made from top to bottom where the top is the base condition, or 0 foot position. The height of the cameras represents the distance between the cameras and the highway and includes the height of the overpass, the height of the van, and the height of the mast. These heights were measured at each site, and the average value was 47 feet.

4.3 Data Collection

The empirical data was collected at 15 separate locations over 15 separate days from June 1 through December 22, 2015. The data was collected during:

1) Daytime hours ranging from 8:00 am to 7:00 pm in the summer and 9:00 am to 5:00 pm in the fall;

2) Clear weather conditions with dry pavements (e.g., no rain or snow) and cloudy conditions, which minimized the effect of vehicle shadows on the accuracy of the detector;
3) Wind speeds below 10 mph. Strong continuous winds or wind gusts were capable of swaying the van's mast, producing erroneous data, and decreasing safety.

In total, 60 hours of valid traffic flow data were collected, with an average of 4.6 hours for each site. Details for the data collection condition and results are shown in Table 4.2. The amount of vehicles observed hourly decreases from east to west on Interstate 80.

Table 4.2 Summary of data collection

Data Collection Sites	Date (in 2015)	Hours	Total Duration (hours)	Number of Vehicles Observed
Pleasantdale	Monday, June 01	4:00 PM to 7:00 PM	3	2695
Milford	Wednesday, June 10	5:00 PM to 6:00 PM	1	886
	Friday, June 12	11:00 AM to 3:00 PM	4	4576
	Tuesday, September 29	1:00 PM to 4:00 PM	3	3175
Seward	Friday, June 12	4:00 PM to 7:00 PM	3	3886
	Tuesday, June 16	10:00 AM to 12:00 PM	2	1690
Beaver Crossing	Friday, November 13	12:00 PM to 3:00 PM	3	3104
York	Tuesday, June 16	3:00 PM to 7:00 PM	4	3284
	Friday, June 19	12:00 PM to 3:00 PM	3	3655
Henderson	Thursday, October 15	1:00 PM to 4:00 PM	3	2357
Grand Island	Wednesday, November 25	11:00 AM to 3:00 PM	4	4610
Shelton	Wednesday, June 03	1:30 PM to 6:30 PM	5	3958
Kearney	Tuesday, December 22	9:00 AM to 1:00 PM	4	2061
Elm Creek	Monday, December 21	12:00 PM to 4:00 PM	4	2055
Lexington	Friday, November 20	9:00 AM to 1:00 PM	4	2055
Cozad	Thursday, November 15	1:30 PM to 5:30 PM	4	1920
Brady	Tuesday, June 02	1:00 PM to 7:00 PM	6	3059
Total			60	48903

4.4 Preliminary Analysis

4.4.1 Vehicle Classification

In this research, the vehicle classification is based on the FHWA 13-Category Rule Set. This standardized vehicle classification system was developed by FHWA in the mid-1980s, and it is currently used for most federal reporting requirements and serves as the basis for most state vehicle classification counting efforts (4). The classification is shown in Figure 2.1. According to this standard, vehicles are classified into the following 6 groups: motorcycles (class 1); passenger cars, pickups, vans, and vehicles with trailers (classes 2 and 3); buses (class 4); single-unit trucks (classes 5, 6, and 7); recreational vehicles (class 5); and heavy trucks (classes 8 to 13).

4.4.2 Vehicle Composition Analysis

The numbers of observed vehicles for each vehicle classification and data collection site are shown in Table 4-3. A total of 48,903 vehicles were observed across the 13 data collection sites. The data included 34,330 passenger cars, 14,231 trucks (1,287 single-unit trucks and 12,944 heavy trucks), 261 buses, and 81 recreational vehicles. The observed vehicles are mainly comprised of passenger cars (70.2%) and trucks (29.1%). Buses and recreational vehicles were 0.7% of the traffic flow and therefore only the effect of trucks on traffic flow was analyzed. Note that 91% of truck traffic was identified as heavy trucks.

Table 4-3 Total number of vehicles for each classification

Data Collection Sites	PC (Passenger Car)	SUT (Single-unit Truck)	HT (Heavy Truck)	Truck (ST+HT)	Bus	RV (Recreational Vehicle)	Total
Pleasantdale	2080	58	535	593	16	6	2695
Milford	6154	266	2168	2434	34	15	8637
Seward	4286	132	1126	1258	29	3	5576
Beaver Crossing	2275	54	759	813	14	2	3104
York	4934	162	1786	1948	41	16	6939
Henderson	1590	49	697	746	16	5	2357
Grand Island	3703	73	821	894	11	2	4610
Shelton	2583	141	1191	1332	21	22	3958
Kearney	1438	49	566	615	7	1	2061
Elm Creek	1486	35	527	562	6	1	2055
Lexington	1168	53	695	748	13	3	1932
Cozad	1013	43	844	887	19	1	1920
Brady	1620	172	1229	1401	34	4	3059
Total	34330	1287	12944	14231	261	81	48903

The hourly volume of each vehicle classification at all data collection sites are shown in Table 4.4 and Figure 4.4. The results of the data are detailed below.

1) The hourly traffic volume decreases from east to west. East of Shelton, the traffic volume is greater than 1000 veh/h, and west of Shelton, it is lower than 1000 veh/h.

2) Passenger car hourly traffic volume decreases from east to west. This decrease is at a much greater rate than that of trucks. East of Shelton, passenger car volume is greater than 500 veh/h, and west of Shelton it is less than 500 veh/h. Truck volume varies between 100 to 300 veh/h across all sites.

3) Hourly volume for both buses and recreational vehicles are very low at all data

collection sites, compared with passenger cars and trucks.

The percentages of vehicles for each classification at all of the sites are shown in Table 4.5 and Figure 4.5.

The data results are provided below.

1) From east to west the truck percentage gradually increases while the passenger car percentage gradually decreases. East of Grand Island, the truck percentage ranges from 19.4% to 31.7%; west of Grand Island, it ranges from 27.4% to 46.2%.

2) Truck percentages are higher than 25% (the highest truck percentage published in the HCM 2010 and HCM 2016) at 10 of 13 sites. Note that the average truck percentage for all of the data collection sites is 30%.

3) The percentage of both buses and recreational vehicles are very low at all data collection sites, compared with passenger cars and trucks.

Table 4.4 and Table 4.5 and Figure 4.4 and Figure 4.5 show the results for hourly volume and truck percentage, the details of which are provided below.

1) On I-80 between Lincoln and North Platte, Nebraska, a lower truck percentage (less than 30%) usually appears with a higher traffic volume (higher than 1000 veh/h) east of Grand Island. In contrast, a higher truck percentage (higher than 30%) usually appears with a lower traffic volume (lower than 1000 veh/h) west of Grand Island.

2) Approximately 90% of the percentage of trucks is heavy trucks. It would be expected that heavy trucks have a much greater effect on traffic than single-unit trucks.

Table 4.4 Hourly volumes for each vehicle classification on all data collection sites (veh/h)

Data Collection Sites	PCs	STs	HTs	Trucks	Buses	RVs	Total Volume
Pleasantdale	693	19	178	197	5	2	1094
Milford	769	33	271	304	4	2	1383
Seward	857	26	225	251	6	1	1366
Beaver Crossing	758	18	253	271	5	1	1306
York	705	23	255	278	6	2	1269
Henderson	530	16	232	248	5	2	1033
Grand Island	926	18	205	223	3	1	1376
Shelton	517	28	238	266	4	4	1057
Kearney	360	12	142	154	2	0	670
Elm Creek	372	9	132	141	2	0	656
Lexington	292	13	174	187	3	1	670
Cozad	253	11	211	222	5	0	702
Brady	270	29	205	234	6	1	745
Average	562	20	209	229	4	1	1025

* Note: PC = passenger car; ST = single-unit truck; HT = heavy truck; RV = recreational vehicle

Table 4.5 Percentages of vehicles for each classification

Data Collection Sites	**% PCs**	**% STs**	**% HTs**	**% Trucks**	**% Buses**	**% RVs**
Pleasantdale	77.2%	2.2%	19.9%	22.0%	0.6%	0.2%
Milford	71.3%	3.1%	25.1%	28.2%	0.4%	0.2%
Seward	76.9%	2.4%	20.2%	22.6%	0.5%	0.1%
Beaver Crossing	73.3%	1.7%	24.5%	26.2%	0.5%	0.1%
York	71.1%	2.3%	25.7%	28.1%	0.6%	0.2%
Henderson	67.5%	2.1%	29.6%	31.7%	0.7%	0.2%
Grand Island	80.3%	1.6%	17.8%	19.4%	0.2%	0.0%
Shelton	65.3%	3.6%	30.1%	33.7%	0.5%	0.6%
Kearney	69.8%	2.4%	27.5%	29.8%	0.3%	0.1%
Elm Creek	72.3%	1.7%	25.6%	27.4%	0.3%	0.1%
Lexington	60.5%	2.7%	36.0%	38.7%	0.7%	0.2%
Cozad	52.8%	2.2%	44.0%	46.2%	1.0%	0.1%
Brady	53.0%	5.6%	40.2%	45.8%	1.1%	0.1%
Average	68.5%	2.6%	28.2%	30.7%	0.6%	0.2%

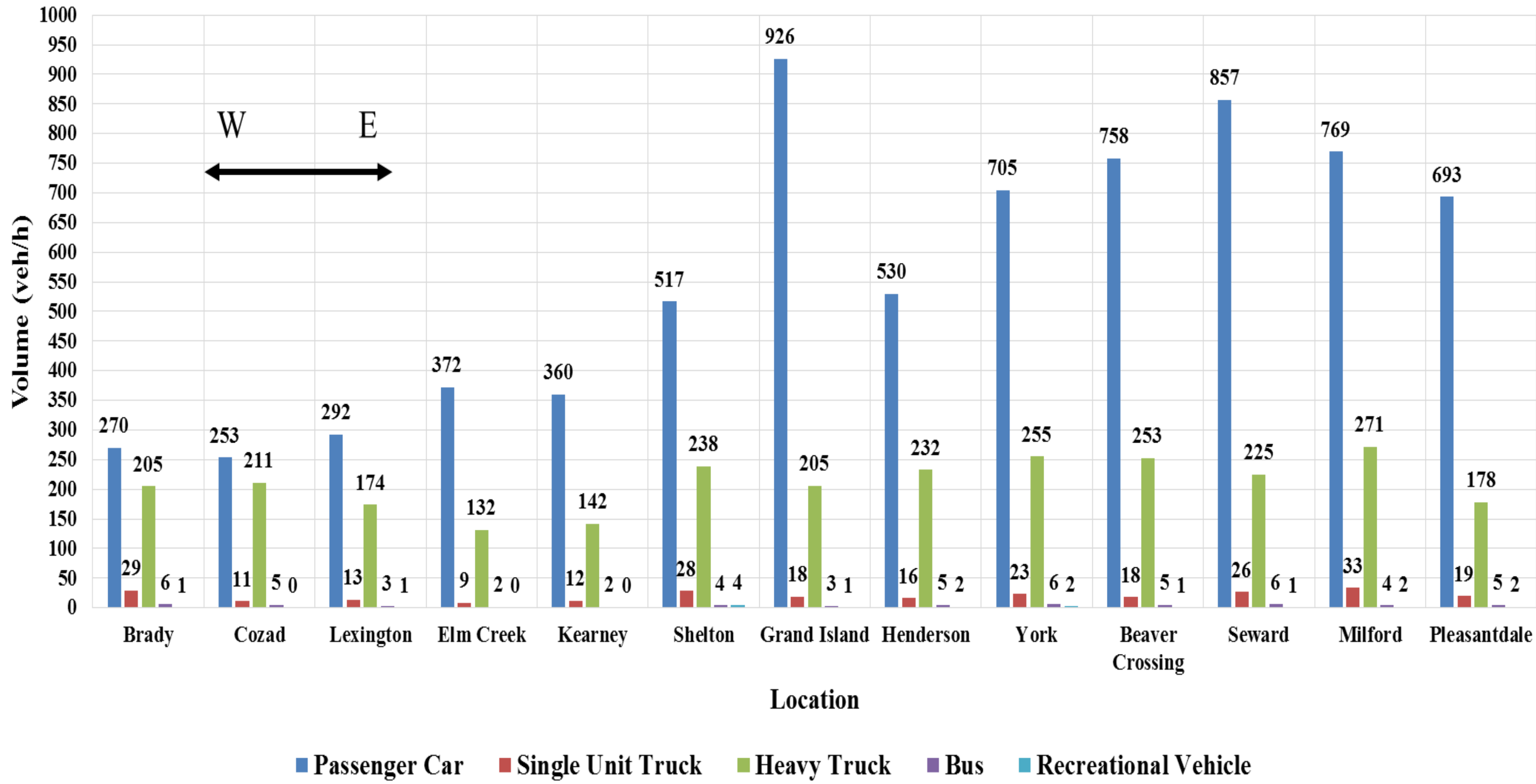


Figure 4.4 Hourly volumes for each classification

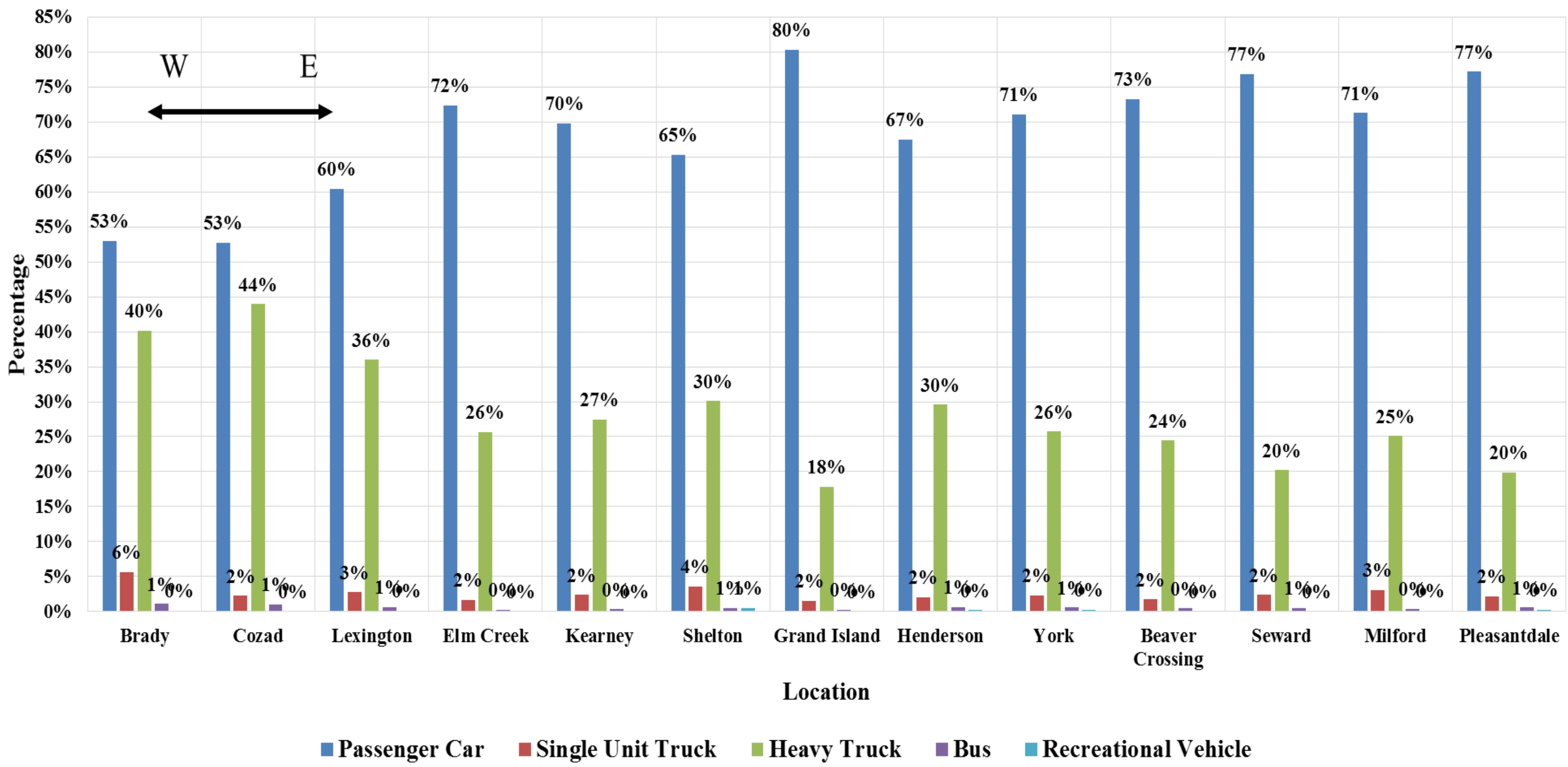


Figure 4.5 Percentages of vehicles for each classification

4.4.3 Speed Distribution

The 15th percentile, average, and 85th percentile of speed distributions for passenger cars, single-unit trucks, and heavy trucks are shown in Figure 4.6.

Based on the collected data, the average speed of a passenger car and a truck is 71.6 mph and 64.5 mph, respectively. The average speed of a passenger car is 7.1 mph higher than that of a truck, and this difference is statistically significant at the 95% level of confidence ($t = 122.28$, $P < 0.05$). The average speed of a single-unit and a heavy truck is 65.9 mph and 64.3 mph, respectively. The average speed of a single-unit truck is 1.6 mph higher than that of a heavy truck, and this difference is statistically significant at the 95% level of confidence ($t = 9.66$, $P < 0.05$). While the difference was statistically significant, it was determined that the difference was low enough that the group could be combined.

The 85th percentile speed for a passenger car, single-unit, and heavy truck is 77 mph, 72 mph, and 70 mph, respectively. The 85th percentile speed for a passenger car is 2 mph higher than the maximum speed limit on I-80 in Nebraska (e.g., 75 mph). The 15th percentile speed for a passenger car, single-unit, and heavy truck is 66 mph, 59 mph, and 58 mph, respectively. The 15th percentile speeds for all vehicle classifications are higher than the minimum speed limit on I-80 in Nebraska (e.g., 40 mph).

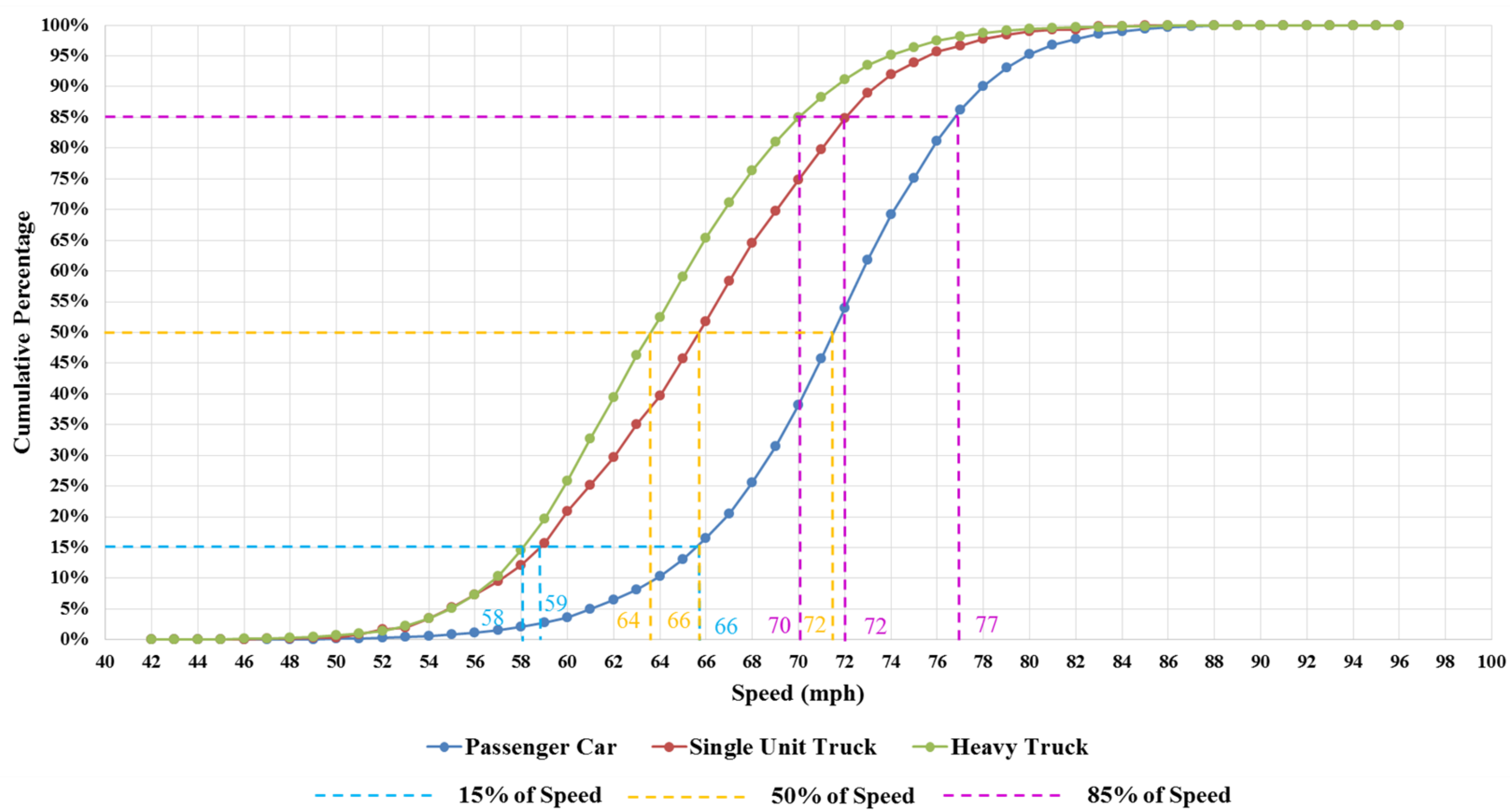


Figure 4.6 Speed distribution for all vehicles

4.5 Summary

In this chapter, the data for conducting preliminary analysis was collected, and the vehicle composition and speed distribution statistics were provided. The vehicle classification is based on the FHWA 13-Category Rule Set and divided into five classifications: passenger cars, single-unit trucks, heavy trucks, buses, and recreational vehicles. The vehicle stream consists mainly of passenger cars (70.2%) and trucks (29.1%). The results of vehicle composition analysis show that hourly traffic volume gradually decreases while truck percentage gradually increases from east to west. A lower truck percentage (less than 30%) occurs periodically east of Grand Island where traffic volumes are higher (e.g., greater than 1000 veh/h). A higher truck percentage (higher than 30%) occurs periodically west of Grand Island where traffic volumes are lower (lower than 1000 veh/h). Due to 91% of the truck traffic identified as heavy trucks and the low speed difference between single-units and heavy trucks, these two vehicle classifications can be combined for analysis.

CHAPTER 5 HIGHWAY CAPACITY ANALYSIS BY HCM 2010 AND 2016 APPROACH

5.1 Recommended PCE Values in HCM 2010 and 2016

5.1.1 Recommended PCE Values in HCM 2010

In the 2010 HCM, PCEs for basic freeway segments are in two forms. The first is for average conditions across three types of terrain: level, rolling, and mountain. The second is for specific segments of a given length and grade, where the PCE is provided as a function of the percentage of trucks. The recommended PCE values for:

1) Freeways according to terrain type;
2) On specific upgrades are shown in Table 5.1 and in Table 5.2.

Table 5.1 PCE value in HCM 2010 for freeway by type of terrain

Terrain	**Level**	**Rolling**	**Mountainous**
Passenger car equivalents for trucks and buses, E_T	1.5	2.5	4.5

Table 5.2 PCE value in HCM 2010 for freeway at level terrain by proportion of trucks

Upgrade (%)	Length (mi)	2%	4%	5%	6%	8%	10%	15%	20%	≥25%
≤ 2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
>2–3	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.50–0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.75–1.00	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>1.00–1.50	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>1.50	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
>3–4	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50–0.75	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	>0.75–1.00	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	>1.00–1.50	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	>1.50	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
>4–5	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.50–0.75	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	>0.75–1.00	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	>1.00	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
>5–6	0.00–0.25	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.30	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.30–0.50	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	>0.50–0.75	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0

	>0.75–1.00	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	>1.00	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
>6	0.00–0.25	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	>0.25–0.30	4.5	4.0	3.5	3.5	3.5	3.0	2.5	2.5	2.5
	>0.30–0.50	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	>0.50–0.75	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	>0.75–1.00	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	>1.00	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

It can be seen that for level terrain types, the HCM 2010 PCE value for trucks and buses is 1.5. The HCM 2010 recommends a value of 1.5 for specific upgrade segments of:

1) Less than two percent;
2) Any length;
3) Any truck percentage.

In HCM 2010, the level terrain is defined as: “any combination of grades and horizontal or vertical alignment that permits heavy vehicles to maintain the same speed as passenger cars, and this type of terrain typically contains short grades of no more than 2%” (HCM 2010, p. 11-44). On the I-80 between Lincoln and North Platte, Nebraska, the grades for all data collection sites were no greater than 2% and therefore can be defined as level terrain in HCM 2010. The level terrain definition indicates PCEs at level terrain are determined based on the assumption that passenger cars and trucks have the same average speed.

For truck percentages higher than 25%, the HCM 2010 recommends the PCE value as 25% trucks percentage, since in previous HCM the PCE is only provided up to 25% truck percentage.

5.1.2 Recommended PCE Values in HCM 2010

In the 2016 HCM, PCEs for basic freeway segments are also recommended in two forms. The first is for average conditions across two types of general terrain: level and rolling. The second is for specific segments of a given length and grade, where the PCE is provided as a function of

1) The percentage of trucks;
2) The percentage of single-unit trucks (SUT) and heavy trucks (trailer trucks, TT).

The recommended PCE values for freeways according to terrain type and specific grades are shown in Table 5.3 and Table 5.4 to Table 5.6.

Table 5.3 PCE value in HCM 2016 for freeway by type of terrain

Terrain Type	**Level**	**Rolling**
Passenger car equivalents for trucks and buses, E_T	2.0	3.0

Table 5.4 PCE value in HCM 2016 for freeway at level terrain by proportion of trucks with 70% heavy trucks, 30% single-unit trucks

Upgrade (%)	Length (mi)	2%	4%	5%	6%	8%	10%	15%	20%	≥25%
-2	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.625	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.875	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	1.25	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	1.5	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
0	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.625	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.875	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	1.25	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	1.5	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
2	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	3.76	2.96	2.78	2.65	2.48	2.38	2.22	2.14	2.09
	0.625	4.47	3.33	3.08	2.91	2.68	2.54	2.34	2.23	2.17
	0.875	4.80	3.50	3.22	3.03	2.77	2.61	2.39	2.28	2.21
	1.25	5.00	3.60	3.30	3.09	2.83	2.66	2.42	2.30	2.23
	1.5	5.04	3.62	3.32	3.11	2.84	2.67	2.43	2.31	2.23
2.5	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	4.11	3.14	2.93	2.78	2.58	2.46	2.28	2.19	2.13
	0.625	5.04	3.62	3.32	3.11	2.84	2.67	2.43	2.31	2.23
	0.875	5.48	3.85	3.51	3.27	2.96	2.77	2.50	2.36	2.28
	1.25	5.73	3.98	3.61	3.36	3.03	2.83	2.54	2.40	2.31
	1.5	5.8	4.02	3.64	3.38	3.05	2.84	2.55	2.41	2.32
3.5	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	4.88	3.54	3.25	3.05	2.80	2.63	2.41	2.29	2.22
	0.625	6.34	4.30	3.87	3.58	3.20	2.97	2.64	2.48	2.38
	0.875	7.03	4.66	4.16	3.83	3.39	3.12	2.76	2.57	2.46
	1.25	7.44	4.87	4.33	3.97	3.50	3.22	2.82	2.62	2.50
	1.5	7.53	4.92	4.38	4.01	3.53	3.24	2.84	2.63	2.51
4.5	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97

	0.375	5.80	4.02	3.64	3.38	3.05	2.84	2.55	2.41	2.32
	0.625	7.90	5.11	4.53	4.14	3.63	3.32	2.90	2.68	2.55
	0.875	8.91	5.64	4.96	4.50	3.92	3.56	3.07	2.82	2.67
	1	9.19	5.78	5.08	4.60	3.99	3.62	3.11	2.85	2.70
5.5	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	6.87	4.58	4.10	3.77	3.35	3.09	2.73	2.55	2.44
	0.625	9.78	6.09	5.33	4.82	4.16	3.76	3.21	2.93	2.77
	0.875	11.20	6.83	5.94	5.33	4.56	4.09	3.45	3.12	2.93
	1	11.60	7.04	6.11	5.47	4.67	4.18	3.51	3.17	2.97
6	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	7.48	4.90	4.36	3.99	3.52	3.23	2.83	2.63	2.51
	0.625	10.87	6.66	5.79	5.21	4.46	4.01	3.39	3.08	2.89
	0.875	12.54	7.54	6.51	5.81	4.94	4.40	3.67	3.30	3.08
	1	13.02	7.78	6.71	5.99	5.07	4.51	3.75	3.37	3.14

Table 5.5 PCE value in HCM 2016 for freeway at level terrain by proportion of trucks with 50% heavy trucks, 50% single-unit trucks

Upgrade (%)	Length (mi)	2%	4%	5%	6%	8%	10%	15%	20%	≥25%
-2	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.625	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.875	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	1.25	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	1.5	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
0	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.625	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.875	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	1.25	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	1.5	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
2	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	3.76	2.95	2.77	2.64	2.47	2.36	2.20	2.11	2.06
	0.625	4.32	3.24	3.01	2.84	2.63	2.49	2.29	2.19	2.12
	0.875	4.57	3.37	3.11	2.93	2.70	2.55	2.33	2.22	2.15
	1.25	4.71	3.45	3.17	2.99	2.74	2.58	2.36	2.24	2.17
	1.5	4.74	3.47	3.19	3.00	2.75	2.59	2.36	2.24	2.17
2.5	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	4.10	3.13	2.92	2.77	2.57	2.44	2.26	2.16	2.10
	0.625	4.84	3.52	3.23	3.03	2.77	2.61	2.38	2.26	2.18
	0.875	5.17	3.69	3.37	3.15	2.87	2.69	2.43	2.30	2.22
	1.25	5.36	3.79	3.45	3.22	2.92	2.73	2.47	2.33	2.24
	1.5	5.40	3.81	3.47	3.24	2.93	2.74	2.47	2.33	2.25
3.5	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	4.89	3.54	3.25	3.05	2.79	2.62	2.39	2.26	2.19
	0.625	6.05	4.15	3.75	3.47	3.11	2.89	2.58	2.42	2.32
	0.875	6.58	4.43	3.97	3.66	3.26	3.01	2.67	2.49	2.39
	1.25	6.88	4.58	4.10	3.77	3.35	3.09	2.72	2.53	2.42
	1.5	6.95	4.62	4.13	3.80	3.37	3.10	2.73	2.54	2.43
4.5	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93

	0.375	5.83	4.03	3.65	3.39	3.05	2.84	2.55	2.39	2.30
	0.625	7.53	4.92	4.38	4.01	3.53	3.24	2.83	2.62	2.50
	0.875	8.32	5.34	4.72	4.29	3.75	3.42	2.97	2.73	2.59
	1	8.53	5.45	4.81	4.37	3.81	3.47	3.00	2.76	2.62
5.5	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	6.97	4.63	4.14	3.81	3.38	3.11	2.74	2.55	2.43
	0.625	9.37	5.89	5.16	4.68	4.05	3.67	3.14	2.88	2.72
	0.875	10.49	6.48	5.65	5.09	4.37	3.93	3.34	3.03	2.85
	1	10.80	6.64	5.78	5.20	4.46	4.01	3.39	3.08	2.89
6	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	7.64	4.98	4.43	4.05	3.56	3.26	2.85	2.64	2.51
	0.625	10.45	6.45	5.63	5.07	4.36	3.92	3.33	3.03	2.85
	0.875	11.78	7.16	6.20	5.56	4.74	4.24	3.56	3.22	3.01
	1	12.15	7.35	6.36	5.69	4.85	4.33	3.62	3.27	3.05

Table 5.6 PCE value in HCM 2016 for freeway at level terrain by proportion of trucks with 30% heavy trucks, 70% single-unit trucks

Upgrade (%)	Length (mi)	2%	4%	5%	6%	8%	10%	15%	20%	≥25%
-2	0.125	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.375	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.625	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.875	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	1.25	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	1.5	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
0	0.125	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.375	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.625	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.875	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	1.25	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	1.5	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
2	0.125	2.67	2.32	2.23	2.17	2.08	2.03	1.94	1.89	1.86
	0.375	3.63	2.82	2.64	2.52	2.35	2.25	2.10	2.02	1.97
	0.625	4.12	3.08	2.85	2.69	2.49	2.36	2.18	2.08	2.02
	0.875	4.37	3.21	2.96	2.78	2.56	2.42	2.22	2.11	2.05
	1.25	4.53	3.29	3.02	2.84	2.60	2.45	2.24	2.13	2.07
	1.5	4.58	3.31	3.04	2.86	2.61	2.46	2.25	2.14	2.07
2.5	0.125	2.75	2.36	2.27	2.20	2.11	2.04	1.95	1.90	1.87
	0.375	4.01	3.02	2.80	2.65	2.46	2.33	2.16	2.06	2.01
	0.625	4.66	3.35	3.08	2.88	2.64	2.48	2.26	2.15	2.08
	0.875	4.99	3.52	3.21	3.00	2.73	2.56	2.32	2.19	2.12
	1.25	5.20	3.64	3.30	3.08	2.79	2.60	2.35	2.22	2.14
	1.5	5.26	3.67	3.33	3.10	2.80	2.62	2.36	2.23	2.15
3.5	0.125	2.93	2.45	2.34	2.26	2.16	2.09	1.98	1.92	1.89
	0.375	4.86	3.46	3.16	2.96	2.69	2.53	2.30	2.18	2.10
	0.625	5.88	3.99	3.59	3.32	2.98	2.76	2.46	2.31	2.22
	0.875	6.40	4.26	3.81	3.51	3.12	2.88	2.55	2.38	2.28
	1.25	6.74	4.43	3.96	3.63	3.21	2.96	2.60	2.42	2.32
	1.5	6.83	4.48	3.99	3.66	3.24	2.98	2.62	2.44	2.33
4.5	0.125	3.13	2.56	2.43	2.34	2.21	2.13	2.01	1.95	1.91

	0.375	5.88	3.99	3.59	3.32	2.98	2.76	2.46	2.31	2.22
	0.625	7.35	4.75	4.22	3.85	3.39	3.10	2.71	2.51	2.39
	0.875	8.11	5.15	4.54	4.13	3.60	3.27	2.83	2.61	2.47
	1	8.33	5.27	4.63	4.21	3.66	3.33	2.87	2.64	2.50
5.5	0.125	3.37	2.69	2.53	2.42	2.28	2.19	2.05	1.98	1.94
	0.375	7.09	4.62	4.11	3.76	3.31	3.04	2.66	2.47	2.36
	0.625	9.13	5.68	4.97	4.49	3.88	3.51	3.00	2.74	2.59
	0.875	10.21	6.24	5.43	4.88	4.18	3.76	3.18	2.89	2.71
	1	10.52	6.41	5.57	5.00	4.27	3.83	3.24	2.93	2.75
6	0.125	3.51	2.76	2.59	2.47	2.32	2.22	2.08	2.00	1.95
	0.375	7.78	4.98	4.40	4.01	3.51	3.20	2.78	2.56	2.44
	0.625	10.17	6.23	5.42	4.87	4.17	3.75	3.18	2.88	2.71
	0.875	11.43	6.88	5.95	5.32	4.53	4.04	3.39	3.06	2.86
	1	11.81	7.08	6.11	5.46	4.64	4.13	3.45	3.11	2.90

For level terrain types, the PCE value for trucks and buses is 2.0. For specific segments, the recommended PCE values increase with the grade, length, and percentage of heavy trucks, and decrease with the truck percentage. The recommended PCEs for level freeway segments with zero grade ranges between 1.83 and 2.62.

Same as HCM 2010, the level terrain in HCM 2016 is also defined as: “any combination of grades and horizontal or vertical alignment that permits heavy vehicles to maintain the same speed as passenger cars, and this type of terrain typically contains short grades of no more than 2%” (HCM 2016, p. 12-35). The grades for all data collection sites can be defined as the level terrain type in HCM 2016. The level terrain definition indicates that, PCEs at the level terrain are determined based on the assumption that passenger cars and trucks have the same average speed.

For truck percentage higher than 25%, HCM 2016 recommends the PCE value as 25% truck percentage, since the highest truck percentage published here is 25%.

5.2 Operational Analysis

5.2.1 Operational Analysis with Empirical Data

1. Operational Analysis in HCM 2010

There are six steps for operational analysis in HCM 2010 (HCM 2010, p. 11-10):

Step 1: Input data – geometric data and demand volume

Step 2: Compute free-flow speed (FFS)

Step 3: Select FFS curve

Step 4: Adjust demand volume

Step 5: Estimate speed and density

Step 6: Determine level-of-service (LOS) (A-E)

In Step 1, the input data include geometric data and demand volume. In Step 2, the free-flow speed is estimated using the Equation 5.1:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22TRD^{0.84} \tag{5.1}$$

FFS	free flow speed of basic freeway segment (mph)
f_{LW}	adjustment for lane width (mph)
f_{LC}	adjustment for right-side lateral clearance (mph)
TRD	total ramps density (ramps/mi)

For all study sites, the lane width is 12 feet. This is the base condition, which produces no negative effects on the free-flow speed. Therefore, the value of f_{LW} is set as 0. The right-site lateral clearance for all study sites is 6 feet. This is also the base condition, and the value of f_{LC} is set as 0. Total ramp density is defined as the quotient of the number of ramps (on and off in one direction) located between 3 miles upstream and 3 miles downstream of the midpoint of the basic freeway segment under study, and the segment distance. The ramp density is used to estimate the impact of merging and diverging vehicles on the FFS (HCM 2010, p. 11-12). Because the study sites are located in western Nebraska, the ramps are located several miles from one another. There is only one exit and one entrance ramp in a 6 mile segment. The ramp density is 2/6 or 1/3 ramps per mile. Therefore, the free-flow speed for all of the study sites is 74.1 mph, which is close to 75 mph. Therefore, in Step 3, the FFS curve for 75 mph is selected as the basic for the speed-volume analysis (HCM 2010, p. 11-3).

In Step 4, the adjustment factor for heavy vehicles is determined using the Equation 5.2 (HCM 2010, p. 11-13).

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} \tag{5.2}$$

f_{HV}	adjustment factor for presence of heavy vehicles in traffic stream
P_T	proportion of trucks and buses in traffic stream
P_R	proportion of recreation vehicles in traffic stream
E_T	passenger car equivalent (PCE) of one truck or bus in traffic stream
E_R	passenger car equivalent (PCE) of one recreation vehicle in traffic stream

Because of very low percentage of buses and recreation vehicles, only trucks are considered in the adjusted factor determination of heavy vehicles. The proportion of trucks in the traffic stream is based on the truck percentage at each 15-min interval, and the passenger car equivalent for a truck is set as 1.5, as recommended in the HCM 2010.

The adjustment demand flow rate is determined using Equation 5.3 (HCM 2010, p. 11-13).

$$v_P = \frac{V}{PHF \times N \times f_{HV} \times f_p} \tag{5.3}$$

v_P	adjusted demand flow rate under equivalent base conditions (pc/h/ln)
V	demand volume under prevailing conditions (veh/h)
PHF	peak-hour factor
N	number of lanes in analysis direction
f_{HV}	adjustment factor for presence of heavy vehicles in traffic stream
f_p	adjustment factor for unfamiliar driver populations

Here, the demand volume under the prevailing condition V is the hourly flow rate based on a 15-min interval empirical volume. Thus, V and the adjusted demand flow rate under equivalent base conditions v_P are calculated for each 15-min interval. The adjustment factor for unfamiliar driver populations was set to 1.0 because it was assumed that the driver was familiar with the routes on level terrain. The PHF represents the variation in traffic flow within an hour, and is determined by the following equation. For the I-80 test sites, the value ranges from 0.69 to 0.98.

After Step 4, the v_P need to be compared with the base capacity of the basic freeway segment shown in the selected FFS curve in Step 3. The capacity of the FFS curve for 75 mph is 2400 pc/h/ln. If the demand exceeds capacity, the LOS is F and a breakdown is identified. Otherwise, the analysis continues to Step 5.

In Step 5, the estimated mean speed of the v_P needs to be computed using the selected FFS curve in Step 3. The equation of the FFS curve for 75 mph is as follows:

$$S = \begin{cases} 75\ (v_P \leq 1000) \\ 75 - 0.00001107\left(v_p - 1000\right)^2\ (v_P > 1000) \end{cases} \tag{5.4}$$

S means speed of traffic stream under base conditions (mph)

v_P adjusted demand flow rate under equivalent base conditions (pc/h/ln)

Equation 5.5 is used to calculate the density for each 15-min interval:

$$D = \frac{v_P}{S} \tag{5.5}$$

D Density (pc/mi/ln)

v_P adjusted demand flow rate under equivalent base conditions (pc/h/ln)

S means speed of traffic stream under base conditions (mph)

In Step 6, the LOS for each 15-min interval is determined by Table 5.7.

Table 5.7 LOS criteria for basic freeway segment (HCM 2010 and HCM 2016)

Level of service (LOS)	**Density (pc/mi/ln)**
A	<=11
B	>11-18
C	>18-26
D	>26-35
E	>35-45
F	>45

2. Operational Analysis in HCM 2016

There are six steps for operational analysis in HCM 2016 (HCM 2016, p. 12-26):

Step 1: Input data – geometric data and demand volume

Step 2: Estimate and adjust free-flow (FFS) and select FFS curve

Step 3: Estimate and adjust capacity

Step 4: Adjust demand volume

Step 5: Estimate speed and density

Step 6: Determine level-of-service (LOS) (A-E)

In Step 1, the input data include geometric data and demand volume. In Step 2, the free-flow speed is estimated using the following equation:

$$FFS = BFFS - f_{LW} - f_{LC} - 3.22TRD^{0.84} \tag{5.6}$$

FFS free flow speed of basic freeway segment (mph)

$BFFS$ base free flow speed for the basic freeway segment (mph)

f_{LW} adjustment for lane width (mph)

f_{LC} adjustment for right-side lateral clearance (mph)

TRD total ramps density (ramps/mi)

Then, an adjusted free-flow speed is calculated by the following equation:

$$FFS_{adj} = FFS \times SAF \tag{5.7}$$

FFS_{adj} adjusted free-flow speed (mph)

FFS free flow speed of basic freeway segment (mph)

SAF speed adjustment factor

The default value for $BFFS$ is 75.4 mph. For all study sites, the value of f_{LW}, f_{LC}, and TRD are the same as HCM 2010 operational analysis. Therefore, the free-flow speed FFS for all of the study sites is 74.1 mph. The speed adjustment factor can represent a combination of sources, including weather and work zone effects. The default value of SAF is 1.0, which is used here. The adjusted free-flow speed FFS_{adj} is 74.1 mph, which is close to 75 mph. Therefore, the FFS curve for 75 mph is selected as the basic for the speed-volume analysis (HCM 2016, p. 12-11).

Step 3 estimates the capacity using the following equation:

$$c = 2200 + 10 \times (FFS_{adj} - 50) \tag{5.8}$$

c base capacity for a basic freeway segment (pc/h/ln)

FFS_{adj} adjusted free-flow speed (mph)

Step 3 adjusts the capacity using the following equation:

$$c_{adj} = c \times CAF \tag{5.9}$$

c_{adj} adjusted capacity of segment (pc/h/ln)

c base capacity for a basic freeway segment (pc/h/ln)

CAF capacity adjustment factor

The base capacity for basic freeway segment c is 2400 pc/h/ln since this is the maximum capacity according to HCM 2016 (HCM 2016, p. 12-32). The capacity adjustment factor has several components, including weather, incident, work zone, driver population, and calibration adjustments. The default value of CAF is 1.0 and used here. Therefore, the adjusted capacity c_{adj} is 2400 pc/h/ln.

In Step 4, the adjustment factor for heavy vehicles is determined using the following equation (HCM 2016, p. 12-34).

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} \tag{5.10}$$

f_{HV} adjustment factor for presence of heavy vehicles in traffic stream

P_T proportion of trucks and buses in traffic stream

E_T passenger car equivalent (PCE) of one truck or bus in traffic stream

The proportion of trucks in the traffic stream is based on the truck percentage at each 15-min interval, and the passenger car equivalent for a truck is set in Table 5.4 as recommended in the HCM 2016. The adjustment demand flow rate is determined using the following equation (HCM 2016, p. 12-33).

$$v_P = \frac{V}{PHF \times N \times f_{HV}} \tag{5.11}$$

v_P adjusted demand flow rate under equivalent base conditions (pc/h/ln)

V demand volume under prevailing conditions (veh/h)

PHF peak-hour factor

N number of lanes in analysis direction

f_{HV} adjustment factor for presence of heavy vehicles in traffic stream

The demand volume under the prevailing condition V is the hourly flow rate based on a 15-min interval empirical volume. Thus, the V and the adjusted demand flow rate under equivalent base conditions v_P are calculated for each 15-min interval. The PHF represents the variation in traffic flow within an hour. For the I-80 test sites, the value ranges from 0.69 to 0.98.

After Step 4, the v_P needs to be compared with the base capacity of the basic freeway segment calculated in Step 3. If the demand exceeds capacity, the LOS is F and a breakdown is identified. Otherwise, the analysis continues to Step 5.

In Step 5, firstly the estimated mean speed of the v_P needs to be computed using the selected FFS curve in Step 3. In HCM 2016, the means speed is a function of adjusted free-flow speed, adjusted capacity, density at capacity, adjusted demand flow rate, and break point. The equation of the curve for FFS 75 mph for freeway segment is as follows:

$$S = \begin{cases} 75 \ (v_P \leq 1000) \\ 75 - 0.00001105\left(v_p - 1000\right)^2 \ (v_P > 1000) \end{cases} \tag{5.12}$$

S means speed of traffic stream under base conditions (mph)

v_P adjusted demand flow rate under equivalent base conditions (pc/h/ln)

The following equation is used to calculate the density for each 15-min interval:

$$D = \frac{v_P}{S} \tag{5.13}$$

D density (pc/mi/ln)

v_P adjusted demand flow rate under equivalent base conditions (pc/h/ln)

S means speed of traffic stream under base conditions (mph)

In Step 6, the LOS for each 15-min interval is determined by Table 5.7.

5.2.2 Comparison between Empirical Data and HCM 2010/2016 Operational Analysis Results

1. Comparison between empirical data with HCM 2010 operational analysis results

The estimated hourly demand flow rate v_P calculated in HCM 2010 operational analysis and the empirical speed for each 15-minute interval are shown in Figure 5.1.

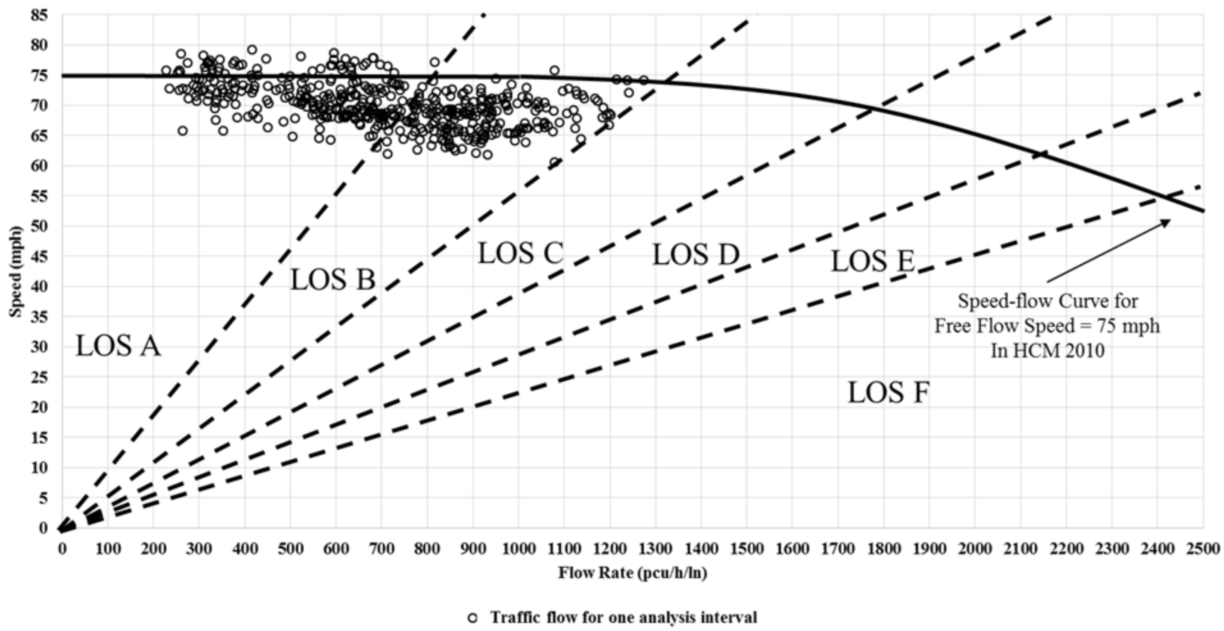


Figure 5.1 The observed traffic flow, FFS = 75 mph speed-flow curve, and the operational analysis results with PCE = 1.5 in HCM 2010

Also shown in the figure is the FFS curve for 75 mph and the corresponding level of service (LOS). In Figure 5.1, one individual data point represents the empirical speed and flow rate data in one 15-minite interval. The curve is the FFS curve for 75 mph, as used in the HCM 2010. The “flow rate” represents the flow rate per lane per hour, and the “speed” represents the average speed for two lanes in a 15-minute interval. The area of the LOS, based on the density values recommended in the HCM 2010, is also shown in the figures. The different levels of LOS are separated by dash lines. The proportion of different LOS levels for all analysis intervals is shown in Figure 5.2. Figure 5.1 and Figure 5.2 is based on a PCE of 1.5, and concludes the following:

1) The hourly traffic flow rate on I-80 is comparatively low. According to the figure, the flow rate per lane per hour is less than 1300 veh/h/ln.

2) For all 15-minute-interval traffic flow observations on two lanes, the level of service for traffic flow ranges from level A to B. Therefore, based on density, the level of service for I-80 is high.

3) In general, the observed speed is lower than the predicted curve. For 85% of all 15-minute intervals, the empirical speed of traffic flow is 0 to 15 mph (0 to 20 percent) lower than the speed calculated by the FFS curve for 75 mph *S* at the corresponding flow rate.

Figure 5.1 demonstrates that the observed speeds correspond to the LOS C or D section on the curve. If the HCM 2010 PCE value is used, 54 percent of the sites would be classified as having a LOS A and 46 percent of the sites would be classified as having a LOS B, as shown in Figure 5.2.

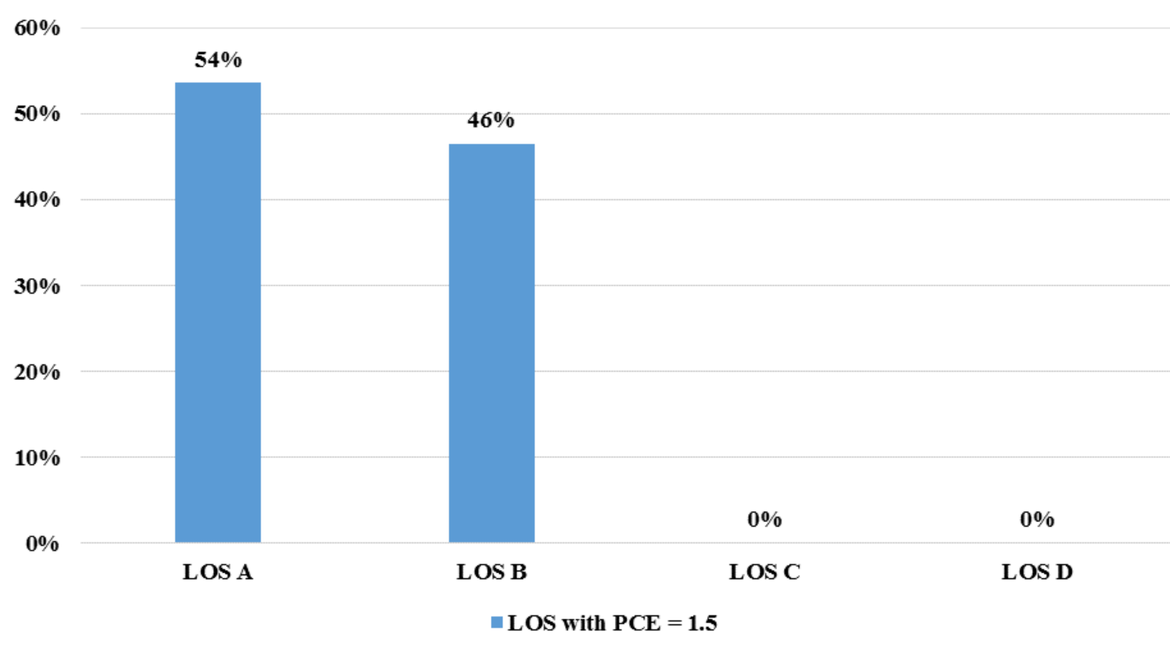


Figure 5.2 Operational analysis results (LOS) using PCE = 1.5 in HCM 2010

2. Comparison between empirical data with HCM 2016 operational analysis results

The estimated hourly demand flow rate v_P calculated in HCM 2016 operational analysis, the empirical speed for each 15-minute interval, the FFS curve for 75 mph, and the corresponding level of service (LOS) are shown in Figure 5.3. Figure 5.3 is explained in the same way as Figure 5.1. The proportion of different LOS levels for all analysis intervals is shown in Figure 5.4.

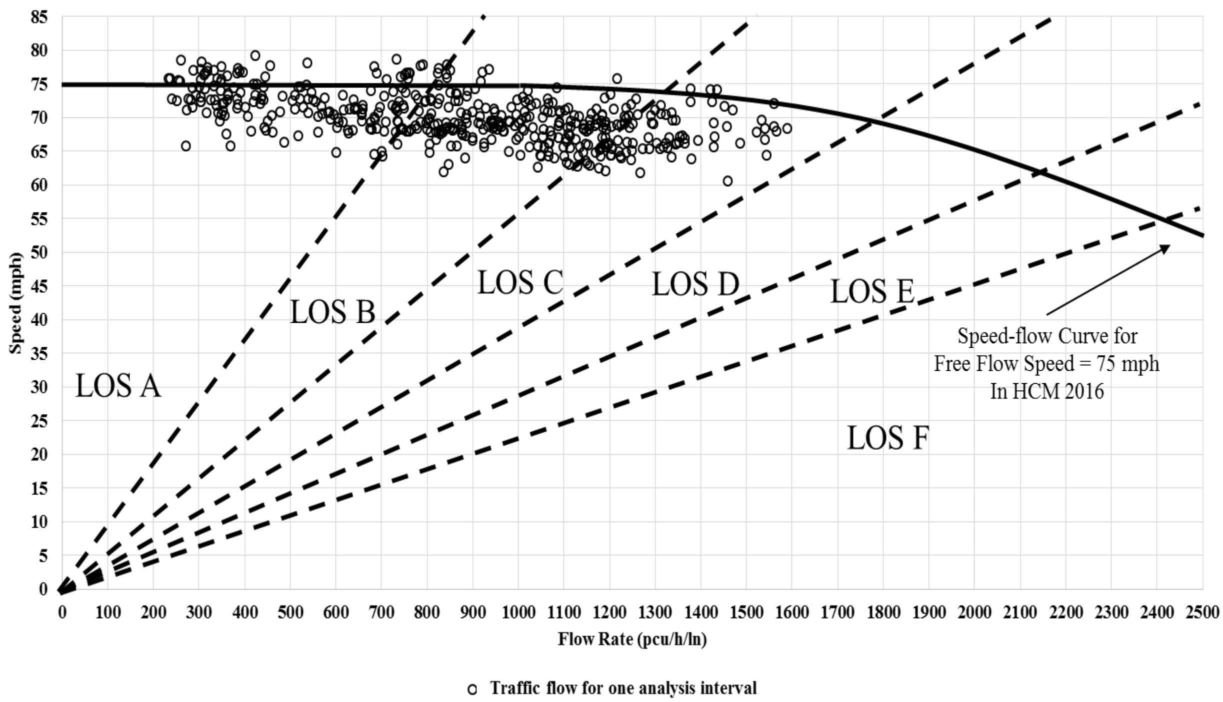


Figure 5.3 The observed traffic flow, FFS = 75 mph speed-flow curve, and the operational analysis results with PCE for level freeway segments in HCM 2016

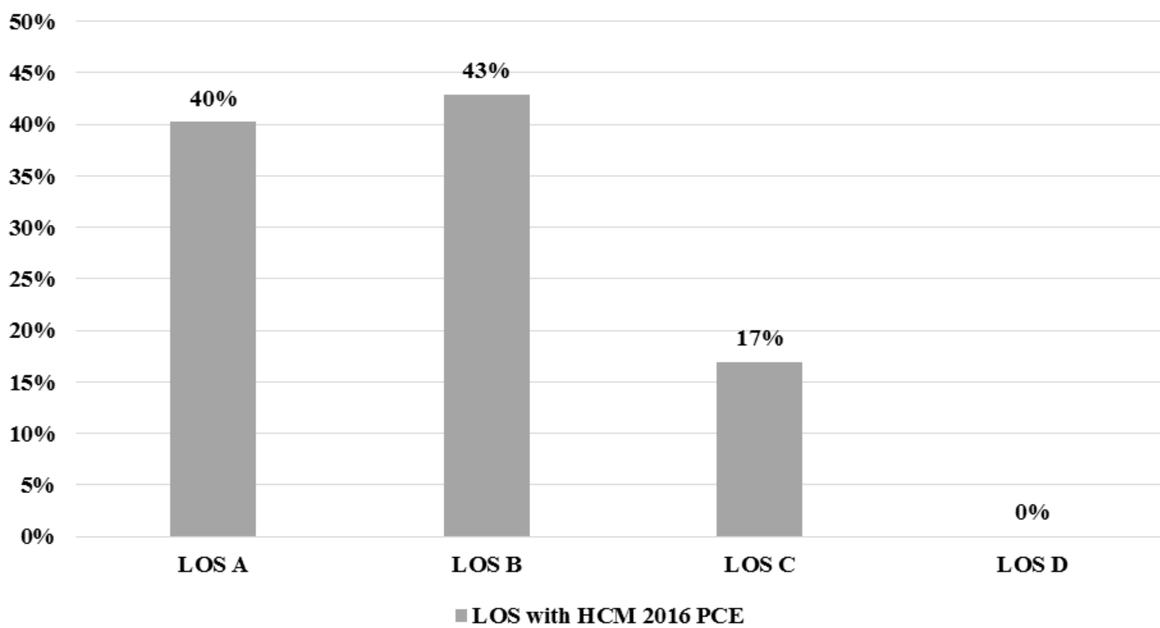


Figure 5.4 Operational analysis results (LOS) using PCE for level freeway segments in HCM 2016

Figure 5.3 and Figure 5.4 is based on the PCE values in Table 5.4, and concludes the following:

1) The hourly traffic flow rate on I-80 is low to moderate. According to the figure, the flow rate per lane per hour is less than 1500 veh/h/ln.
2) For all 15-minute-interval traffic flow observations on two lanes, the level of service ranges from level A to C. Therefore, based on density, the level of service for I-80 is high to moderate.
3) In general, the observed speed is lower than the predicted curve. For 85% of all 15-minute intervals, the empirical speed of traffic flow is 0 to 15 mph (0 to 20 percent) lower than the speed calculated by the FFS curve for 75 mph S at the corresponding flow rate.

Figure 5.3 The observed traffic flow, FFS = 75 mph speed-flow curve, and the operational analysis results with PCE for level freeway segments in HCM 2016 displays that the observed speeds correspond to the LOS C or D section on the curve. If the HCM 2016 PCE value is used, 40 percent of the sites would be classified as having a LOS A, 43 percent of the sites would be classified as having a LOS B, and 17 percent of sites would be classified as having a LOS C, as shown in Figure 5.4 Operational analysis results (LOS) using PCE for level freeway segments in HCM 2016. It is argued that the higher PCE values may better capture the relationship between heavy vehicles and passenger cars on this corridor because of the presence of the large percentage of moving bottlenecks.

5.3 Summary

In this chapter, the recommended value for the PCE and the operational analysis method in HCM 2010 and 2016 are introduced. In the HCM 2010, for basic freeway segments, the PCEs are 1.5 for level terrain at all lengths and truck percentages. For other grades the PCEs increase with grade and length, and decrease with truck percentage. In HCM 2016, for basic freeway segments, the recommended PCE values are 2.0 for level terrain. The PCEs increase with the grade, length (expect -2% and 0% grade), and percentage of heavy trucks, and decrease with the truck percentage. In either HCM 2010 or 2016, PCEs at level terrain are determined under the assumption that passenger cars and trucks have the same average speed. The operational analysis based on the empirical data is carried on and compared to the FFS curve provided in the HCM 2010 and 2016. The results show that for I-80 between Lincoln and North Platte, Nebraska, the level of service is at level A or B by HCM 2010 method, and at level C or D by HCM 2016 method. However, the operating speeds are lower than those predicted by the FFS curve. These speeds correspond to a LOS at C or D. It is hypothesized that a moving bottleneck or a speed difference between passenger cars and trucks results. If this is true, the HCM 2010 and HCM 2016 PCE values may be too low.

CHAPTER 6 MOVING BOTTLENECKS AND LOCALIZED CONGESTION ANALYSIS

6.1 Moving Bottlenecks Identification

6.1.1 Moving Bottlenecks Definition

In general, a moving bottleneck is caused by fast vehicles that catch up with slower vehicles without being able to pass them (36). Vehicles that do not catch up with other vehicles, or vehicles that are not impeding other vehicles, are defined as “free vehicles.” These vehicles are not in the moving bottleneck. There is no universally accepted standard for identifying a moving bottleneck, and studies that investigated car-following interactions aimed at identifying free vehicles are limited. On freeways, highways, and urban roads, one accepted definition of a free vehicle is that its speed is not influenced by the speed of the vehicle traveling ahead of it (31 and 60). This is usually identified by headways between the leading and following vehicles. In order to be considered a free vehicle, the headways should be greater than a specific threshold (32 and 33). In this dissertation, a moving bottleneck is defined as a group of vehicles traveling on either the median or the shoulder lane in the same direction, which influence the speed of one another.

In this dissertation, a moving bottleneck is identified based on the leading headway and lagging headway. The leading headway represents the time between the front bumper of the leading vehicle passing a specific location and the front bumper of the following vehicle passing the same location. Similarly, the lagging headway represents the time between the rear bumper of the leading vehicle passing a specific location and the rear bumper of the following vehicle passing the same location. The thresholds for moving bottleneck identification are defined as the “critical leading headway” and the “critical lagging headway", respectively. Vehicles on all lanes in one direction with leading headways less than or equal to the critical leading headway, or with lagging headways less than or equal to the critical lagging headway, are considered to belong to the same moving bottleneck. In contrast, vehicles with both leading and lagging headways greater than the corresponding critical headways are considered independent (e.g., not in a moving bottleneck). All vehicles identified in moving bottlenecks are defined as "in-moving-bottleneck vehicles", and all vehicles identified not in moving bottlenecks are defined as "free-flow vehicles". The moving bottleneck is defined by Equation 6.1. This concept is illustrated by Figure 6.1.

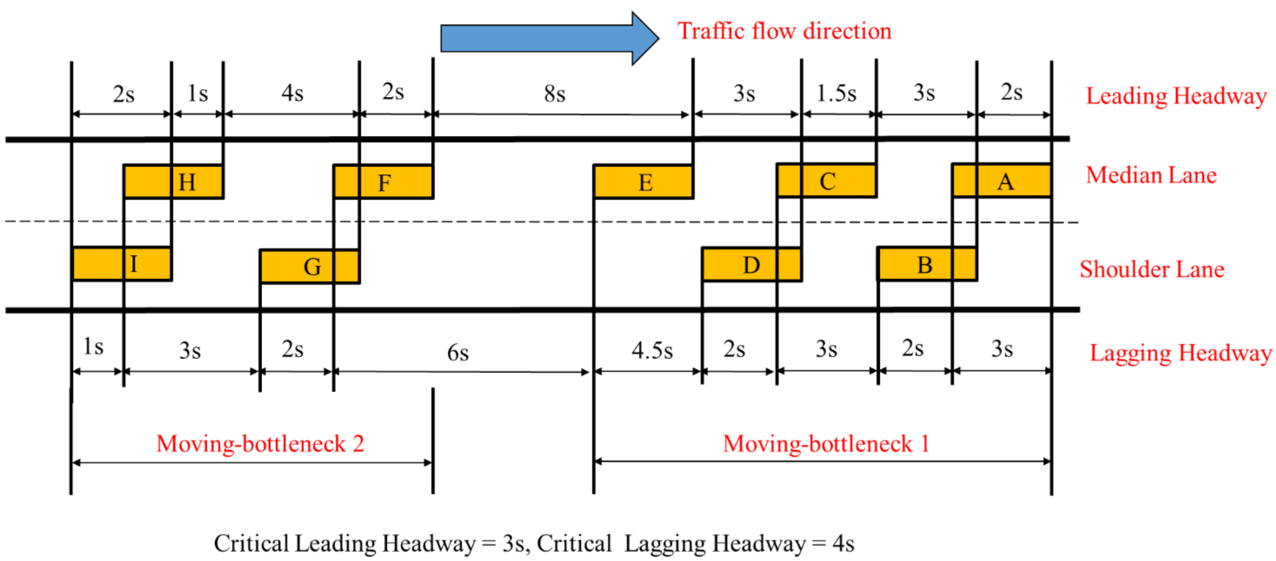


Figure 6.1 Moving-bottlenecks definition

In this example, the critical leading headway is set as 3s, and the critical lagging headway is set as 4s.

$$p_i = \begin{cases} 1 \left(F_i - F_{i+1} \le C_{leading} \; and \; R_i - R_{i+1} \le C_{lagging}\right) \\ 1 \left(F_i - F_{i+1} \le C_{leading} \; and \; R_i - R_{i+1} > C_{lagging}\right) \\ 1 \left(F_i - F_{i+1} > C_{leading} \; and \; R_i - R_{i+1} \le C_{lagging}\right) \\ 0 \left(F_i - F_{i+1} > C_{leading} \; and \; R_i - R_{i+1} > C_{lagging}\right) \end{cases} \quad (6.1)$$

p_i	indicator for vehicle whether in moving bottleneck or not, 1-in moving bottleneck, 0-not in moving bottleneck
i	order of vehicle, $i + 1$ means its following vehicle
F_i	time front bumper of leading vehicle *i* passes specific location
F_{i+1}	time front bumper of following vehicle $i + 1$ passes specific location
$C_{leading}$	critical leading headway
R_i	time rear bumper of leading vehicle *i* passes specific location
R_{i+1}	time rear bumper of following vehicle $i + 1$ passes specific location
$C_{lagging}$	critical lagging headway

It may be seen that there are two moving bottlenecks. Each moving bottleneck has one or two moving bottleneck leaders that influence the speed of the following vehicles. For moving bottlenecks occurring on two lanes, both the first vehicle on the shoulder lane and the first vehicle on the median lane are regarded as the “moving bottleneck leaders.” In Figure 6.1, for moving bottleneck 1, the leaders are A and B, and for moving bottleneck 2 the leaders are F and G. Note that for moving bottlenecks occurring on one lane, only the first vehicle in the moving bottleneck is regarded as the “moving bottleneck leader.” All of the vehicles in the moving bottleneck except for the leaders are, by definition, “moving bottleneck followers” (e.g., vehicle except A, B, F, and G in Figure 6.1).

6.1.2 Critical Headway and Free Vehicles Identification

The critical headway is considered not only different between the leading and lagging headways, but also affected by the classification of the leading and lagging vehicles. There are eight critical headway classifications in this report: four related to leading headways and four related to lagging headways:

1) Car following car, leading headway (cc-leading);
2) Car following truck, leading headway (ct-leading);
3) Truck following car, leading headway (tc-leading);
4) Truck following truck, leading headway (tt-leading);
5) Car following car, lagging headway (cc-lagging);
6) Car following truck, lagging headway (ct-lagging);
7) Truck following car, lagging headway (tc-lagging); and
8) Truck following truck, lagging headway (tt-lagging).

The critical headway is determined based on the following standards.

1) The speed of the vehicles with headways no greater than the critical headway is lower than the speed of the vehicles with headways greater than the critical headway.
2) The vehicles with headways no greater than the critical headway show a high linear relationship between the speed and headway, while vehicles with headways greater than critical headway show a low linear relationship between the speed and headway.

Table 6.1 shows the critical headway for each of the eight types. The figures for the critical headways determination are shown in the appendix. Note that the average headway is 5.25s, relatively close to 6 seconds predicted by Al-Kaisy (31).

Table 6.1 Results of critical headway determination

Headway Type	Critical Headways (s)	Headway Type	Critical Headways (s)
cc leading	3.0	cc lagging	3.0
ct leading	8.0	ct lagging	7.0
tc leading	6.0	tc lagging	6.0
tt leading	5.0	tt lagging	4.0

6.1.3 Frequency and Percentage of In-Moving-Bottlenecks and Free Vehicles

Based on the critical leading and lagging headways in Table 6.1, it was found that for the 48,561 passenger cars and trucks observed, 42,308 vehicles (87.1% of all vehicles) were identified as belonging to a moving bottleneck, and 6,253 vehicles (12.9% of all vehicles) were identified as free-flow vehicles. The frequency and percentage of in-moving bottleneck and free-flow vehicles are shown in Figure 6.2 and Figure 6.3, respectively. It is hypothesized that moving bottlenecks affect the speed of vehicles on I-80 and that the moving bottlenecks occur because of the different free-flow speeds of passenger cars and trucks.

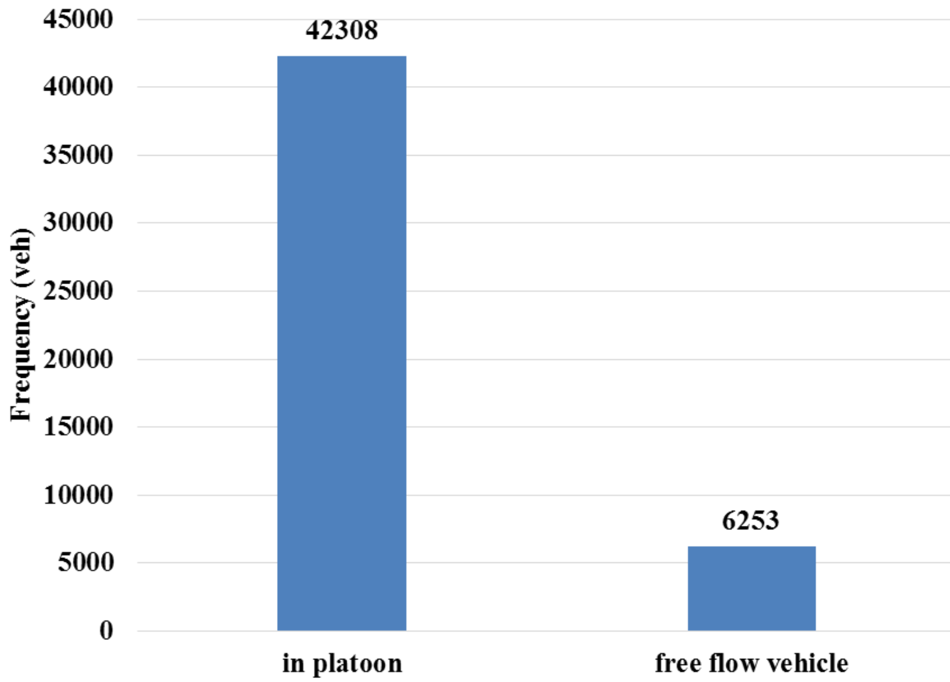


Figure 6.2 Frequency of vehicles in moving bottlenecks

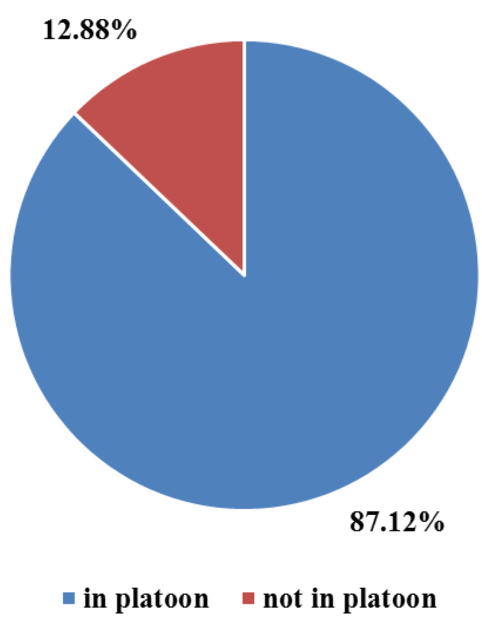


Figure 6.3 Frequency of vehicles in moving bottlenecks

6.1.4 Moving Bottlenecks Type and Vehicle Type

In this dissertation, moving bottlenecks are divided into eight types, summarized in Table 6.2. The frequency and percentage of different moving bottleneck types are shown in Figure 6.4 and Figure 6.5, respectively.

Table 6.2 Summary of moving bottleneck type

Moving bottleneck Type	Lanes	Leader on Shoulder Lane	Leader on Median Lane
Type I	Two lanes	Passenger car	Passenger car
Type II	Two lanes	Truck	Passenger car
Type III	Two lanes	Passenger car	Truck
Type IV	Two lanes	Truck	Truck
Type V	One lane	Passenger car	--
Type VI	One lane	--	Passenger car
Type VII	One lane	Truck	--
Type VIII	One lane	--	Truck

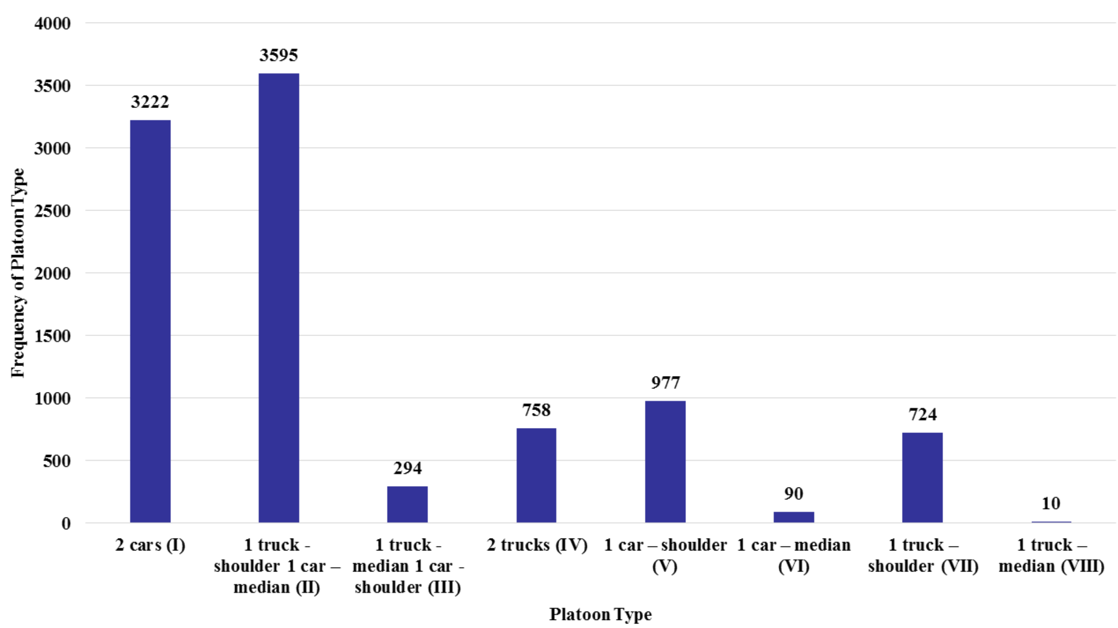


Figure 6.4 Frequency of moving bottleneck types

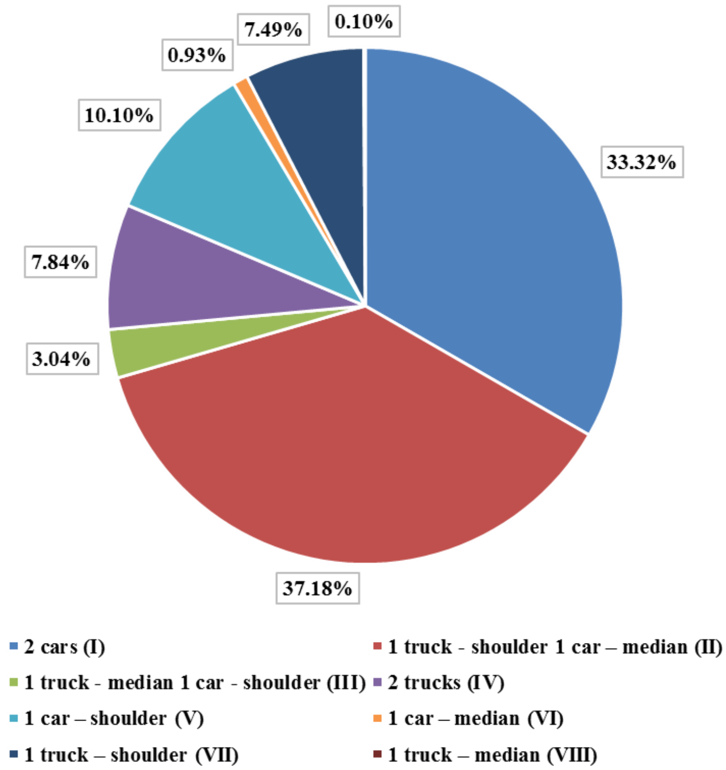


Figure 6.5 Percentage of moving bottleneck types

The results are detailed below.

1) Nearly 70% of moving bottlenecks are classified into Type I and Type II moving bottlenecks, which are led by two cars, or one truck on a shoulder and one car on a median.

2) The number of two-lane moving bottlenecks (e.g., Type I, II, III, and IV) is 30% higher than the number of one-lane moving bottlenecks.

3) One-lane moving bottlenecks occur more frequently on the shoulder lane than the median lane (e.g., the number of one-lane moving bottlenecks on the shoulder lane is 17 times that of a median lane);

4) For two-lane moving bottlenecks with one car moving bottleneck leader and one truck moving bottleneck leader, the car moving bottleneck leader is more likely to be in the median lane (e.g., the number of car moving bottleneck leaders on the median lane is 12 times that of the shoulder lane), and the truck moving bottleneck leader is more likely to be in the shoulder lane (e.g., the number of truck moving bottleneck leaders on the shoulder lane is 12 times that of the median lane).

The vehicles are grouped into 4 types according to two vehicle classification (e.g., passenger cars and trucks) and lane type (e.g., shoulder lane and median lane), summarized in Table 6.3. The frequency and percentage of the four vehicle-type lane combinations are shown in Figure 6.6 and Figure 6.7, respectively.

Table 6.3 Summary of vehicle type

Vehicle Type	**Vehicle Classification**	**Lane**
Type A	Passenger car	Shoulder lane
Type B	Passenger car	Median lane
Type C	Truck	Shoulder lane
Type D	Truck	Median lane

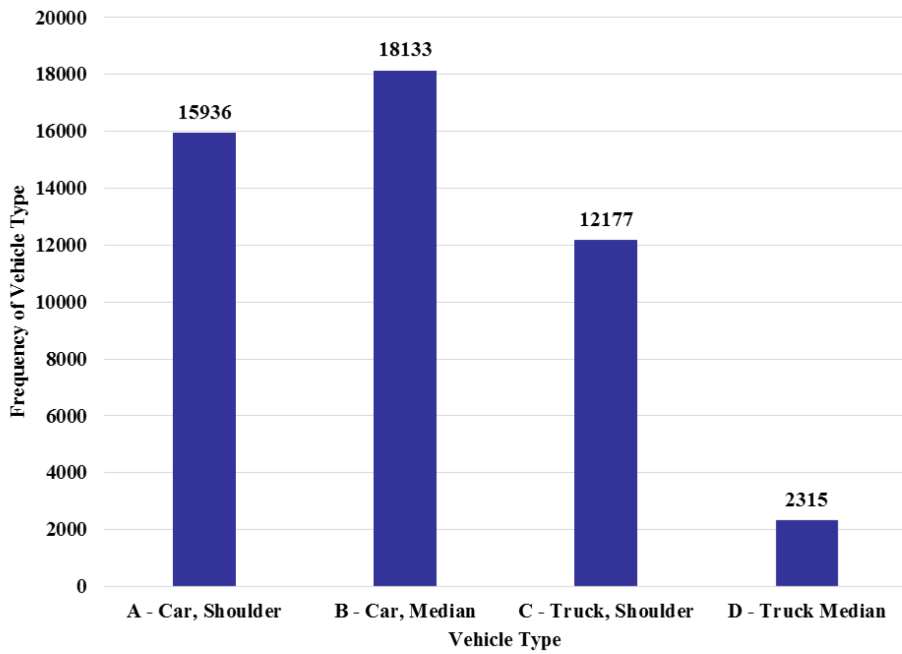


Figure 6.6 Frequency of vehicle type

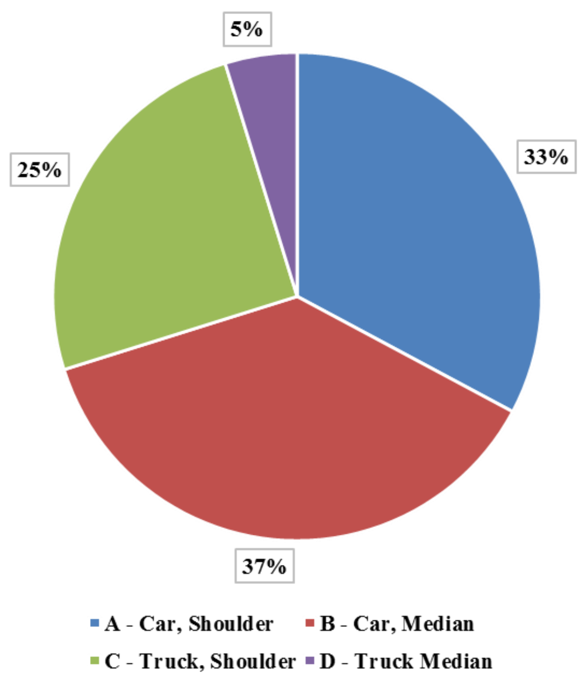


Figure 6.7 Percentage of vehicle type

The vehicles also have classifications of “in-moving-bottleneck vehicles” and “free-flow vehicles” that are not shown. For passenger cars, the number of vehicles on the median lane is 13.8% higher than that on the shoulder lane. For trucks, the number of vehicles on the shoulder lane is 4.26 times as high as that on the median lane. Nearly 95% of all of the vehicles are classified into type A, B, and C. Very few vehicles are type D, indicating that very few trucks operate on the median lane.

6.2 Analysis for Impeded and Free-flow Vehicles

6.2.1 Definition of Impeded and Free Vehicles

Based on moving bottleneck identification, the vehicles are classified into three groups according to whether:

1) The vehicle is impeding other vehicles;
2) The vehicle is impeded by other vehicles;
3) The vehicle is neither impeding nor being impeded upon (e.g., free-flow vehicles).

In this research, all moving bottleneck followers are defined as “impeded vehicles,” under the assumption that the operating speed is constrained by the leading vehicles of the moving bottleneck. All moving bottleneck leaders are defined as “impeders,” on the assumption that the following vehicles adjust their operating speed because of the presence of the “impeders.” All free-flow vehicles are defined as “non-impeded vehicles,” because their operating speeds are not affected by impeding or impeded vehicles.

6.2.2 Frequency and Percentage of Impeder, Impeded, and Free Vehicles

The frequency and percentage of the impeder, impeded, and non-impeded vehicles are shown in Figure 6.8 and Figure 6.9, respectively. These figures show that 51% of vehicles are classified as impeded vehicles, 36% are impeders, and approximately 13% are non-impeded vehicles. The relatively high number of vehicles classified as impeders indicates there are a high number of small platoons. This will be analyzed later in this chapter.

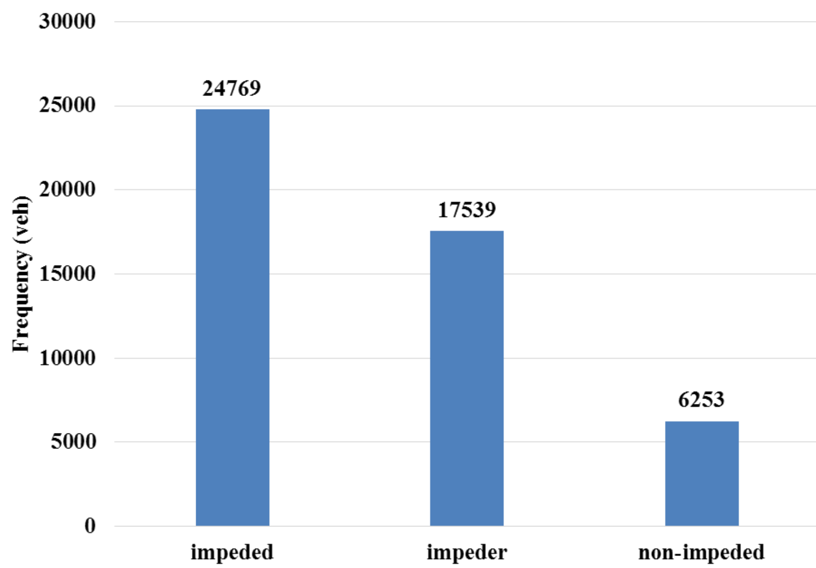


Figure 6.8 Frequency of impeder, impeded and non-impeded vehicles

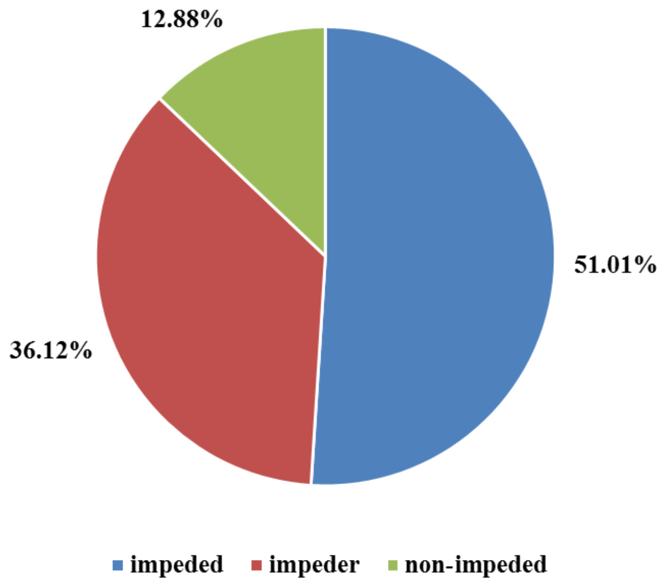


Figure 6.9 Percentage of impeder, impeded and non-impeded vehicles

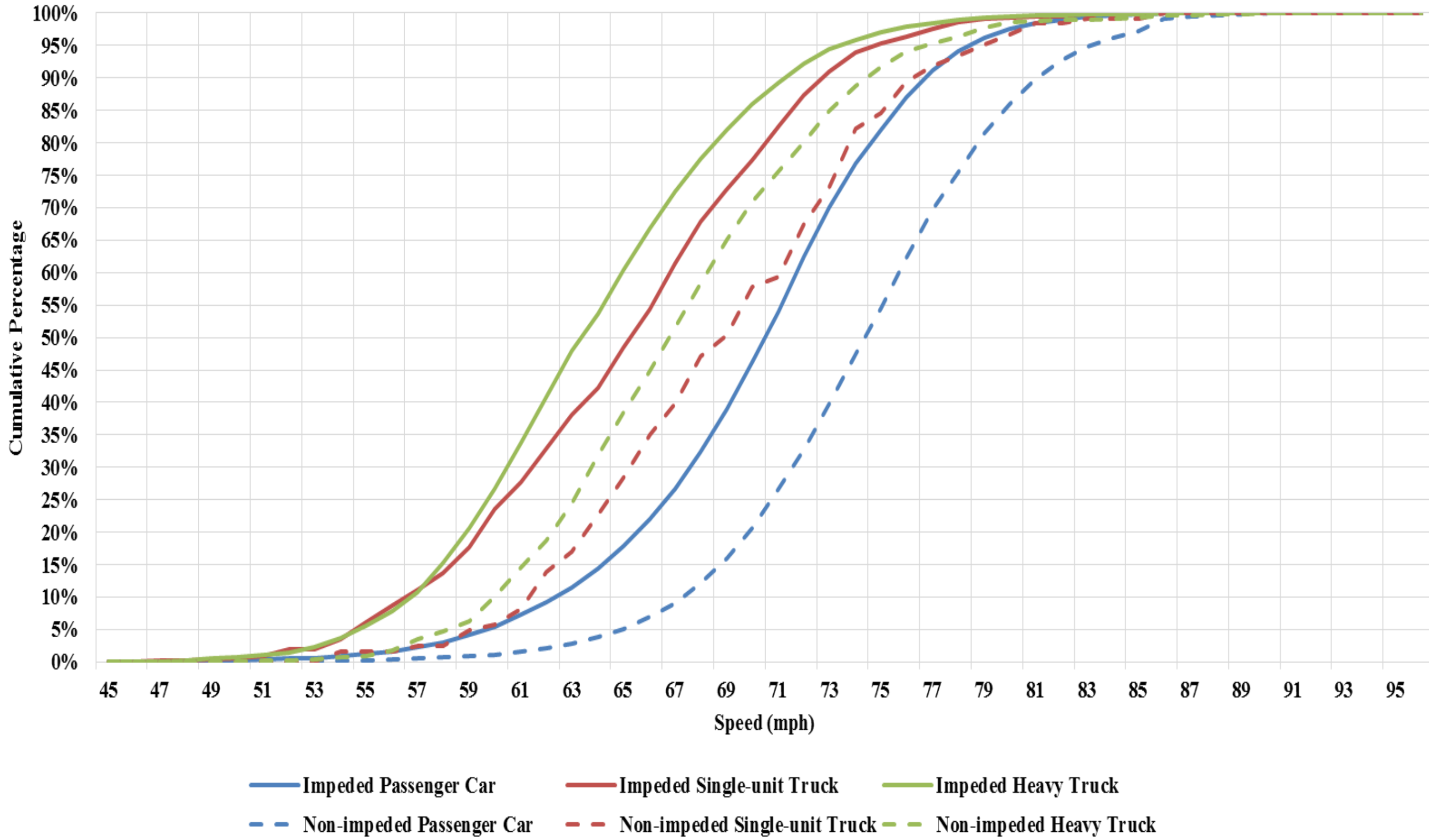


Figure 6.10 Speed distribution for impeded and non-impeded vehicles

6.2.3 Speed Distribution for Impeded and Free Vehicles

The speed distributions for impeded and non-impeded passenger cars, single-unit trucks, and heavy trucks are shown in Figure 6.10.

The difference in the average speeds between impeded and non-impeded vehicles for passenger cars, single-unit trucks, and heavy trucks are also analyzed using a t-test, and the results are shown in Table 6.4.

Table 6.4 Results of T-test for comparisons between impeded and non-impeded vehicle speed

Comparison	**Difference (Impeded – Non-impeded) (mph)**	**Value of T-test**	**P-value**
Speed of Impeded v.s. Non-impeded Passenger Car	-5.2	-47.1 (Difference < 0)	< 0.05
Speed of Impeded v.s. Non-impeded Single-unit Truck	-4.8	-6.47 (Difference < 0)	< 0.05
Speed of Impeded v.s. Non-impeded Heavy Truck	-3.3	-17.91 (Difference < 0)	< 0.05

It is found that the average speeds for cars, single-unit trucks, and heavy trucks classified as impeded vehicles are 7.1%, 6.0%, and 4.5% lower than non-impeded vehicles, statistically significant at the 95% confidence level. Note that for non-impeded vehicles, the average speed of passenger cars is 9.5% higher than trucks, statistically significant at the 95% confidence level. The results indicate that the moving bottleneck causes travel delay to impeded vehicles rather than non-impeded vehicles. It is also found that even if the single-unit and heavy trucks are classified as non-impeded vehicles, their speeds (e.g., with an average speed 68.1 mph) are still lower than passenger cars classified as impeded vehicles (e.g., with an average speed 71.5 mph).

6.2.4 Additional Analysis for Impeded and Free Vehicles

Relationships between traffic volume and the amount of impeded vehicles, percentage of impeded vehicles, and percentage of time impeding are separately analyzed. Figure 4.4 and Figure 4.5 in Chapter 4 show traffic volume and truck percentage as a function of distance from west to east. A negative correlation between traffic volume and truck percentage exists because traffic volume increases as truck percentage decreases. In order to account for this correlation, the empirical data is divided into six groups based on traffic volume and truck percentage:

1) Low volume (<=700 veh/h), medium truck percentage (>25%, <=35%);
2) Low volume (<=700 veh/h), high truck percentage (>35%);
3) Medium volume (>700 veh/h, <=1100 veh/h), low truck percentage (<=25%);
4) Medium volume (>700 veh/h, <=1100 veh/h), medium truck percentage (>25%, <=35%);
5) Medium volume (> 700 veh/h, <=1100 veh/h), high truck percentage (>35%);
6) High volume (>1100 veh/h), low truck percentage (<25%).

The relationship between traffic volume and the number of impeded vehicles is shown in Figure 6.11. The hourly traffic volume (veh/h) and the amount of impeded vehicles (veh/h) are determined based on the 15-minute-interval observation and converted to an hourly count. The diagonal in the figure represents the upper bound situation where all vehicles are impeded. Figure 6.11 shows, that for all combinations of traffic volume and truck percentage, the amount of impeded vehicles increases with the traffic volume. There are 240 estimates in Figure 6.11 (e.g., 60 hour 15-minute intervals). The amount of impeded vehicles ranges from 100 veh/h to 1100 veh/h.

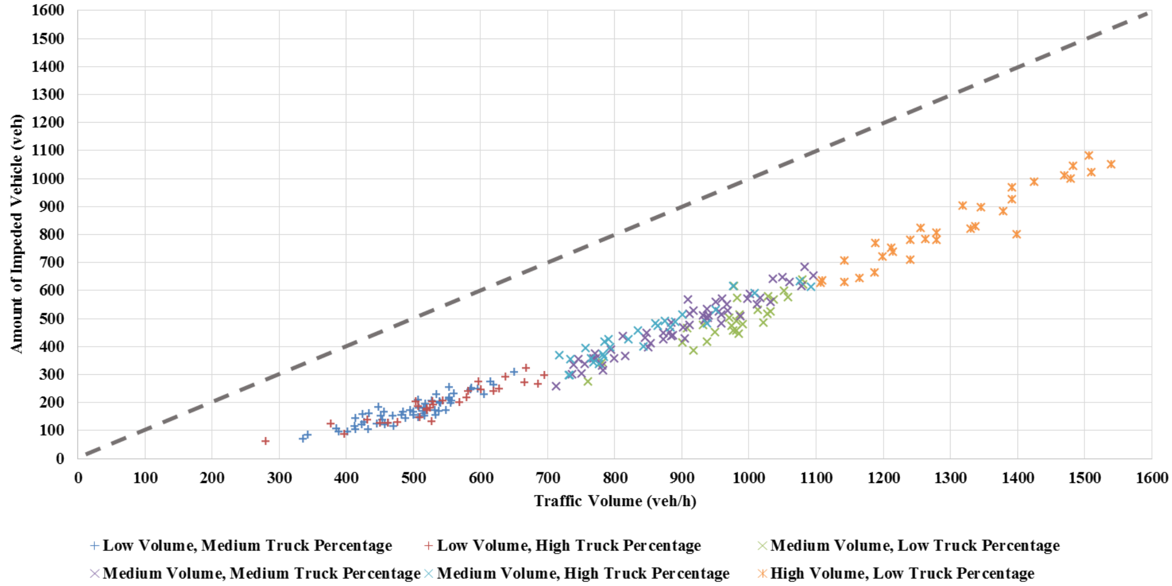


Figure 6.11 Relationship between traffic volume and amount of impeded vehicle

The relationship between the traffic volume and percentage of impeded vehicles is shown in Figure 6.12. The hourly traffic volume (veh/h) and the percentage of impeded vehicles are determined based on the 15-minute interval observation. For all of the combinations of traffic volume and truck percentage, the percentage of impeded vehicles increases with traffic volume, but at a slightly decreased rate. The percentage of impeded vehicles ranges from 20% to 75%.

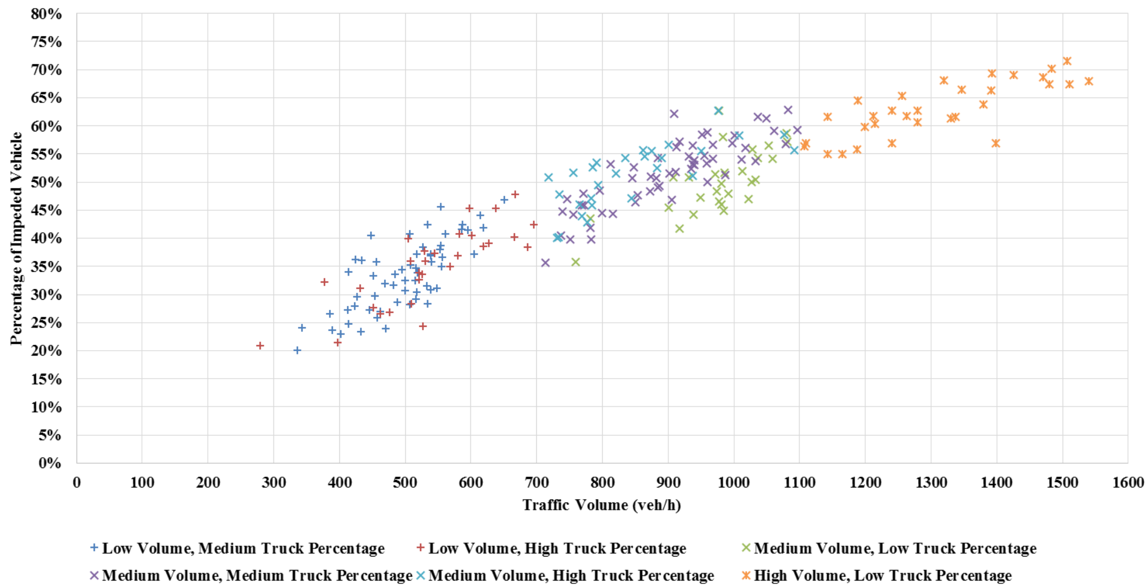


Figure 6.12 Relationship between traffic volume and percentage of impeded vehicle

In order to estimate the time a vehicle is being impeded in a moving bottleneck, the Percentage of Time Impeding (PTI) was created. The PTI is the quotient of the time duration of vehicles being impeded and the interval lasting time, as shown in Equation 6.2.

$$PTI = \frac{T_{impeded}}{T_{interval}} \tag{6.2}$$

PTI	percentage of time impeding
$T_{impeded}$	the time duration of vehicles being impeded
$T_{interval}$	the interval lasting time (e.g. 15 min)

The duration for vehicles being impeded is defined as the sum of the duration of detectors being occupied by impeded vehicles, and the gaps between impeded vehicles and their leading vehicles, as shown in Equation 6.3.

$$T_{impeded} = \sum_{i=1}^{n} T_{detectori} + \sum_{i=1}^{n} T_{gapi} \tag{6.3}$$

$T_{impeded}$ the time duration of vehicles being impeded

$T_{detectori}$ the duration of detector being occupied by the *i*th impeded vehicle

T_{gapi} the gap between the *i*th impeded vehicle and the leading vehicle

i the *i*th impeded vehicle

n the number of impeded-vehicles

The hourly traffic volume (veh/h) and percentage of time impeding are determined based on the 15-minute-interval observation. The relationship between the PTI and traffic volume is shown in Figure 6.13. For all of the combinations of traffic volume and truck percentage, the PTI increases with the traffic volume. The PTI ranges from 5% to 45%.

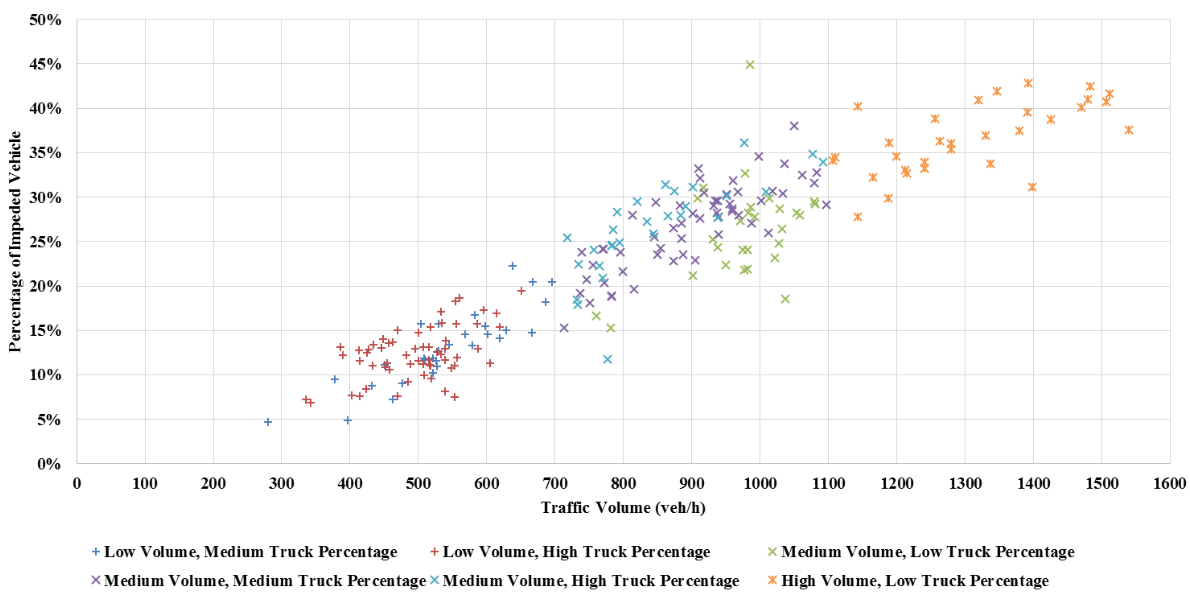


Figure 6.13 Relationship between traffic volume and percentage of time impeding

6.3 Moving Bottlenecks Characteristics

6.3.1 Overview of Moving Bottlenecks Characteristics

This section aims at showing effects of moving bottleneck types on different moving bottleneck characteristics. Three categories of moving bottleneck characteristics were evaluated for traffic flow:

1) Speed-related moving bottleneck characteristics, including the speed of impeded and non-impeded vehicles, the difference in speed of impeded and non-impeded vehicles, and the ratio of impeded vehicle speed to free flow speed;
2) Spatial-related moving bottleneck characteristics, including the number and density of impeded vehicles in moving bottlenecks;
3) Characteristics related to moving bottleneck existence, including moving bottleneck existence time, moving bottleneck existence distance, and moving bottleneck-caused-delay experienced by passenger car with FFS=75 mph.

6.3.2 Speed-Related Moving Bottleneck Characteristics

1. Speed of impeded and non-impeded vehicles (IVS/FFS)

Figure 6.14 shows a box plot of observed speed for ten categories. The first eight are for vehicles classified or belonging to a moving bottleneck (e.g. impeded vehicles). The eight categories correspond to the classification scheme shown in Table 6.2 and are based on the moving bottleneck leader vehicle type and lane position.

Figure 6.15 shows a box plot of passenger car speed, following the same classification as Figure 6.14.

Figure 6.16 shows a box plot of truck speed, following the same classification as Figure 6.14 and Figure 6.15.

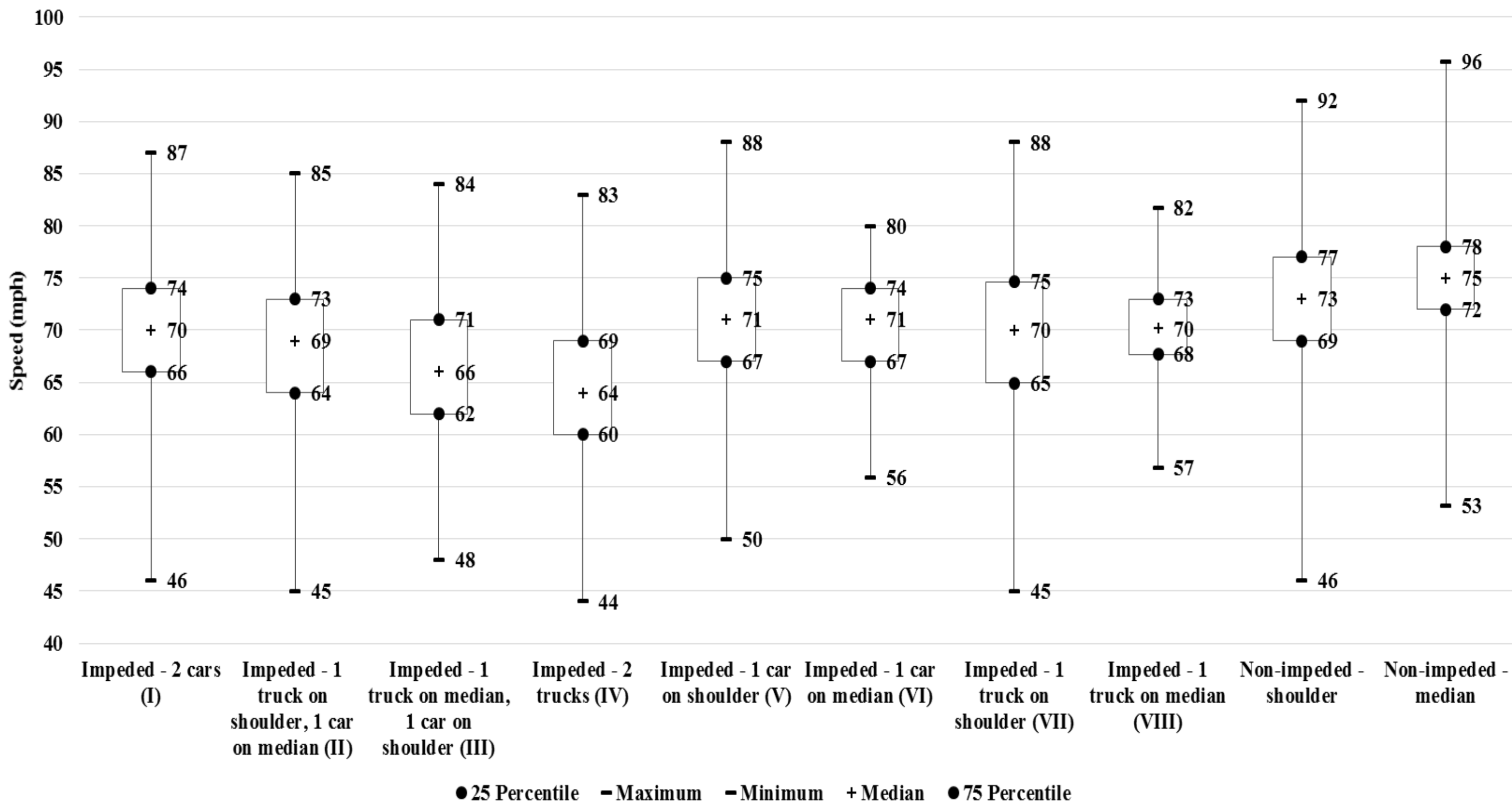


Figure 6.14 Impeded vehicle speed vs non-impeded vehicle speed (for all vehicles)

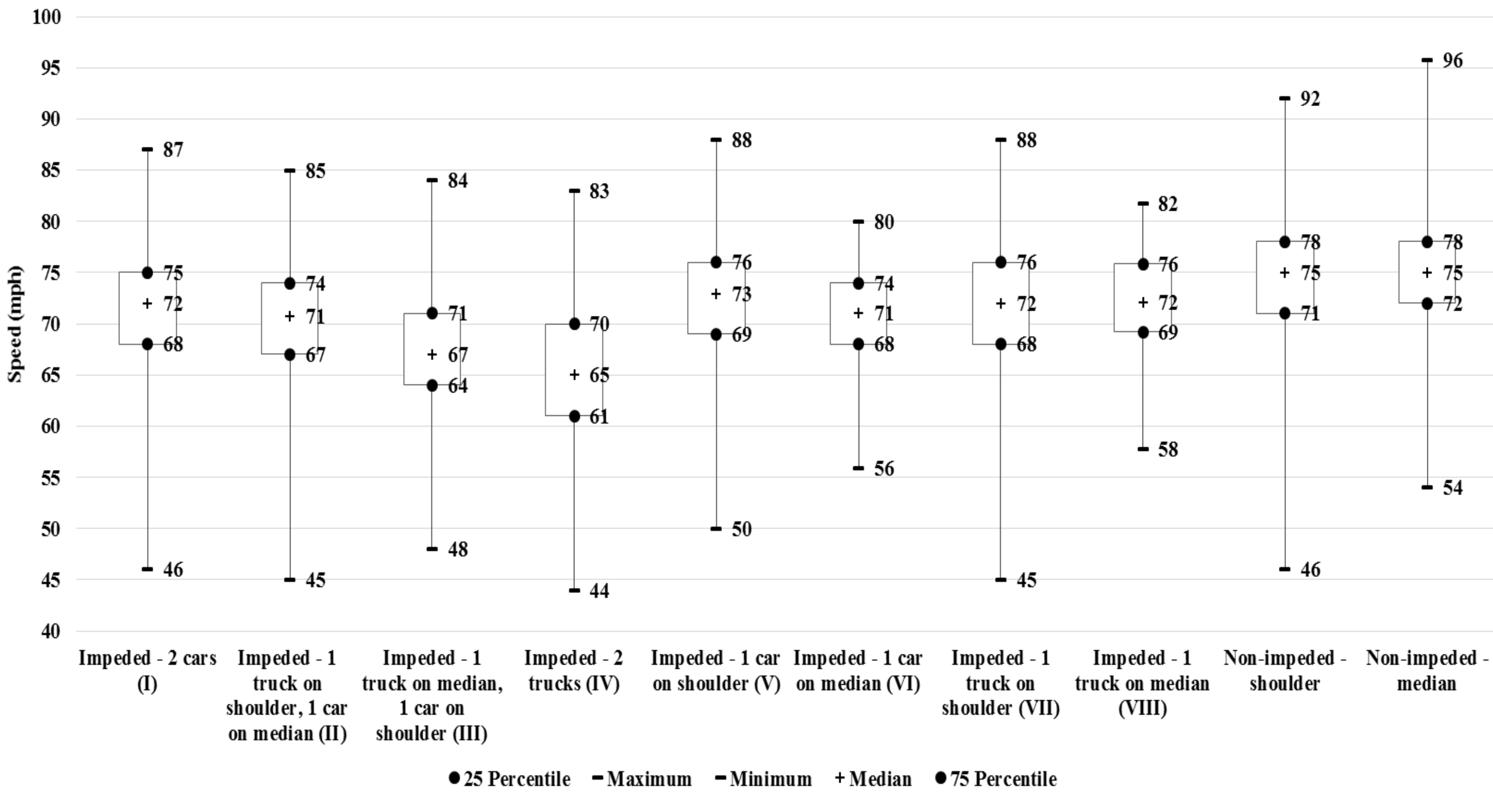


Figure 6.15 Impeded vehicle speed vs non-impeded vehicle speed (for passenger cars)

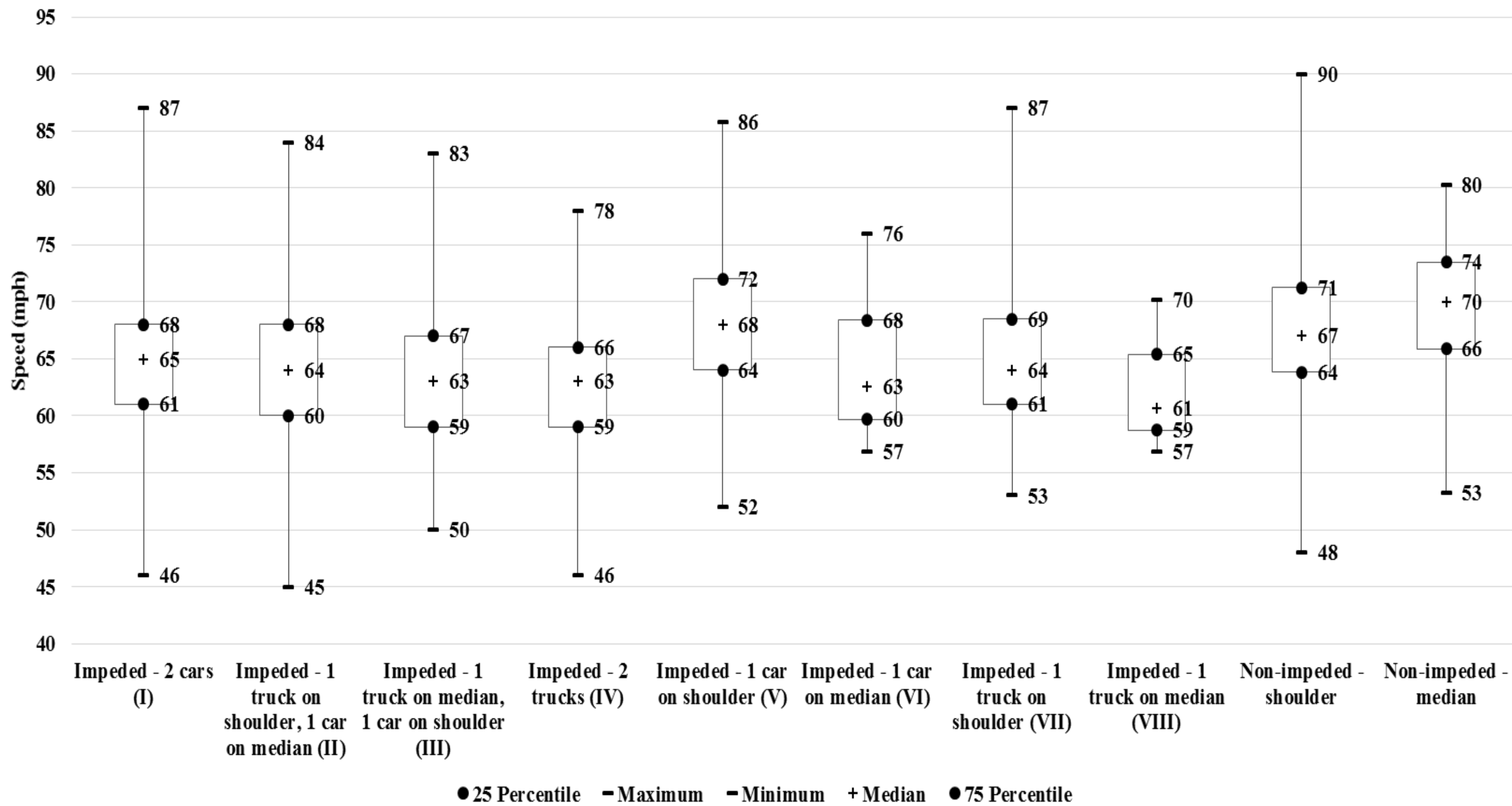


Figure 6.16 Impeded vehicle speed vs non-impeded vehicle speed (for trucks)

These box plots show that:

1) Speeds of impeded vehicles (IVS) are lower than the speeds of the non-impeded vehicles (free flow speed, FFS). The difference is statistically significant at 95% level of confidence. The median of impeded vehicle speed ranges from 64 mph to 71 mph, and the median of non-impeded vehicle speeds range from 73 to 75 mph. The difference in mean speed between impeded and non-impeded passenger cars is higher than for trucks, indicating passenger cars experience more speed reduction than trucks;
2) Speeds of impeded vehicles in two-lane moving bottlenecks are lower than in one-leader moving bottlenecks. The difference is statistically significant at 95% level of confidence. For all vehicles, the median of impeded vehicle speed in two-lane moving bottlenecks ranges from 64 mph to 70 mph, and the median of impeded vehicle speed in one-lane moving bottlenecks ranges from 70 to 71 mph. The difference in mean speed between passenger cars in two-lane and in one-lane moving bottlenecks is higher than that for trucks, indicating passenger cars experience more speed reduction than trucks if caught in two-lane moving bottlenecks;
3) Speeds of impeded vehicles in moving bottlenecks with two trucks as leaders are lower than in moving bottlenecks with two cars as leaders, which indicates truck-leading-moving bottlenecks may cause more delay compared with car-leading-moving bottlenecks. The difference is statistically significant at 95% level of confidence.

The results of a t-test for these comparisons are shown in

Table **6.5**.

Table 6.5 Results of T-test for comparisons for speed of impeded vehicles

Comparison	Difference (mph)	Value of T-test	P-value
Speed for impeded v.s. non-impeded vehicle	-5.1	-39.7 (Difference < 0)	< 0.05
Speed for impeded vehicle in two-lane moving bottleneck v.s. one-lane moving bottleneck	-3.5	-12.61 (Difference < 0)	< 0.05
Speed for impeded vehicle: Moving bottleneck led by two trucks versus moving bottleneck led by two cars	-6.4	-26.30 (difference < 0)	<0.05

2. Difference in speed of impeded and non-impeded vehicles (DiffIVS_FFS)

Equation 6.4 indicates how much lower the speed of impeded vehicles is compared to the mean speed of non-impeded vehicles in corresponding 15-minute intervals.

$$DiffIVS\_FFS = IVS - FFS \tag{6.4}$$

$DiffIVS\_FFS$	difference in speed of impeded vehicle and non-impeded vehicles
IVS	speed of impeded vehicle
FFS	mean speed of non-impeded vehicles in corresponding 15-minute interval

A low value indicates a high amount of speed reduction. The difference between speed of impeded vehicles and the mean speed of non-impeded vehicles in corresponding

15-minute intervals for vehicles in each moving bottleneck type is analyzed. The box plot for distributions of DiffIVS_FFS for each moving bottleneck type is shown in Figure 6.17. The figure shows that impeded vehicles in two-lane moving bottlenecks experience more speed reduction than impeded vehicles in one-lane moving bottlenecks, and impeded vehicles in moving bottlenecks led by two trucks experience the highest speed reduction.

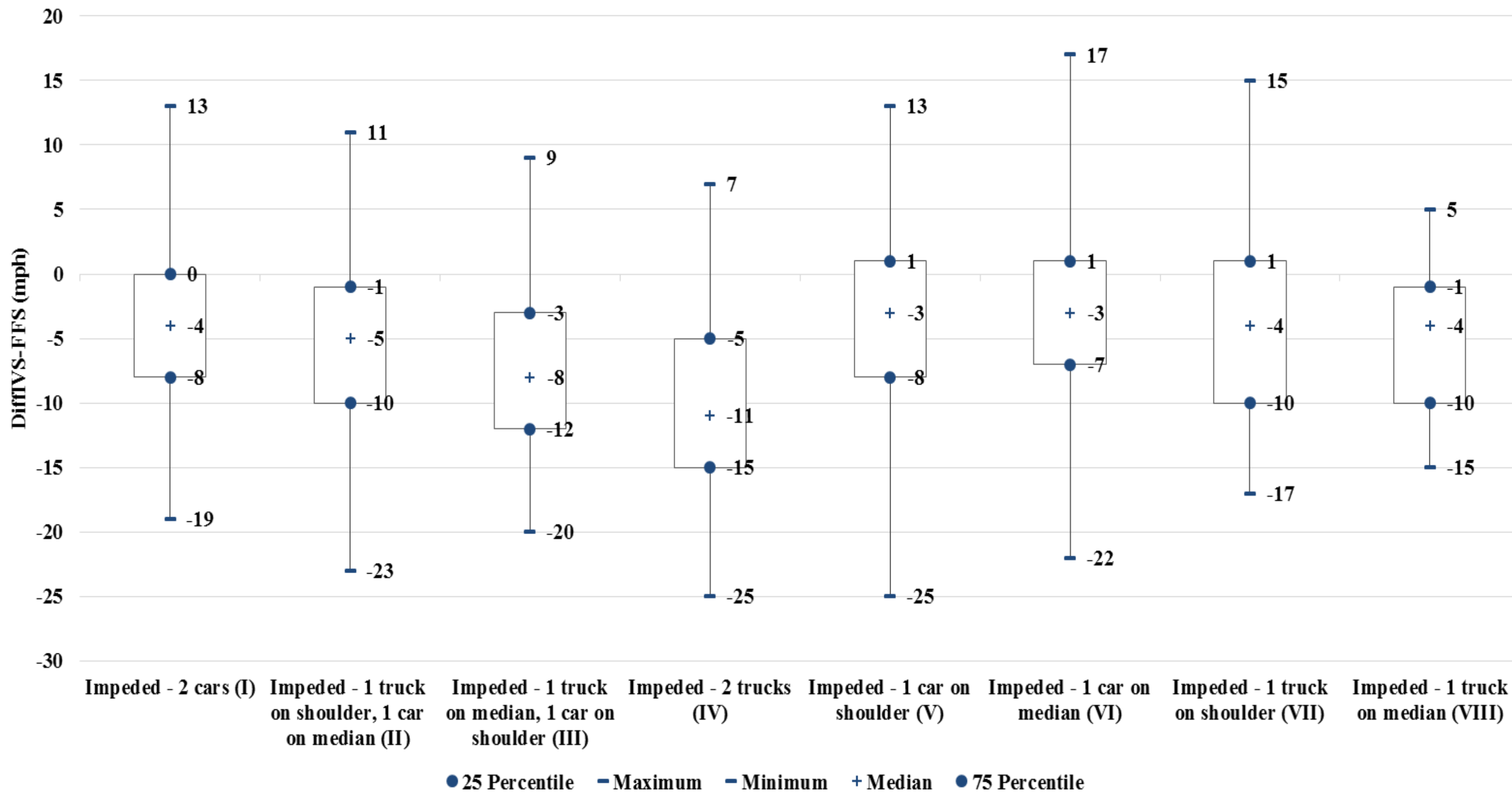


Figure 6.17 Distribution of DiffIVS-FFS for each moving bottleneck type

3. Ratio of impeded vehicle speed to non-impeded vehicle speed (IVS_FFS%)

Equation 6.5 measures percentage of speed reduction for impeded vehicles compared with the mean speed of non-impeded vehicles in corresponding 15-minute intervals.

$$IVS\_FFS\% = \frac{IVS}{FFS} \times 100\% \tag{6.5}$$

$IVS\_FFS\%$	ratio of impeded vehicle speed to the mean speed of non-impeded vehicles
IVS	speed of impeded vehicle
FFS	mean speed of non-impeded vehicles in corresponding 15-minute interval

Similar to DiffIVS_FFS, the low value indicates a high percentage of speed reduction. The box plot for distributions of IVS_FFS% for each moving bottleneck type is shown in Figure 6.18. The figure shows that IVS_FFS% for impeded vehicles in two-leader moving bottlenecks is lower than in one-leader moving bottlenecks, and impeded vehicles in moving bottlenecks led by two trucks experience the highest percentage of speed reduction.

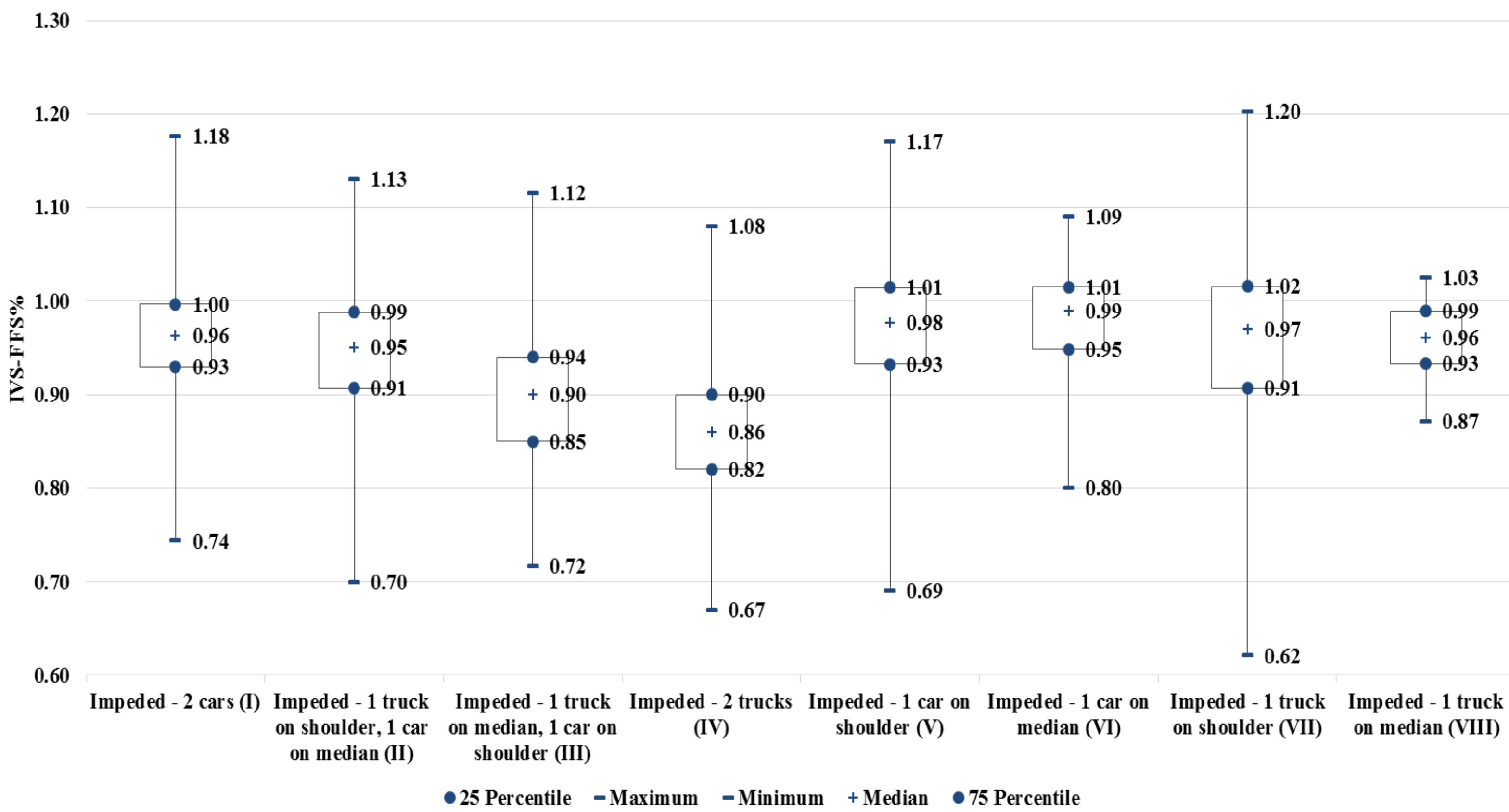


Figure 6.18 Distribution of IVS-FFS% for each moving bottleneck type

6.3.3 Spatial-Related Moving Bottlenecks Characteristics

The number of impeded vehicles (NIV) measures the length of moving bottlenecks. The high value in the number of impeded vehicles indicates the moving bottleneck length is high. The density of impeded vehicles (DIV) measures the degree of congestion for impeded vehicles in moving bottlenecks. A high value in the density of impeded vehicles represents a high degree of congestion and low level of service. Density of impeded vehicles is defined as the number of impeded vehicles in a directional traffic flow over 1 mile per lane, and estimated by the following equation:

$$DIV = \frac{NIV}{\overline{IVS} \times T_{impeded} \times N} \tag{6.6}$$

DIV	density of impeded vehicles
NIV	number of impeded vehicles
$\overline{IVS}$	average speed of impeded vehicles
$T_{impeded}$	Time duration of vehicles being impeded in one moving bottleneck
N	number of lanes

The time duration of vehicles being impeded in one moving bottleneck is the sum of the duration of detectors being occupied by impeded vehicles in one moving bottleneck and the gaps between impeded vehicles and their leading vehicles in one moving bottleneck. Equation 6.3 is used here.

The average number and density of impeded vehicles for different moving bottleneck types are shown in Figure 6.19 and Figure 6.20 respectively.

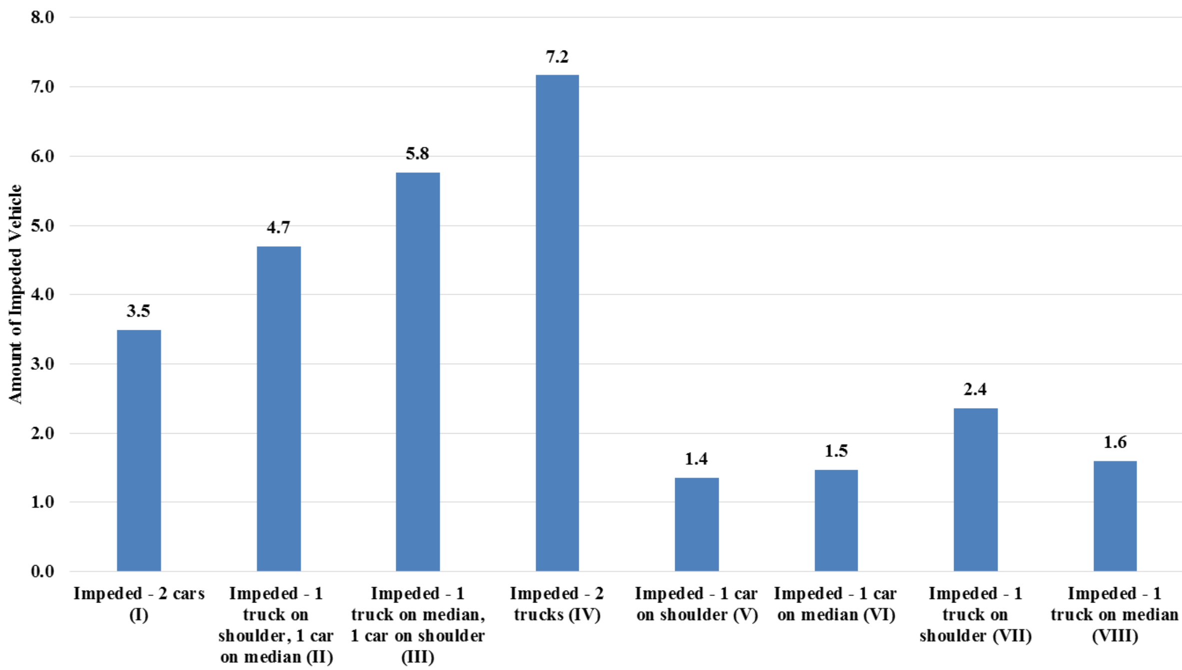


Figure 6.19 Average number of impeded vehicles for different moving-bottleneck types

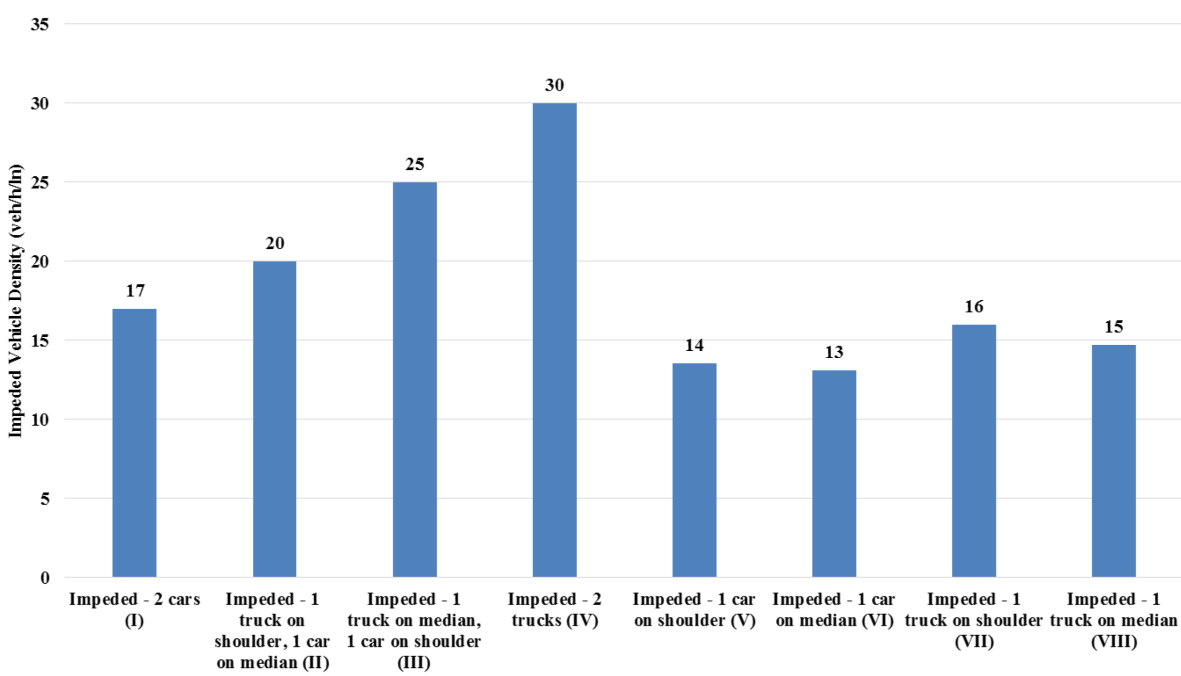


Figure 6.20 Average density of impeded vehicles for different moving-bottleneck types

The figure shows that the average number of impeded vehicles for two-lane moving bottlenecks is higher than one-lane moving bottlenecks with the highest value occurring in Type IV moving bottlenecks (7.2), indicating the Type IV moving bottleneck has the highest length. The average density of impeded vehicles for two-lane moving bottlenecks is higher than one-lane moving bottlenecks with the highest value occurring in Type IV moving bottlenecks (30 veh/mile/ln), indicating that vehicles experience the most severe congestion if impeded in moving bottlenecks led by two trucks.

6.3.4 Characteristics Related to Moving Bottlenecks Existence

In this section, the moving bottleneck existence time, moving bottleneck existence distance, and moving bottleneck-caused-delay experienced by passenger cars with free flow speed 75 mph are analyzed. Moving bottleneck existence time and distance measures how long and for what distance vehicles are actually impeded in a moving bottleneck and affected by it. This analysis considers the formation and dispersion of moving bottlenecks, not just the length and impeding time of moving bottleneck shown in video recordings from fixed location. The existence time and distance of two-lane moving bottlenecks can be estimated if there is a difference in speed between leader vehicles.

In this research, it is assumed two-lane moving bottlenecks are formed due to faster impeded vehicles tending to overtake its leading slower impeded vehicles on the same lane; then, the faster impeded vehicles change lanes during overtaking and a two-lane moving bottleneck is formed. A two-lane moving bottleneck can be separated into two parts. The part tending to overtake its leading vehicles with faster average speed is a faster platoon, led by faster leaders and followed by faster impeded vehicles. Similarly, the part being overtaken with a slower average speed is a slower platoon, led by slower leaders and followed by slower impeded vehicles. The absolute value of difference in average speed between faster and slower platoon is defined as “moving bottleneck speed difference”. Two-lane moving bottlenecks start when the faster leader starts overtaking the last slower impeded vehicle, and ends when the last faster impeded vehicle ends overtaking the slower leader. At the beginning of two-lane moving bottleneck existence, all vehicles in faster platoons fall behind all vehicles in slower platoons; at the end of two-lane moving bottleneck existence, all vehicles in faster platoons pass all vehicles in slower platoons.

Moving bottleneck existence time (t) is defined as the time duration between the beginning and end of moving bottleneck existence. The moving bottleneck existence distance (s) is defined as the travel distance for faster leader during moving bottleneck existence time. The moving bottleneck delay (pd) is defined as delay experienced by impeded vehicles in the moving bottleneck, which is equal to the difference between actual travel time when vehicles are impeded in moving bottlenecks and the travel time when vehicles travel with free flow speed. Illustrations for these definitions are shown in Figure 6.21.

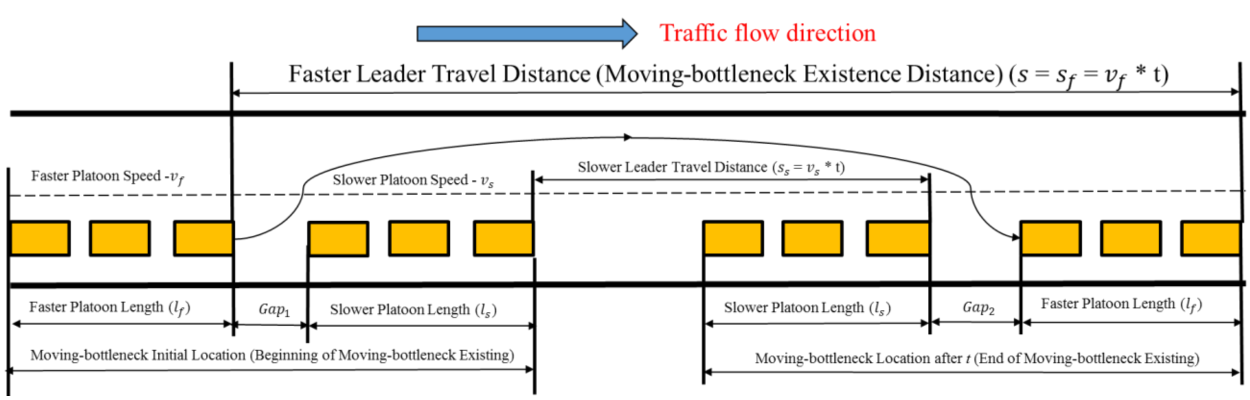


Figure 6.21 Definition of moving bottleneck existence distance and existence time

According to this figure, the moving bottleneck existence time, distance, and moving bottleneck delay is determined by the following equations:

$$\begin{cases} t = \left(l_f + l_s + Gap_1 + Gap_2\right)/\left(v_f - v_s\right) \\ s = v_f * t \\ pd = s * (1/v - 1/FFS) \end{cases} \tag{6.7}$$

t	moving bottleneck existence time
s	moving bottleneck existence distance
pd	moving bottleneck delay experienced by individual impeded vehicle
l_f	length of faster moving bottleneck
l_s	length of slower moving bottleneck
v_f	average speed of vehicles in faster moving bottleneck
v_s	average speed of vehicles in slower moving bottleneck
Gap_1	safety distance between rear bumper of last slower impeded vehicle and front bumper of faster leader
Gap_2	safety distance between rear bumper of last faster impeded vehicle and front bumper of slower leader
v	speed of individual impeded vehicle
FFS	free flow speed of individual impeded vehicle

Moving bottleneck existence time and distance vary with moving bottleneck speed difference and length of faster and slower platoons. Effects of moving bottleneck speed difference are theoretically analyzed under two assumed conditions, with only one car in faster moving bottleneck and slower moving bottleneck (e.g. Type I moving bottleneck), and only one truck in faster platoons and slower platoons (e.g. Type IV moving bottleneck). The two conditions can also be described as a faster car overtaking a slower car, and a faster truck overtaking a slower truck. Therefore, the time and distance for one car overtaking another car or one truck overtaking another truck represents the moving bottleneck existence time and distance. Other assumptions include

1) Speed of slower car and truck are set at 70 mph and 67 mph, respectively;
2) Speed of faster car and truck are set as one to four mph higher than the slower vehicle;
3) Length of car and truck are set as 8ft and 35ft, respectively;
4) The two gaps are set to 328ft, which is widely used in confirming inter-vehicle distance on freeways.

The delays of impeded passenger cars caused by these two-lane moving bottlenecks are also analyzed. The free flow speed of impeded passenger cars is assumed as 75 mph. Figures for conditions of theoretical analysis are shown in the appendix. The results of theoretical analysis are shown in Table 6.6.

Table 6.6 Theoretical analysis for effects of moving bottleneck speed difference on moving bottleneck existence time, distance and moving bottleneck delay for passenger cars with FFS = 75 mph

Variable	**Moving bottleneck Type**	**Leader Speed Difference (mph)**			
		1	2	3	4
Moving bottleneck Existence Time (s)	Type I	458	228.6	152.4	114.5
	Type IV	495	247.5	165	123.8
Moving bottleneck Existence Distance (mile)	Type I	9.03	4.57	3.09	2.35
	Type IV	9.35	4.74	3.2	2.4
Moving bottleneck delay for passenger cars with 75mph free-flow-speed(s)	Type I	24.1	9.14	6.18	1.52
	Type IV	46.2	19.78	10.97	6.49

The results of moving bottleneck existence time, distance, and moving bottleneck delay for passenger cars with 75 mph free flow speed based on empirical data are shown in Table 6.7, Figure 6.22, and Figure 6.23.

Table 6.7 Empirical analysis for average moving bottleneck existence time, distance and moving bottleneck delay for passenger cars with FFS = 75 mph

Moving bottleneck Type	**Type I**	**Type II**	**Type III**	**Type IV**
Moving bottleneck Existence Time (s)	367	530	777	1104
Moving bottleneck Existence Distance (mile)	7.56	10.01	13.46	16.93
Moving bottleneck delay for passenger cars with 75mph free-flow-speed(s/moving bottleneck)	38	59	90	140

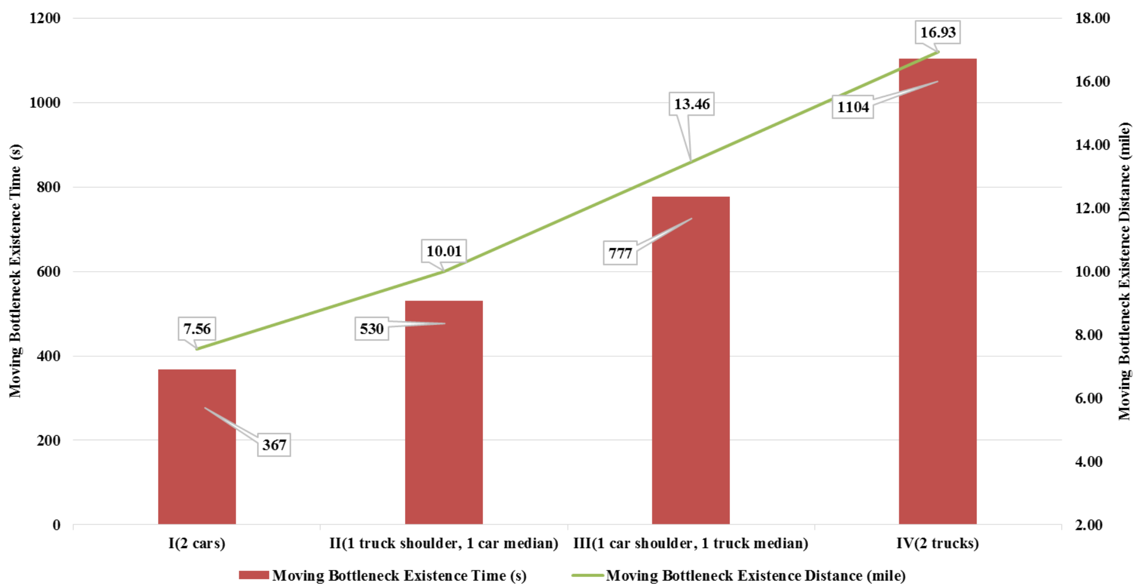


Figure 6.22 Moving-bottleneck existence distance and time for different moving-bottleneck types based on empirical data

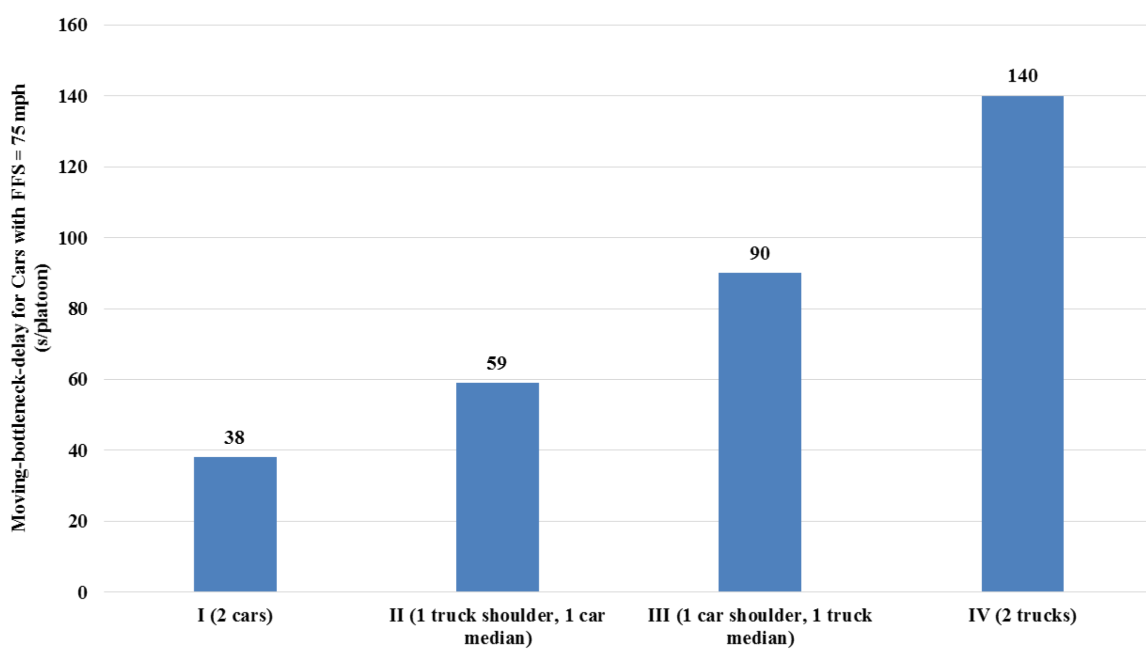


Figure 6.23 Moving-bottleneck delay for passenger cars with FFS = 75 mph based on empirical data

As expected, the theoretical analysis results show that moving bottleneck existence time, distance, and moving bottleneck delay decreases with the increase in moving bottleneck speed difference. Delay caused by Type I moving bottlenecks are 55% lower than Type IV moving bottlenecks, which means moving bottlenecks led by two trucks affect impeded vehicles longer and cause more delays than moving bottlenecks led by two cars. Empirical analysis results show that lowest average existence time, distance, and delay occur in moving bottlenecks led by two cars (Type I moving bottleneck), and are highest for moving bottlenecks led by two trucks (Type IV moving bottleneck). Moving bottlenecks led by a truck in the median lane and a car in the shoulder lane (Type III moving bottleneck) had the longer existence time, distance, and delays than those led by a truck in the shoulder lane and a car in the median lane (Type II moving bottleneck). Overall, if vehicles are impeded in moving bottlenecks led by two trucks, the vehicles will be most severely affected by moving bottlenecks with longest time and distance, and experience the highest moving bottleneck delay.

6.4 Metrics for Localized Congestion

The empirical analyses above show the moving bottlenecks will cause localized congestion. To measure the localized congestion caused by the moving-bottlenecks for a certain four-lane freeway segment, the metric “Density uniformity factor” (DUF) is proposed in this research. Consider a section of freeway with homogeneous geometric characteristics. It may be disaggregated in S sections, or may be analyzed over a series of T time periods. The variables used in the analysis are defined as the following:

i Indicator of time periods, $i = 1, 2, \ldots, T$

T	Total number of time periods
Δt_i	Duration of time period i (second)
j	Indicator of freeway segment, j = 1, 2, …, S
S	Total number of freeway segments
Δs_j	Length of freeway segment j (mile)
$V_{i,j}$	Volume of vehicles that passing the detectors setting at the beginning of the freeway segment j at time periods i (veh)
N_l	Number of lanes
$q_{i,j}$	Flow rate of vehicles that passing the detectors setting at the beginning of the freeway segment j at time periods i (veh/h/ln)
$\bar{v}_{i,j}$	Average speed of vehicles that passing the detectors setting at the beginning of the freeway segment j at time periods i (mph)
$k_{i,j}$	Density at time periods i at freeway segment j (veh/mi/ln)

The flow rate of vehicles entering the freeway segment j at time periods i is calculated using the following equation:

$$q_{i,j} = \frac{V_{i,j}/N_l}{\Delta t_i/3600} = \frac{3600 * V_{i,j}}{N_l * \Delta t_i} \tag{6.8}$$

$q_{i,j}$	Flow rate of vehicles passing the detectors set at the beginning of the freeway segment j at time periods i (veh/h/ln)
$V_{i,j}$	Volume of vehicles passing the detectors set at the beginning of the freeway segment j at time periods i (veh)
N_l	Number of lanes

Δt_i Duration of time period i (second)

The density at time periods i at freeway segment j is calculated using Equation 6.9 (3). This equation is derived from the definition of the density, defined as the number of vehicles per mile per lane and calculated by dividing the number of vehicles per lane at segment j by the length of segment j (Δs_j).

$$k_{i,j} = \frac{\frac{\bar{N}_{i,j}}{N_l}}{\Delta s_j} = \frac{\frac{V_{i,j}}{N_l * L_{i,j}} * \Delta s_j}{\Delta s_j} = \frac{\frac{V_{i,j}}{N_l * \bar{v}_{i,j} * \frac{\Delta t_i}{3600}} * \Delta s_j}{\Delta s_j} \tag{6.9}$$

$$= \frac{3600 * V_{i,j}}{N_l * \Delta t_i * \bar{v}_{i,j}} = \frac{q_{i,j}}{\bar{v}_{i,j}}$$

$k_{i,j}$ Density at time periods i at freeway segment j (veh/mi/ln)

$\bar{N}_{i,j}$ Average number of vehicles at time periods i at freeway segment j (veh)

N_l Number of lanes

Δs_j Length of freeway segment j (mile)

$V_{i,j}$ Volume of vehicles passing the detectors set at the beginning of the freeway segment j at time periods i (veh)

$L_{i,j}$ Travel length of the first vehicle entering the freeway segment j at time periods i during the duration of time period Δt_i (mile)

$\bar{v}_{i,j}$	Average speed of vehicles passing the detectors set at the beginning of the freeway segment j at time periods i (mph)
Δt_i	Duration of time period i (second)
$q_{i,j}$	Flow rate of vehicles passing the detectors set at the beginning of the freeway segment j at time periods i (veh/h/ln)

The variables are illustrated using an example in Figure 6.24.

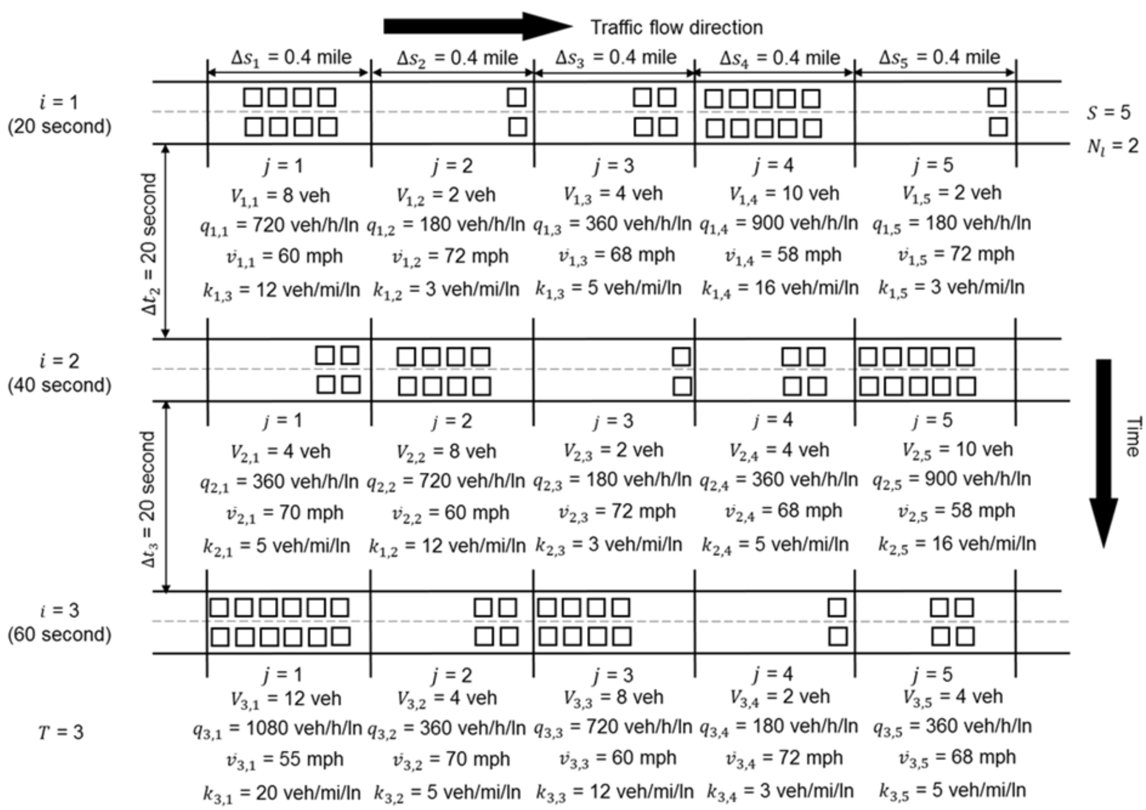


Figure 6.24 Variables for localized congestion analysis

The density uniformity factor (DUF) over several consecutive time periods and several consecutive freeway segments is defined by the following equation:

$$DUF = \frac{\sum_{i=1}^{T} \sum_{j=1}^{S} k_{i,j}}{T * S * \max_{\substack{i=1\ to\ T \\ j=1\ to\ S}} \{k_{i,j}\}} \tag{6.10}$$

DUF	Density uniformity factor over T consecutive time periods and S consecutive freeway segments, range between 0 and 1
i	Indicator of time periods, i = 1, 2, …, T
T	Total number of time periods
j	Indicator of freeway segment, j = 1, 2, …, S
S	Total number of freeway segments
$k_{i,j}$	Density at time periods i at freeway segment j (veh/mi/ln)

The DUF can also be calculated over several consecutive time periods given a freeway segment j, or calculated over several consecutive freeway segments given a time period i, defined by the following equations:

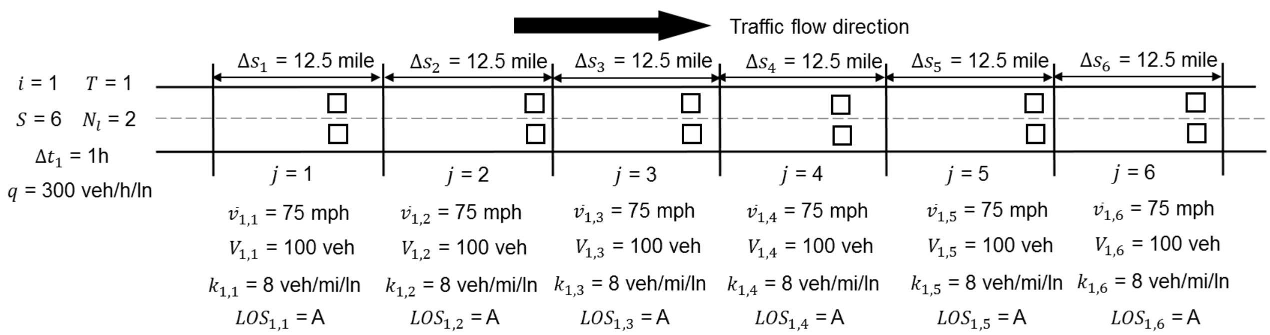
$$DUF_j = \frac{\sum_{i=1}^{T} k_{i,j}}{T * \max_{i=1\ to\ T} \{k_{i,j}\}} \tag{6.11}$$

$$DUF_i = \frac{\sum_{j=1}^{S} k_{i,j}}{S * \max_{j=1\ to\ S} \{k_{i,j}\}} \tag{6.12}$$

DUF_j	Density uniformity factor over T consecutive time periods at freeway segment j, range between 0 and 1
DUF_i	Density uniformity factor over S consecutive freeway segment at time period i, range between 0 and 1
i	Indicator of time periods, i = 1, 2, …, T
T	Total number of time periods
j	Indicator of freeway segment, j = 1, 2, …, S
S	Total number of freeway segments
$k_{i,j}$	Density at time periods i at freeway segment j (veh/mi/ln)

The DUF is proposed for measuring the localized congestion and the consistency of density over several consecutive time periods and several consecutive freeway segments. The value of DUF ranges between 0 and 1. The equation shows that, when there is low difference in density for each time period and freeway segment, the value of DUF tends to increase. This is because the density in each time period and freeway segment is close to the maximum density over all consecutive time periods and freeway segments. The high value of DUF means, over several consecutive time periods and freeway segments, a) the density tends to be consistent, b) the average density for these time periods and freeway segments tends to be equal to the density of each time period and freeway segments, and c) there is no obvious localized congestion.

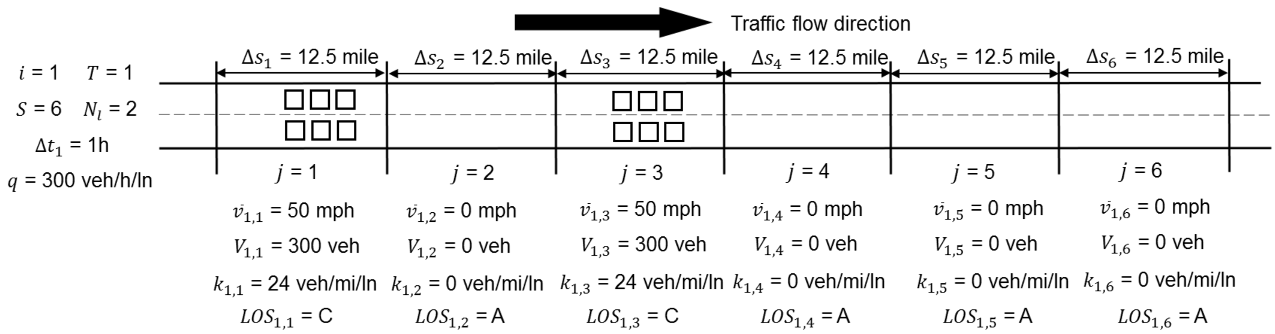
Contrary, when there are differences in density over several consecutive time periods and freeway segments, the value of DUF tends to decrease. This is because the density in some time periods at some freeway segments may be much lower than the maximum density over all consecutive time periods and freeway segments. The low value of DUF means, over several consecutive time periods and freeway segments, a) the density tends to be inconsistent, b) the average density for these time periods and freeway segments tends to be unequal to the density of each time period and freeway segments, and c) there is obvious localized congestion. The meaning of DUF is explained by two examples in Figure 6.25 and Figure 6.26.



$\bar{k}_{i,j}$ = 8 (veh/mi/ln) – LOS A, DUF_i = 1.00.

*The example is calculated for one hour for six segments; one square represents 50 vehicles.

Figure 6.25 Explanation of the meaning of DUF - an example of high DUF value



$\bar{k}_{i,j}$ = 8 (veh/mi/ln) – LOS A, DUF_i = 0.33.

*The example is calculated for one hour for six segments; one square represents 50 vehicles.

Figure 6.26 Explanation of the meaning of DUF - an example of low DUF value

As explained by Figure 6.27, when the DUF tends to 1, the density over several consecutive time periods and freeway segments can be considered as “uniform”, as the condition described in HCM 2010 Chapter 11 (Basic freeway segment analysis) and HCM 2016 Chapter 12 (Basic freeway segment analysis), which implied “all analyses are applied to segments with uniform characteristics”. Uniform characteristics means the segments must have the same geometric and traffic characteristics (3). When the DUF tends to 0, as explained by Figure 6.27, the density over several consecutive time intervals can be considered as “non-uniform”, as the condition empirically observed in western rural U.S. Therefore, in this research it is hypothesized that the HCM recommended PCE values are applicable to the “uniform” density conditions but may not be applicable to the “non-uniform” density conditions. New PCEs may need to be estimated for the “non-uniform” density conditions as empirically observed in western rural U.S. The DUF-based discussions on applicable conditions of HCM PCEs or new estimated PCEs will be shown in the next two chapters.

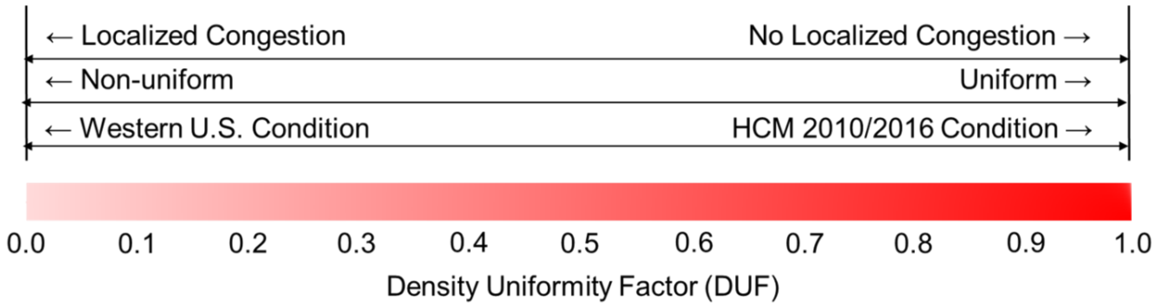


Figure 6.27 Definition of density uniformity factor (DUF)

6.5 Summary

Rural freeways in the western rural U.S. have experienced truck percentages much greater than the assumed 25 percent maximum listed in the HCM 2016. In addition, many heavy trucks have speed limiters installed, at the behest of their fleet owners, in order to improve fuel efficiency. The combination of these two factors results in the formation of moving bottlenecks, and these moving bottlenecks may adversely affect traffic flow and capacity in a manner that is not included in standard HCM 2016 techniques.

Because of the above changes, this dissertation proposed a new moving-bottleneck identification methodology for four-lane freeways. It is based on critical headways that vary according to vehicle type. The critical headways range from 3.0 s to 8.0 s and are a function of whether a leading or lagging headway is required.

Using the new moving-bottleneck identification methodology and empirical data from western Nebraska, an analysis of moving-bottleneck formation was conducted. The analysis showed that

1) 51 percent of vehicles impeded in moving bottlenecks, of which speeds are approximately 6.0 percent lower than free-flow vehicles;

2) The number and percentage of vehicles impeded in moving bottlenecks increase with traffic volume.

Identified moving bottlenecks were classified into eight groups based on their leader types. A number of moving-bottleneck characteristic metrics were identified and show that, on average, vehicles impeded by two-truck-leading moving bottlenecks experience the highest speed reduction, degree of congestion, and moving-bottleneck delay. In addition the two-truck-leading moving bottlenecks have the longest moving-bottleneck length, existence time, and distance. Overall, this study demonstrates that vehicles impeded in two-truck-leading moving bottlenecks at a high volume and truck percentage condition are most severely affected by moving bottlenecks.

Additionally, this dissertation proposed the metric “Density uniformity factor” (DUF) to measure the localized congestion caused by the moving-bottlenecks for a certain four-lane freeway segment. The DUF is defined as the ratio of the sum of density over all consecutive time periods and freeway segments, to the maximum of density over all consecutive time periods and freeway segments. The DUF can be used for describing whether the traffic flows are “uniform” or “non-uniform”. The value of DUF ranges between 0 and 1. The high value of DUF means the density tends to be uniform and there is no obvious localized congestion, and verse visa.

CHAPTER 7 CALCULATING PCES BASED ON EQUAL-DENSITY METHOD

7.1 Simulation Model Development for Equal-Density Method

7.1.1 Description of Simulation Model

In this dissertation, the simulation model for equal-density PCE (represented by "ED_PCE") calculation is developed by CORSIM 6.3 and VISSIM 9.0. The reasons for using CORSIM are:

1) It uses FRESIM as the microscopic simulation model representing traffic on highways and freeways;
2) HCM 2010 PCEs are calculated based on data simulated by FRESIM.

The output data, including traffic volume and average speed, are aggregated by specific time intervals. One important factor to note is that CORSIM only allows vehicles to follow a single free-flow speed. This was not an issue for the HCM 2010 PCE estimation process, as it assumed that all vehicle types travel with the same average free-flow speed (2). However, this is not true of I-80 in western Nebraska, and it is hypothesized that this is not true for much of the rural freeway sections in the western rural U.S. because of the wide spread use of speed limiters (81). VISSIM allows different vehicle classes to follow different free-flow speed distributions and is therefore better suited to modeling operational conditions on western rural U.S. freeways. The output data include detection of entering and exiting time, ordinal number, class, length, and speed for each vehicle (63).

Because one of the most critical traffic characteristics on western rural U.S. freeway is large numbers of moving bottlenecks observed under high truck percentage conditions, it is necessary to validate and calibrate the model for these conditions. The settings of geometric, vehicle input, and traffic flow characteristics parameters for the simulation model in CORSIM 6.3 and VISSIM 9.0 are summarized in Table 7.1 and

Table **7.2**.

Table 7.1 Geometric, vehicles, and traffic characteristics for CORSIM 6.3 simulation model

Item	Parameters	Values
Geometric	Freeway type	Mainline
	Free flow speed	70 mph
	Super elevation	0%
	Pavement	Dry Asphalt
	Radius	0 ft
	Grade	0%
	Startup delay	1.0s
	Car-following sensitivity multiplier	100%
	Number of through lanes	2
	Auxiliary lanes	None
	First/Second barrier lane	None
	Direction of curvature	Straight
	Trucks biased/restricted status	Biased to a set of lanes
	Trucks biased/restricted to	Biased to rightmost 1 through lane
	Number of HOV lanes	0
	Incidents	None
	Detector type	Doppler radar
Vehicles	Vehicle length	Passenger car: 14 ft – 16 ft Single-unit truck: 30 ft – 45 ft Heavy truck: 50 ft – 95 ft
	Average occupancy	Passenger car: 1.00 Single-unit/heavy truck: 1.00
	Headway factor	Passenger car: 100% Single-unit/heavy truck: 120%
	Jerk value	Passenger car: 7.0 ft/s^3 Single-unit/heavy truck: 7.0 ft/s^3
	Maximum deceleration (Emergency)	Passenger car: 15.0 ft/s^2 Single-unit/heavy truck: 18.0 ft/s^2
	Maximum deceleration (Non-emergency)	Passenger car: 8.0 ft/s^2 Single-unit/heavy truck: 5.6 ft/s^2
	Vehicle entry headway	Normal distribution
Traffic flow characteristics	Traffic volume	500 to 1500 veh/h/ln with 200 veh/h/ln interval
	Truck percentage	0% to 90% with 5% as interval
	Desired speed	Single free-flow speed 70 mph

Table 7.2 Geometric, vehicles, and traffic characteristics for VISSIM 9.0 simulation model

Item	Parameters	Values
Geometric	Number of lanes	2
	Behavior type	3 – Freeway (Slow lane rules)
	Display type	1- Road grey
	Level	1 - Base
	Lane width	12 ft
	Blocked vehicle class	None
	No lane change left	None
	No lane change right	None
	Grade	0%
	Detector type	Point detector
	Lateral - desired position at free flow	Middle of lane
Vehicles	Vehicle length	Passenger car: 14 ft – 16 ft Single-unit truck: 30 ft – 45 ft Heavy truck: 50 ft – 95 ft
	Weight-to-horsepower	Passenger car: 40 lbs/hp – 60 lbs/hp Single-unit truck: 125 lbs/hp – 150 lbs/hp Heavy truck: 125 lbs/hp – 150 lbs/hp
	Maximum acceleration	Passenger car: 11.5 ft/s^2 Single-unit/heavy truck: 4.7 ft/s^2
	Desired acceleration	Passenger car: 11.5 ft/s^2 Single-unit/heavy truck: 4.7 ft/s^2
	Maximum deceleration	Passenger car: 8.0 ft/s^2 Single-unit/heavy truck: 5.6 ft/s^2
	Desired deceleration	Passenger car: 8.0 ft/s^2 Single-unit/heavy truck: 5.6 ft/s^2
	Occupancy	Passenger car: 1.00 Single-unit/heavy truck: 1.00
Traffic flow characteristics	Traffic volume	500 to 1500 veh/h/ln with 200 veh/h/ln as interval
	Truck percentage	0% to 90% with 5% as interval
	Desired speed	1) Single speed distribution for all vehicle types with average 70 mph (Figure 7.3) 2) Different speed distributions for different vehicle types with different average speed (Figure 7.4)

For geometrics information, in either CORSIM 6.3 or VISSIM 9.0, the simulation network is designed as a 4-link grid network, as shown in Figure 7.1 and Figure 7.2. Two of the links are 3.28 miles in length, and two are 2.63 miles in length. Thus, the road network is a ring where vehicles travel in a clockwise direction. This set-up is chosen for display purposes only because the network essentially acts as a linear “pipe” where vehicles enter at one end and exit at the other. Each link is one way and has two 12-feet lanes, zero gradient, and “freeway” behavior type. All lanes on the four links are open for all vehicle types, and there is no lane changing restrictions for any vehicle types on all lanes. Four data collection points are chosen and set at equal distance on the network. At each data collection point, one point detector is set for each lane. Eight data detectors are used in total, where information at that point across time can be obtained.

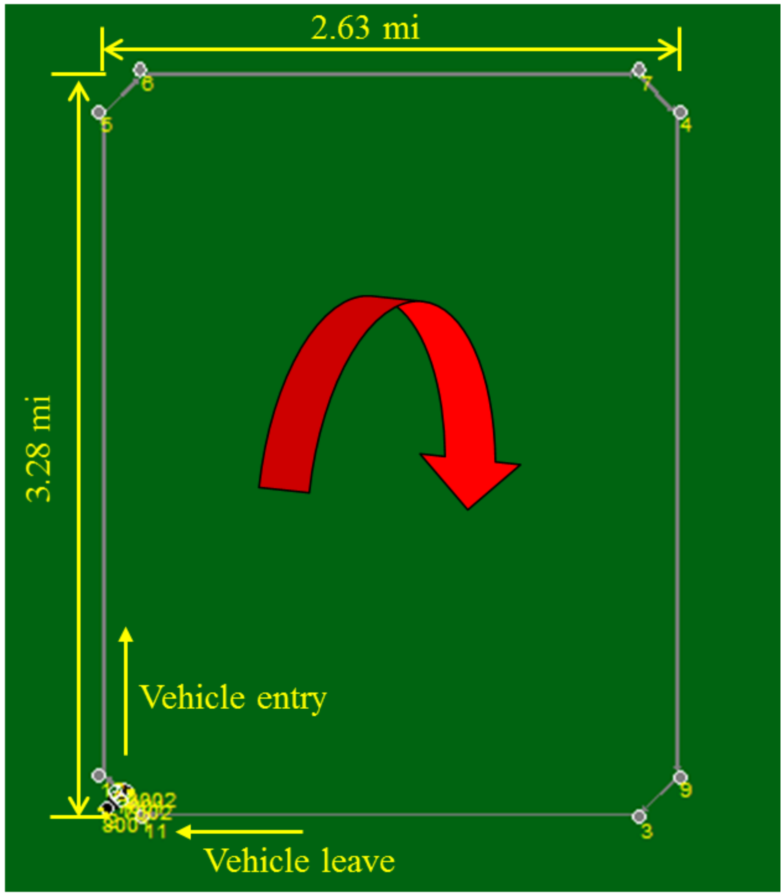


Figure 7.1 Simulation model developed in CORSIM 6.3 for equal-density method

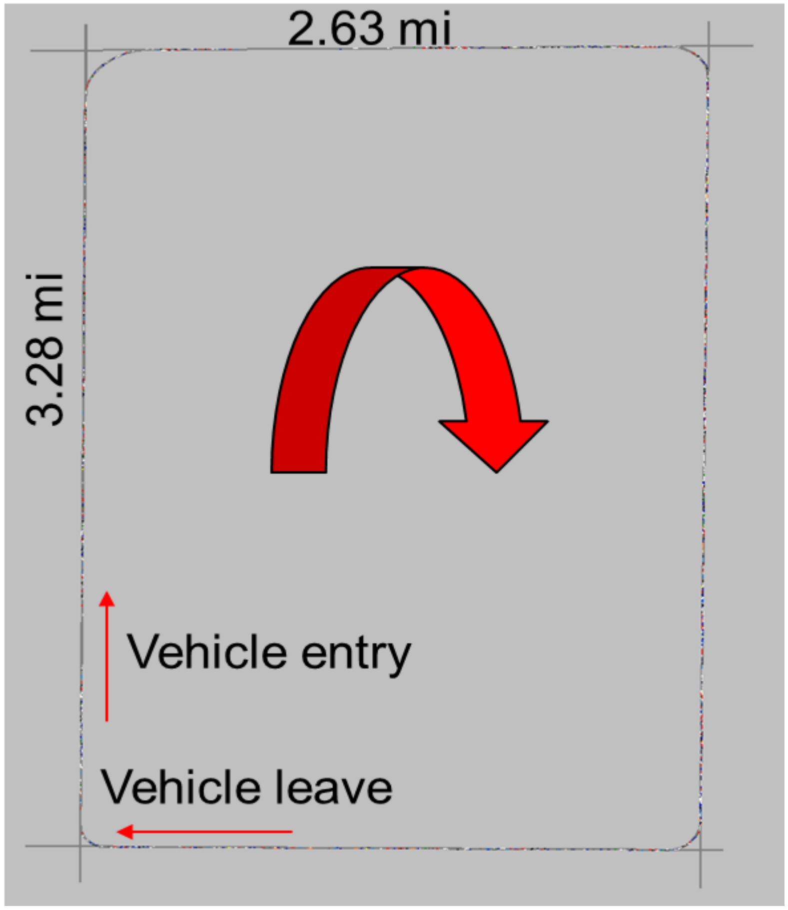


Figure 7.2 Simulation model developed in VISSIM 9.0 for equal-density method

For vehicle input information, the input vehicles are grouped into three classes: passenger cars, single-unit trucks, and heavy trucks. Here, the single-unit trucks are modeled as trucks without trailers. Single unit trucks are set to 10 percent of trucks based on the empirical analysis. Heavy trucks are modeled as trucks with at least one trailer and 3 axles. The weight-to-horsepower ratio for trucks is set between 125 lbs/hp and 150 lbs/hp (2). The trucks maximum acceleration and deceleration curves are derived from a truck performance model described in NCHRP report 505 (94). The single-unit and heavy trucks are modeled separately because the mean speed of single unit trucks are found to be different than passenger cars and heavy trucks, and this difference is found to be statistically significant using a standard t-test at the 95 percent level of confidence (95).

For traffic flow characteristics information, in order to create the density volume curves, a number of different combinations of volume and truck percentage are simulated. Six levels of traffic volume are simulated beginning at 500 veh/h/ln and increasing to 1500 veh/h/ln in 200 veh/h/ln increments. Nine levels of truck percentage are simulated beginning at 5 percent, and increasing to 85 percent at 10 percent increments. There are twenty runs for each combination of traffic volume and truck percentage, and each run has a different random number seed.

There are three scenarios needing to be simulated:

1) Using CORSIM 6.3 with a single free-flow speed 70 mph for all vehicle types to replicate the HCM 2010 results;
2) Using VISSIM 9.0 with a single free-flow speed distribution with average 70 mph for all vehicle types, as shown in Figure 7.3, to see if similar ED_PCE values were obtained;
3) Using VISSIM 9.0 with empirical free-flow speed distributions with different average speeds among vehicle types, as shown in Figure 7.4, to see the impacts of moving bottlenecks.

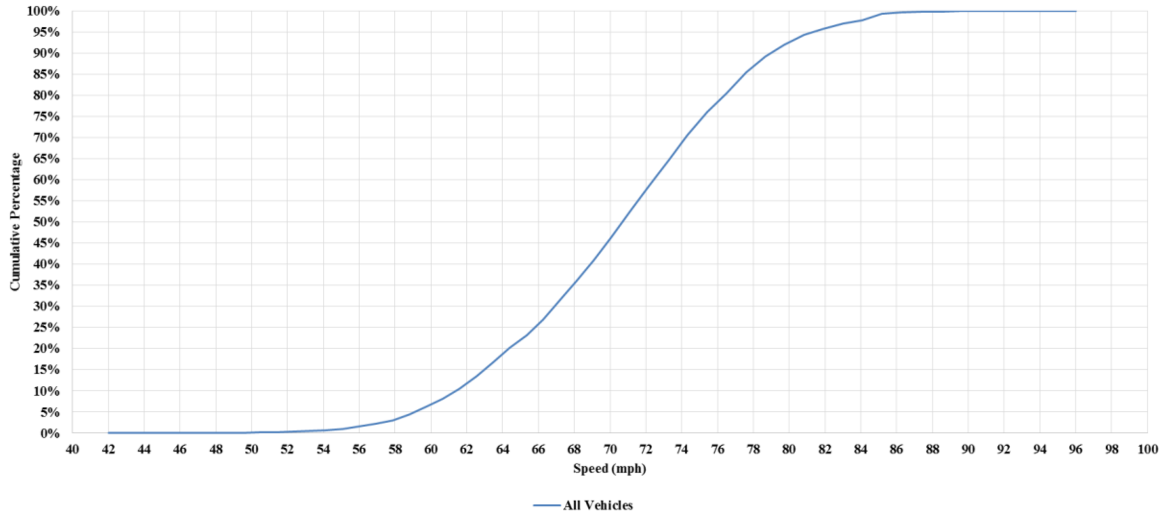


Figure 7.3 Single free-flow speed distributions for all vehicle types in VISSIM 9.0 for equal-density method

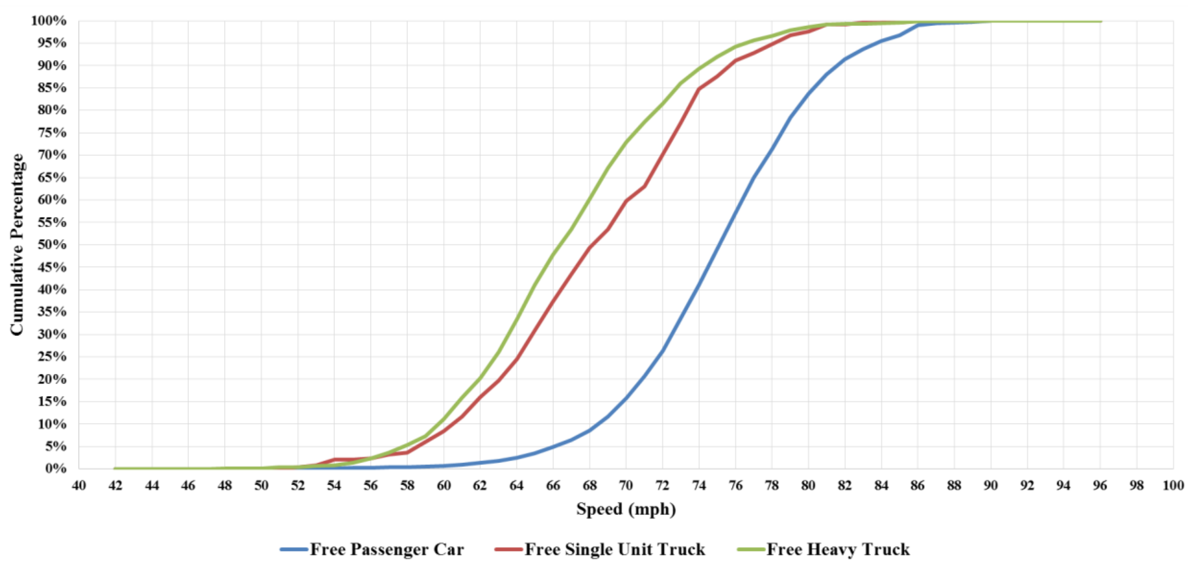


Figure 7.4 Empirical free-flow speed distributions for different vehicle types in VISSIM 9.0 for equal-density method

7.1.2 Simulation Model Calibration

In this research, the parameters in CORSIM 6.3 and VISSIM 9.0 are calibrated by Genetic Algorithm (GA). There are six steps in the calibration process (96):

Step 1 is population initialization. This step determines:

1) Parameters needing to be calibrated;
2) Size of generation, meaning the total number of alternative solutions;
3) Method for encoding and decoding the true values of parameters for crossover and mutation;
4) Maximum number of iteration generations.

After that, the values of parameters in each chromosome (one "chromosome" is seen as one "solution" in GA), meaning the values of parameters in each initial solution, are randomly initialized.

Step 2 is traffic simulation. For one chromosome, firstly, the traffic flow characteristics data and simulation-related settings (e.g. traffic flow rate, truck percentage, simulation time, etc.) are inputted into the simulation model. Then, the parameter values in one chromosome are input into the simulation model. After that, the step runs the simulation model, and obtains output values of the variables (e.g. speed, headway) for calculating the objective function and fitness function of one chromosome. This procedure is repeated for all chromosomes.

Step 3 is fitness calculation. Fitness measures the probability of being inherited or eliminated for one chromosome. The value of fitness is usually calculated based on the objective function for optimization. This step determines the equation for objective function and fitness function, and objective values and fitness for each chromosome are calculated using simulation data obtained from Step 2.

Step 4 is selection. This step determines the method for selecting chromosomes that could be put into the next generation, also seen as select “elitism”. Then, the selected elitisms are put into the next generation. The total number of selected “elitism” is equal to the number of chromosomes in the next generation, and the number of chromosomes in one generation is not changed for each generation. Thus, one chromosome could be selected as the “elitism” more than one time and put into the next generation.

Step 5 is crossover and mutation. This step:

1) Encodes parameter values;
2) Determines the probability of crossover;
3) Implements the crossover for the codes between two randomly selected chromosomes (e.g. switch “0” and “1” between two chromosomes if encoded as binary format);
4) Determines the probability of mutation;
5) Implements the mutation for the codes after crossover (e.g. use “0” to replace “1” if encoded as binary format);
6) Decodes the results to parameter values to get a new generation.

Step 6 is to repeat steps 2 to 5, until the maximum number of generations is reached. The chromosome with the best fitness function in Step 3 in all generations is the best solution for parameter calibration.

In Step 1, the parameters needing to be calibrated in CORSIM 6.3 are:

1) Time to complete a lane changing maneuver;

2) Mandatory lane change gap acceptance parameter;
3) Percentage of drivers desiring to yield right-of-way to lane changing vehicles attempting to merge;
4) Multiplier for desire to make a discretionary lane change;
5) Advantage threshold for discretionary lane change;
6) Lag acceleration and deceleration;
7) Minimum separation for generation of vehicles;
8) Leader's maximum deceleration perceived by follower.

All these parameters are FRESIM properties in CORSIM 6.3. The parameters need to be calibrated in VISSIM 9.0 are lane changing parameters and *Wiedemann 99* model car following parameters. The lane changing parameters include

1) Maximum look ahead/back distance;
2) Maximum deceleration of own/trailing vehicle.

The *Wiedemann 99* model car following parameters include

1) CC0 (standstill distance);
2) CC1 (headway time);
3) CC2 ('Following' variation);
4) CC3 (Threshold for entering 'Following');
5) CC4/CC5 (Negative/Positive 'Following' threshold);
6) CC6 (Speed dependency of oscillation).

The size of generation is 30. For encoding and decoding methods, the true parameter values are represented by binary format, meaning each true value in parameter is encoded as a binary bit string. The decoding process changes the value from binary format to decimal integer format. Using the binary format it is easy to operate crossover and mutation. For example, the 2, 4, 8 can be encoded as 010, 100, 111, and the 101, 110, 001 can be decoded as 5, 7, 1 respectively. The maximum number of generations (iterations) is 30. The initial values of parameters for each chromosome are randomly generated within the value default range of these parameters.

In Step 2, because the simulation model is calibrated based on empirical data from all data collection sites, a one hour empirical data set from each of the 13 data collection sites is selected into simulation, which means there are 13 hours of empirical data used for calibration in total. For each hourly empirical data, the traffic flow rate and truck percentage in this hour are used as simulation input. The simulation time for each empirical hourly data is 10800 simulation seconds (ss), consisting of 3600ss for loading, 3600ss for steady state, and 3600ss for unloading. The 13 hours of traffic flow are simulated one after one. For one-time consecutive 13 hours of traffic flow simulation, only the parameter values from one chromosome are used. In this research the operating speed and headways are used as the variables for calculating the objective function and fitness function. Thus, in this step, we can obtain the average operating speed and average headway of each hour corresponding to one chromosome.

In Step 3, the MAER (the mean absolute error ratio) is defined as the average ratio of all deviations to their observations in operating speed or headway taken without regard to sign, as shown in the following equation:

$$MAER_i(S_r) = \frac{1}{n}\sum_{a=1}^{n}\frac{|D_{ia} - D_{iar}|}{D_{ia}} \quad (7.1)$$

$MAER_i(S_r)$	mean absolute error ratio for the *r*th candidate solution corresponding to variable i
i	indicator of variables, 1 = operating speed, 2 = headway
S_r	the *r*th candidate solution
n	number of empirical hourly data used for simulation – for here, n = 13
a	indicator for empirical hourly data
D_{ia}	observed average value of variable i in hour a
D_{iar}	simulation average value of variable i in hour a with the *r*th candidate solution

The average MAER of operating speed and headway is used as the objective function and metrics for the calibration, as shown in the following equation.

$$MAER(S_r) = \frac{1}{2}\left(MAER_1(S_r) + MAER_2(S_r)\right) \quad (7.2)$$

$MAER(S_r)$	average mean absolute error ratio for the *r*th candidate solution
$MAER_1(S_r)$	mean absolute error ratio based on operating speed for the *r*th candidate solution

$MAER_2(S_r)$	mean absolute error ratio based on headway for the *r*th candidate solution

It also needs to define a “fitness function” in the GA. Since the objective of MAER is to get the minimum value, we need to equivalently change it to another function to get the maximum value. The fitness function corresponding to operating speed or headway is defined as:

$$h_i(S_r) = Ce^{-\beta_i * \mathrm{MAER}_i(S_r)} \tag{7.3}$$

$h_i(S_r)$	fitness of the *r*th candidate solution corresponding to variable i
i	indicator of variables, 1 = operating speed, 2 = headway
$MAER_i(S_r)$	mean absolute error ratio for the *r*th candidate solution corresponding to variable i
C, β_i	coefficients, where C = 100, β_1 = β_2 = 2.5

The final fitness function is the weighted sum of the fitness function for operating speed and headway, as displayed in the following equation:

$$h(S_r) = \alpha * h_1(S_r) + (1 - \alpha) * h_2(S_r) \tag{7.4}$$

$h(S_r)$	weighted sum of fitness for the *r*th candidate solution
$h_1(S_r)$	fitness of the *r*th candidate solution corresponding to operating speed

$h_2(S_r)$ fitness of the *r*th candidate solution corresponding to headway

α control factor, where α = 0.5

In Step 4, the elitism is selected by a “roulette wheel” selection scheme. The scheme is also called “fitness proportionate” selection. During the procedure, each chromosome was assigned a slice on a Monte Carlo-based roulette wheel proportional to its fitness. The “wheel” was “spun” in a simulated fashion N times, to select N elitisms into the next generation. N is the number of chromosomes in one generation (the value is 30 here).

In Step 5, the parameter values of each chromosome are encoded as binary format. The probability of crossover is 0.7. In this research, for one crossover operation, it only occurs between two chromosomes, meaning no more than two chromosomes are crossed at one time. The probability of mutation is 0.01. Here, the “mutation” means change 0 to 1 or vice versa. After that, the new results are decoded from binary to decimal format.

The calibration processes, including the best MAER and best fitness at each generation (iteration), are shown in Figure 7.5 and Figure 7.6.

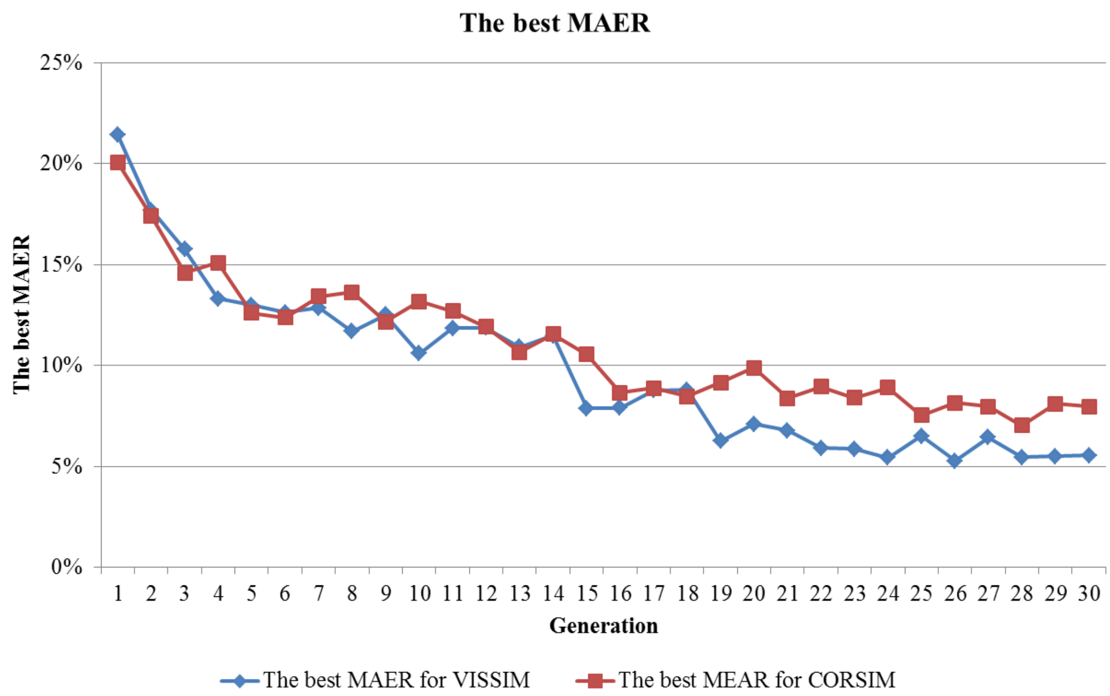


Figure 7.5 The best MAER at each generation for equal-density method

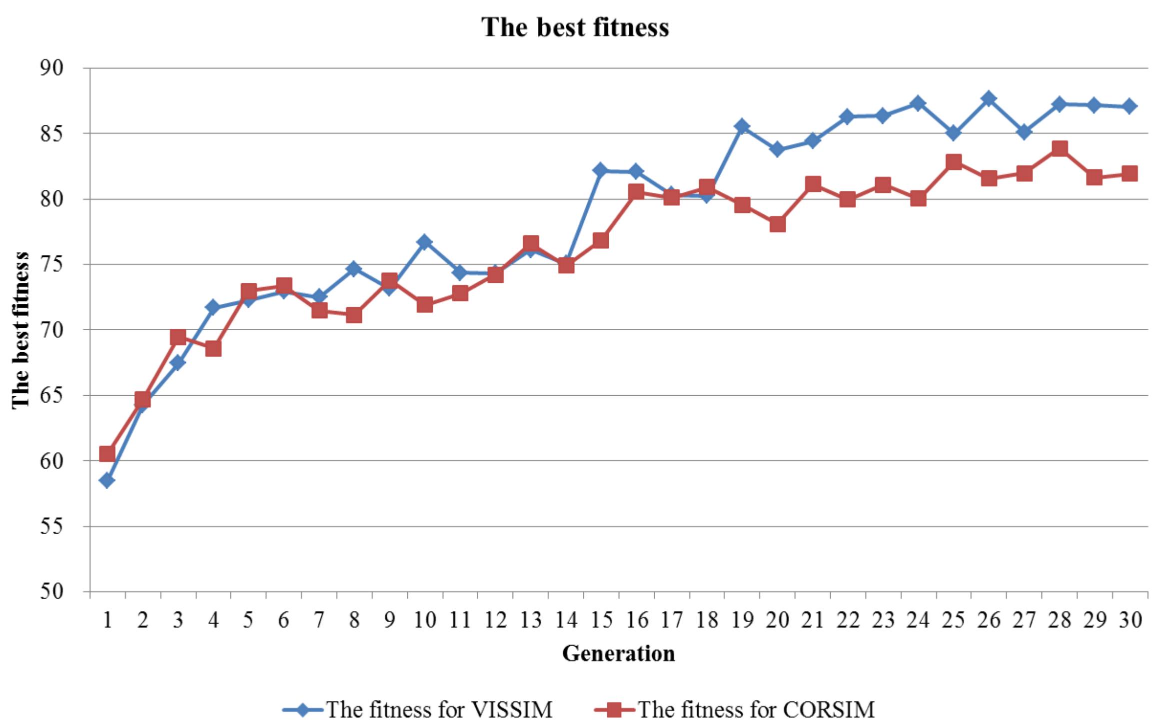


Figure 7.6 The best fitness at each generation for equal-density method

7.1.3 Simulation Approach

For each combination of traffic volume and truck percentage, one simulation run consists of the same three parts:

1) One-hour network loading so that the vehicles achieve a steady-state;
2) Two-hour steady-state with constant volume, where moving bottlenecks could be observed (this is the data used in the analysis);
3) One-hour traffic unloading.

In either CORSIM 6.3 or VISSIM 9.0 simulation models, there are six steps for calculating the ED_PCE values based on the equal-density method (as seen in Chapter 2.5.2):

Step 1: Develop the volume-density curve for the base (e.g. passenger car only) flow using simulation data with six traffic volume levels and a fixed random seed. Hourly volume (veh/h/ln) is estimated based on the 15-min interval traffic volume, and density (veh/mi/ln) is estimated by dividing the 15-min based hourly volume by the average speed in the 15-min interval.

Step 2: Develop the volume-density curve for mixed (e.g. p_t percent trucks and (1-p_t) percent passenger cars) flow using simulation data with the same six traffic volume levels and the same random seed as Step 1. The hourly volume and density are estimated in the same manner as Step 1. Nine levels of truck percentage are simulated as described above, and mixed flow density volume curves are estimated.

Step 3: Develop the volume-density curve for the subject (e.g. (p_t+5) percent trucks and (1-p_t-5) percent passenger cars) flow using data with the six traffic volume levels and the same random seed as Step 1. The hourly volume and density are estimated in the same manner as Step 1. Nine levels of truck percentage are simulated (e.g. 10 percent to 90 percent at 10 percent intervals), and the subject flow density volume curves are estimated.

Step 4: The subject volume simulated ranges from 500 to 1500 veh/h/ln at intervals of 200 veh/h/ln. For each subject flow volume (q_s), the corresponding equal density value is estimated from the volume-density curve for subject flow. Then, the mixed volume (q_M) and base flow volume (q_B) that has the same equal density are estimated using the volume-density curves for the mixed and base flow, respectively.

Step 5: The ED_PCE for each combination of volume (q_S) and truck percentage (p_t) for a specific random seed is estimated using Equation 2.1.

Step 6: The above steps are repeated 19 times using different random number seeds following standard simulation protocols. A total of 20 ED_PCEs are obtained for each combination of q_s and p_t. These 20 ED_PCE values are averaged to provide a final ED_PCE estimate. The developed volume-density curves in Step 1, Step 2, and Step 3 for each of the 20 simulation times under each of the simulation scenarios (CORSIM 6.3 with a single free-flow speed, VISSIM 9.0 with a single free-flow speed distribution, and VISSIM 9.0 with empirical free-flow speed distributions), are shown in the appendix.

7.2 PCE Based on Equal-Density Method

7.2.1 ED_PCE Estimation Results

ED_PCE mean values based on CORSIM 6.3 simulation data by equal-density method are shown in Table 7.3, as a function of traffic volume and truck percentage. The table shows values ranging from 1.3 to 1.9. In general, as traffic volume increases, so do the ED_PCE values. However, there is no clear relationship observed between PCE values and truck percentage.

Table 7.3 Average ED_PCEs with a single free-flow speed in CORSIM 6.3

Traffic Volume (veh/h/ln)	**Truck Percentage (%)**								
	5	15	25	35	45	55	65	75	85
500	1.5	1.6	1.4	1.5	1.5	1.4	1.4	1.3	1.5
700	1.7	1.4	1.5	1.6	1.6	1.6	1.6	1.6	1.6
900	1.6	1.5	1.6	1.6	1.8	1.7	1.5	1.7	1.6
1100	1.5	1.7	1.8	1.7	1.7	1.7	1.7	1.7	1.7
1300	1.6	1.6	1.7	1.7	1.7	1.9	1.9	1.7	1.8
1500	1.7	1.7	1.8	--	--	--	--	--	--

*Note: -- represents lacking adequate observations for this group.

Figure 7.7 shows the results graphically as a function of traffic volume and truck percentage. The larger rectangles represent the combined conditions while the smaller rectangles within it represent the values from one of the 20 repetitions (same as Figure 7.8 and Figure 7.9). Not surprisingly, results show that the ED_PCE values obtained are all very similar to each other for a given run. As before, results in Table 7.3 and Figure 7.7 are based on the assumption that all vehicle classes have the same single free-flow speed.

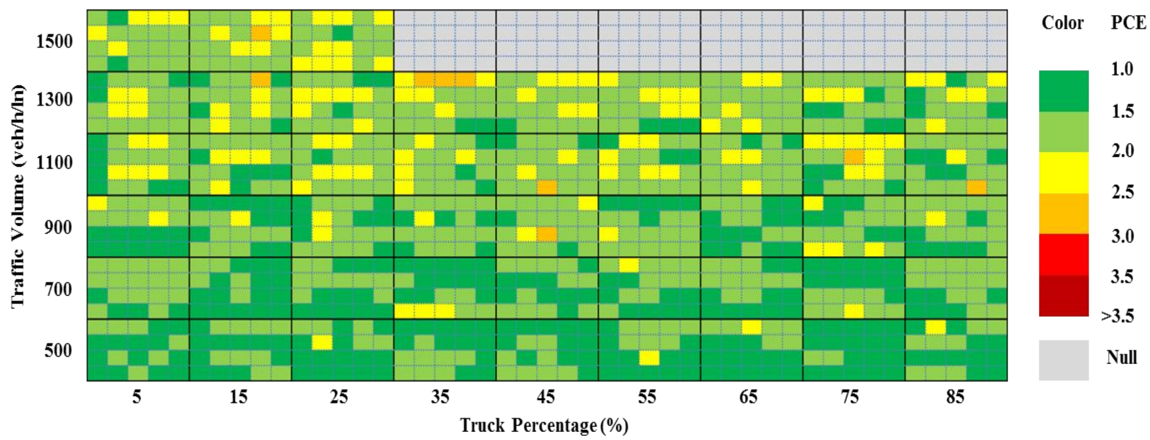


Figure 7.7 ED_PCEs with a single free-flow speed in CORSIM 6.3

The ED_PCE mean values based on VISSIM 9.0 simulation data, calculated using a single free-flow speed distribution, are shown in Table 7.4 and Figure 7.8 as a function of traffic volume and truck percentage. Table 7.4 and Figure 7.8 show values ranging from 1.5 to 2.1, which is similar to values from the CORSIM analysis. As before, traffic volume increases with ED_PCE values. There is no clear relationship observed between the ED_PCE values and truck percentage. These results indicate that VISSIM 9.0 and CORSIM 6.3 simulation models give similar ED_PCE results if a single free-flow speed is used in CORSIM 6.3 and a single free-flow speed distribution is used in VISSIM 9.0.

Table 7.4 Average ED_PCEs with a single free-flow speed distribution in VISSIM 9.0

Traffic Volume (veh/h/ln)	Truck Percentage (%)								
	5	15	25	35	45	55	65	75	85
500	1.5	1.6	1.6	1.7	1.5	1.6	1.6	1.5	1.7
700	1.7	1.6	1.7	1.7	1.7	1.6	1.7	1.6	1.6
900	1.6	1.9	1.8	1.7	1.8	1.7	1.8	1.7	1.7
1100	1.8	1.9	1.7	1.8	1.7	1.7	1.9	1.8	1.8
1300	1.8	2.0	1.9	1.9	1.8	1.8	2.1	2.1	1.9
1500	1.9	1.9	2.0	--	--	--	--	--	--

*Note: -- represents lacking adequate observations for this group.

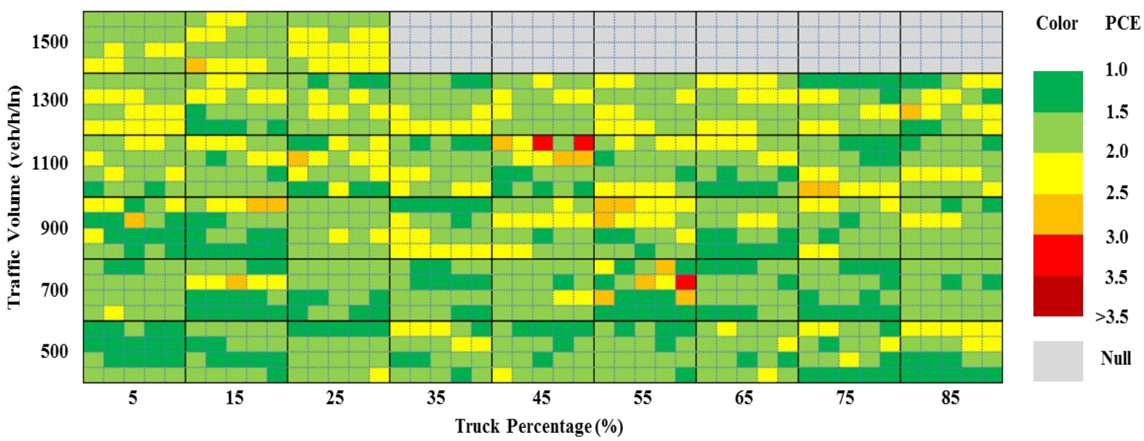


Figure 7.8 ED_PCEs with a single free-flow speed distribution in VISSIM 9.0

The ED_PCE mean values based on VISSIM 9.0 simulation data and calculated using the three empirical free-flow speed distributions are shown in Table 7.5 and Figure 7.9 as a function of traffic volume and truck percentage. Table 7.5 and Figure 7.9 show values ranging from 2.2 to 3.0. As before, traffic volume increases with ED_PCE values. It is hypothesized that the speed differential causes moving bottlenecks to form similar to what was observed in the field. This translates into higher density values in traffic for a given truck volume and resulted in higher ED_PCE values. There is no clear relationship observed between ED_PCE values and truck percentage. This result is similar to what was found in the 2010 HCM.

Table 7.5 Average ED_PCEs with empirical free-flow speed distributions in VISSIM 9.0

Traffic Volume (veh/h/ln)	Truck Percentage (%)								
	5	15	25	35	45	55	65	75	85
500	2.2	2.3	2.5	2.4	2.5	2.5	2.4	2.3	2.5
700	2.4	2.3	2.4	2.6	2.4	2.3	2.5	2.5	2.5
900	2.5	2.5	2.7	2.5	2.8	2.5	2.6	2.7	2.6
1100	2.7	2.5	2.8	2.8	2.7	2.7	2.7	2.8	2.5
1300	2.8	2.8	2.9	2.7	2.8	2.8	2.8	2.7	2.7
1500	2.8	2.7	2.7	--	--	--	--	--	--

*Note: -- represents lacking adequate observations for this group.

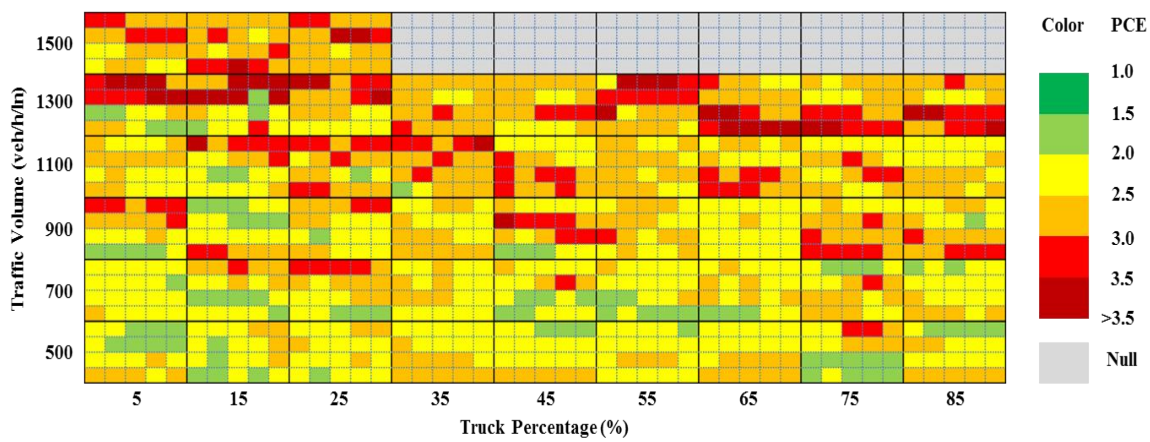


Figure 7.9 ED_PCEs with empirical free-flow speed distributions in VISSIM 9.0

7.2.2 Influencing Factor Analysis

To explore the influence of factors, a set of ANOVA analysis is conducted:

1) For each of three scenarios, the difference in ED_PCEs among different traffic volume levels;

2) For each of three scenarios, the difference in ED_PCEs among different truck percentage levels;
3) The difference in ED_PCEs between calculated by CORSIM 6.3 data with a single free-flow speed and VISSIM 9.0 data with a single free-flow speed distribution;
4) The difference in ED_PCEs between calculated by VISSIM 9.0 data with the empirical free-flow speed distributions and VISSIM 9.0 data with a single free-flow speed distributions, or CORSIM 6.3 data with a single free-flow speed.

Results for ANOVA analysis are shown in Table 7.6. It is found that with respect to the ED_PCEs values:

1) The ED_PCEs increase with traffic volume for all scenarios;
2) ED_PCEs do not vary appreciably with truck percentage for all scenarios;
3) There is no difference between ED_PCEs calculated by VISSIM 9.0 data with a single free-flow speed distribution and by CORSIM 6.3 data with a single free-flow speed;
4) ED_PCEs calculated using VISSIM 9.0 data with the empirical free-flow speed distributions are higher than those calculated using either VISSIM 9.0 data or CORSIM 6.3 data with a single free-flow speed distribution.

The above results are all statistically analyzed based on a F-test at the 95 percent level of confidence.

Table 7.6 Results of ANOVA analysis for ED_PCEs

Analysis	F-test results	P-value	Conclusion
Different traffic volume levels (CORSIM 6.3, a single free-flow speed)	26.01	< 0.05	Significant
Different traffic volume levels (VISSIM 9.0, a single free-flow speed distribution)	24.72	< 0.05	Significant
Different traffic volume levels (VISSIM 9.0, empirical free-flow speed distributions)	49.56	< 0.05	Significant
Different truck percentage levels (CORSIM 6.3, a single free-flow speed)	1.66	0.10	Not significant
Different truck percentage levels (VISSIM 9.0, a single free-flow speed distribution)	1.62	0.11	Not significant
Different truck percentage levels (VISSIM 9.0, empirical free-flow speed distributions)	1.47	0.16	Not significant
VISSIM 9.0, a single free-flow speed distribution v.s. CORSIM 6.3, a single free-flow speed	1.09	0.10	Not significant
VISSIM 9.0, empirical free-flow speed distributions v.s. CORSIM 6.3, a single free-flow speed	> 100	< 0.05	Significant
VISSIM 9.0, empirical free-flow speed distributions v.s. VISSIM 9.0, a single free-flow speed distribution	> 100	< 0.05	Significant

7.3 Comparison between ED_PCEs and the Recommended PCEs in HCM 2010

A summary of the ED_PCEs calculated under western rural U.S. conditions with HCM 2010 PCE values are shown in Table 7.7. The table shows that calculated values in this research, when it is assumed that trucks and passenger cars travel at the same free-flow speed, are only marginally higher than PCEs recommended in HCM 2010.

Table 7.7 Comparison between the new ED_PCEs with the recommended PCEs in HCM 2010

Truck (%)	**5**	**15**	**25**	**35**	**45**	**55**	**65**	**75**	**85**
HCM 2010	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Average new PCE (Single free-flow speed in CORSIM 6.3)	1.6	1.6	1.6	1.6	1.7	1.7	1.6	1.6	1.6
Average new PCE (Single free-flow speed distribution in VISSIM 9.0)	1.7	1.8	1.8	1.8	1.7	1.7	1.8	1.7	1.7
Average new PCE (Empirical free-flow speed distributions in VISSIM 9.0)	2.6	2.5	2.7	2.6	2.6	2.6	2.6	2.6	2.6

*Note:
1) The new ED_PCEs in Tables 7.3, 7.4, and 7.5 are averaged so that they could be compared with the values in HCM 2010.
2) The recommended PCE value on level terrain is 1.5 for the 2010 HCM.

However, when the empirical free-flow speed distributions are used, the calculated ED_PCEs are considerably higher, approximately 50 percent to 100 percent higher than HCM 2010 recommended values at level terrain. Especially for truck percentages higher than 25 percent, ED_PCE values in this research are nearly 70 percent higher than recommended PCEs in HCM 2010.

It is also argued that the higher calculated ED_PCE values better capture the relationship between trucks and passenger cars on this corridor. For each of the data collection sites it was found that observed speeds are 0 to 20 percent lower than which was to be expected from HCM 2010, if the speed-flow curve for FFS = 75 mph (Exhibit 11-2 in HCM 2010) was selected for analysis, as shown in Figure 7.10.

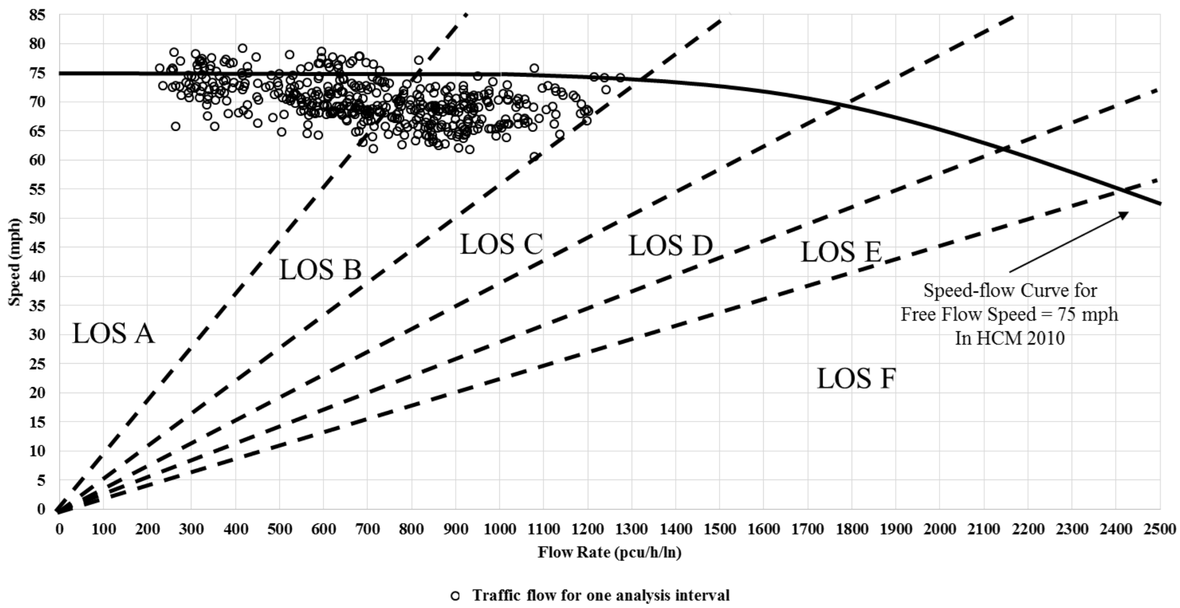


Figure 7.10 Relationship between observed traffic flow and FFS = 75 mph speed-flow curve in HCM 2010 with PCE = 1.5

In Figure 7.10, the observed speeds correspond to the LOS C or D section on the curve. If the larger ED_PCEs calculated in this research are used, the estimated LOS will change, as shown in Figure 7.11.

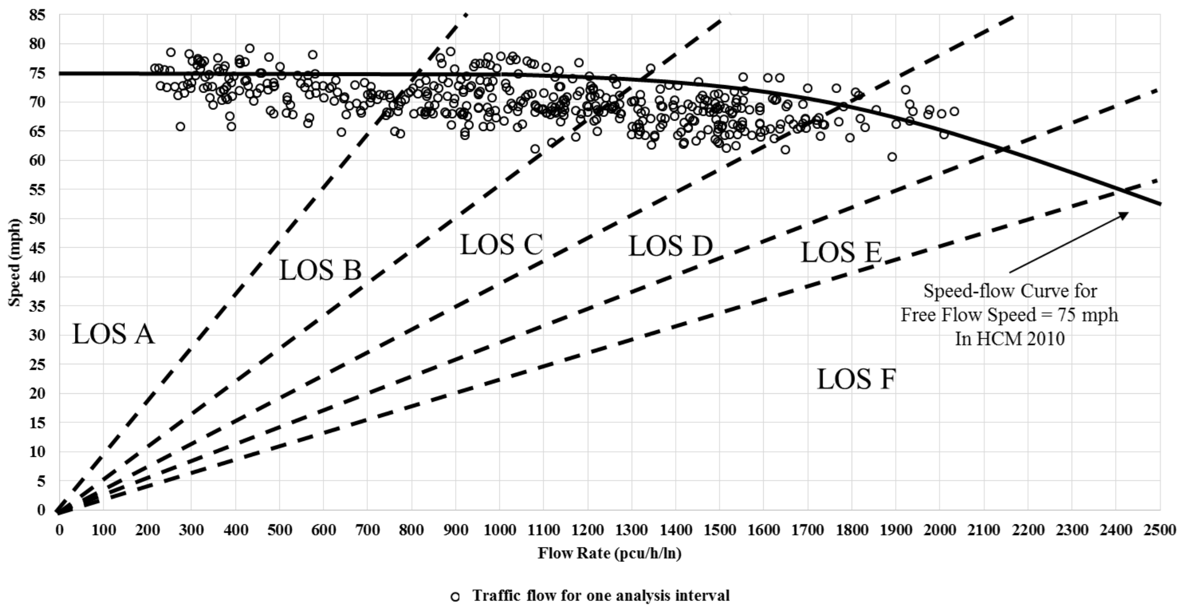


Figure 7.11 Relationship between observed traffic flow and FFS = 75 mph speed-flow curve in HCM 2010 with new ED_PCEs

As seen in Figure 7.12, if the HCM 2010 recommended PCE values are used, with the operational analysis procedures in Chapter 5, 54 percent of the sites would be classified as having a LOS A and 46 percent of the sites would be classified as having a LOS B. However, if the new calculated ED_PCEs are used, with the same operational analysis procedures in Chapter 5, the results would be 23 percent LOS A, 33 percent LOS B, 37 percent LOS C, and 7 percent LOS D, which means drivers feel traffic flows do not move as smoothly as expected under moving-bottleneck conditions.

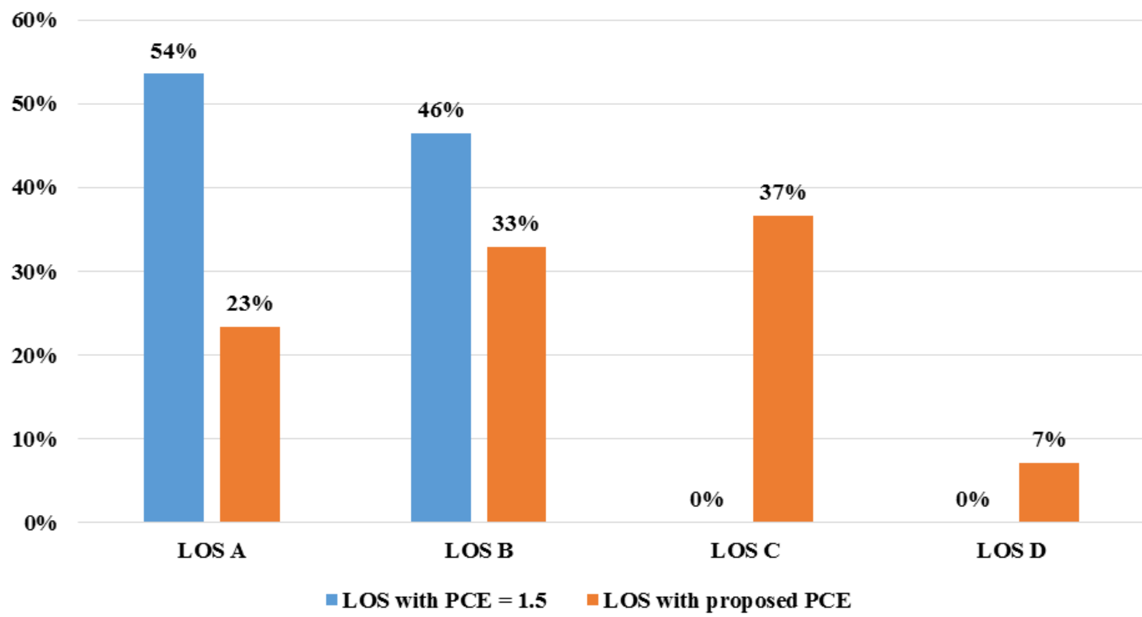


Figure 7.12 Comparison of LOS results between using PCE = 1.5 and new ED_PCEs

7.4 Summary

In this research, ED_PCEs are calculated by simulation data from CORSIM 6.3 and VISSIM 9.0. Firstly, this chapter develops simulation models in CORSIM 6.3 and VISSIM 9.0 using a 4-link clockwise grid road network. Values of geometrics and vehicles parameters are set based on Nebraska and western rural U.S. empirical data, and HCM 2010. A number of different combinations of volume and truck percentages are simulated. Six levels of traffic volume are simulated beginning at 500 veh/h/ln and increasing to 1500 veh/h/ln. Nine levels of truck percentages are simulated beginning at 5 percent, and increasing to 85 percent at 10 percent increments. There are 20 runs for each combination of traffic volume and truck percentage. Three scenarios need to be simulated including:

1) CORSIM 6.3 with a single free-flow speed;

2) VISSIM 9.0 with a single free-flow speed distribution;

3) VISSIM 9.0 with empirical free-flow speeds distribution.

Secondly, the chapter calibrates simulation models based on Nebraska empirical data. Simulation models are calibrated with genetic algorithm (GA). Operating speed and headway are used as traffic flow metrics for calibration. Average of mean absolute error (MAER) of operating speed and MAER of headway is used as objective function. Finally, simulation models run four hours for each combination of influencing factors at each scenario, using the equal-density method procedure with five steps.

Results of ED_PCEs show that:

1) With CORSIM 6.3 data under a single free-flow speed condition, values range from 1.3 to 1.9, marginally higher than recommended values in HCM 2010;
2) With VISSIM 9.0 data under a single free-flow speed distribution condition, values range from 1.5 to 2.1, marginally higher than HCM 2010 values and similar to values from the CORSIM 6.3 analysis;
3) With VISSIM 9.0 data under empirical free-flow speed distributions condition, values range from 2.2 to 3.0, higher than HCM 2010 values, significantly higher than CORSIM 6.3 analysis and VISSIM 9.0 single free-flow speed analysis values.

ANOVA results show that for all scenarios, at four-lane level freeway segments, ED_PCEs increase with traffic volume but do not vary appreciably with truck percentage. It is also argued in this research that higher calculated ED_PCE values better capture the relationship between trucks and passenger cars on this corridor, since operational analysis results show the LOS based on the new ED_PCEs are lower than those based on the recommended values in HCM 2010.

CHAPTER 8 CALCULATING PCES BASED ON EQUAL-CAPACITY METHOD

8.1 Simulation Model Development for Equal-capacity Method

In this research the traffic flows are simulated by VISSIM 9.0 under two conditions – the HCM 2016 conditions, and the western rural U.S. conditions. These two conditions differ in three key aspects:

1) The HCM 2016 conditions simulate the traffic flow data on freeways segments with three lanes per direction. However, under the western rural U.S. conditions, the traffic flows are simulated on freeway segments with two lanes per direction.
2) The HCM 2016 conditions use the same desired free-flow speed 70 mph for all vehicle types on level terrain, as shown by the yellow dotted line in the Figure 8.1. However, under the western rural U.S. conditions, the empirical free-flow speed distributions are used for passenger cars and trucks, as shown by the three different solid lines in the Figure 8.1.

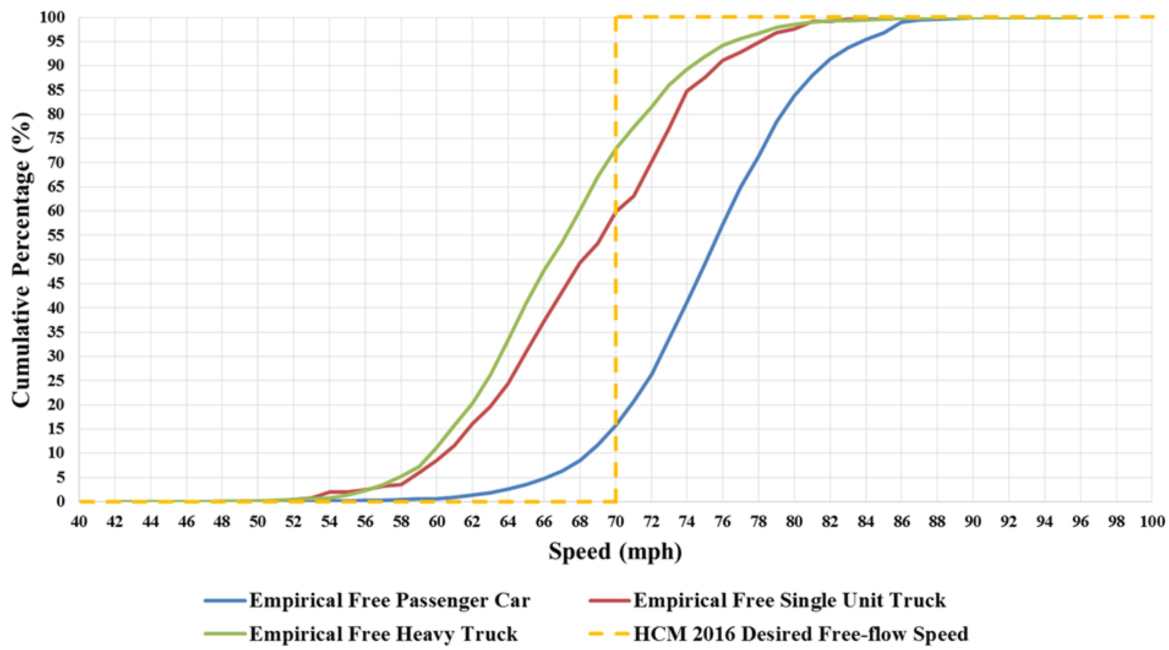


Figure 8.1 Free-flow speed distributions for free vehicles on level terrain under HCM 2016 and western rural U.S. (Nebraska empirical) conditions

3) In HCM 2016 conditions, the single-unit trucks (SUT) are modeled as having a length of 33 ft, and the heavy trucks (trailer trucks, TT) are modeled having the same length as an AASHTO WB50 tractor plus one AASHTO WB50 trailer. These assumed lengths are shown in Figure 8.2 by the dashed lines. Under the western rural U.S. conditions, the trucks are modeled having the empirical truck length distributions observed in Nebraska, which were 30 to 45 ft for SUT and 50 to 95 ft for TT, as shown in Figure 8.2 by the solid lines. It has been found the length of trucks affects the car-following and lane-changing behaviors (63).

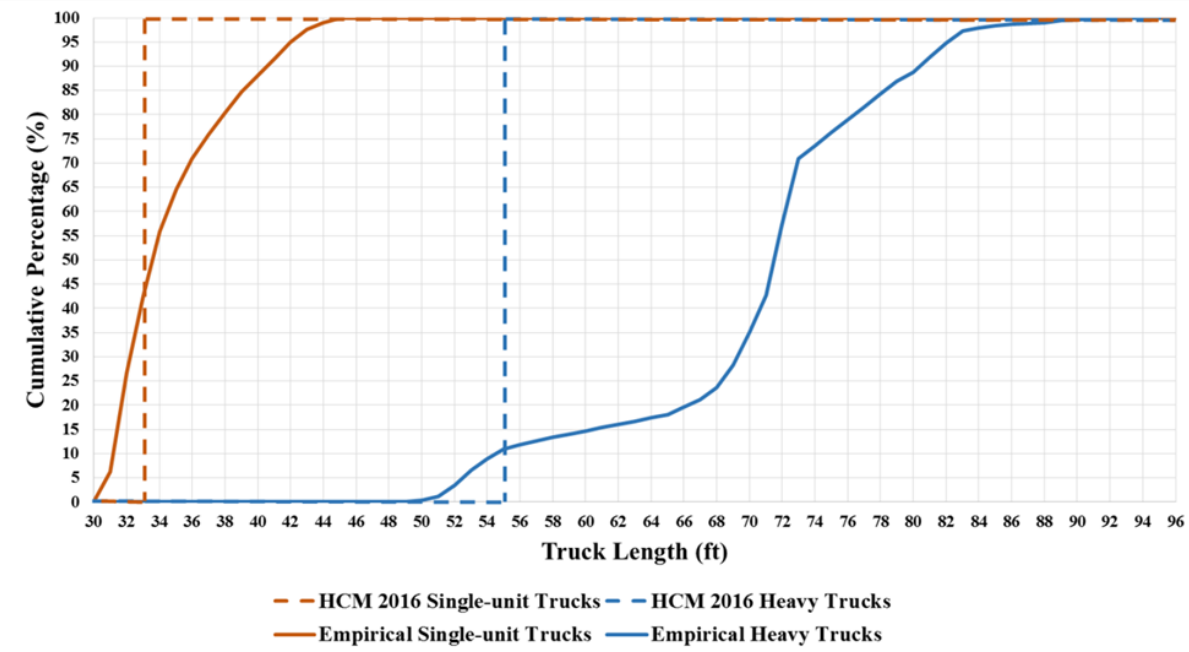


Figure 8.2 Truck length distributions under HCM 2016 and western rural U.S. (Nebraska empirical) conditions

The key parameters of the simulation model are shown in the Table 8.1. Any parameters not shown were set to VISSIM 9.0 values.

Table 8.1 Key parameters for HCM 2016 and western rural U.S. conditions in VISSIM 9.0 simulation model

Group	Item	Settings
Highway geometrics	Highway length	15 miles uni-directional freeway segment, consisting of 8 miles level followed by 6 miles grade and followed by 1 mile level
	Lane width	12 feet
	Grade	13 levels, from -6% to 6% with 1% as interval
	Number of lanes*	HCM 2016 conditions: 3 lanes per direction. Western rural U.S. conditions: 2 lanes per direction.
Vehicle	Vehicle type	Passenger cars, single-unit trucks (SUT), heavy trucks (TT)
	Weight distribution for trucks	SUT: 1,000 to 60,000 kg with median 43,000 kg, TT: 1,000 0 to 90,000 kg with median 50,000 kg, as shown in Figure 8.3.
	Power distribution for trucks	SUT: 80 to 350 kw with median 200 kw, TT: 100 to 300 kw with median 200 kw, As shown in Figure 8.4.
	Truck length*	HCM 2016 conditions: 33 ft for SUT, 55 ft for TT. Western rural U.S. conditions: 30 to 45 ft for SUT, 50 to 95 ft for TT, as shown in Figure 8.2
	Maximum acceleration	Passenger car: 11.5 ft/s^2, SUT: 6.6 ft/s^2, TT: 4.7 ft/s^2
	Desired acceleration	Passenger car: 11.5 ft/s^2, SUT: 21.3 ft/s^2, TT: 21.3 ft/s^2
	Maximum deceleration	Passenger car: -16.7 to -24.6 ft/s^2, SUT/TT: -5.6 ft/s^2
	Desired deceleration	Passenger car: -9.0 ft/s^2, SUT/TT: -1.2 ft/s^2
Traffic flow	Truck percentage	0%, 2%, 5%, 10%, 15%, 20%, 30%, 40%, 50%, 60%, 70%, 80%, 90%, 100%.
	Flow rate	0 to 7200 veh/h (240, 600, 1200, 1800, 1920, 2040, 2160, 2880, 2400 veh/h/ln)
	Desired speed*	HCM 2016 conditions: 70 mph for all vehicle types. Western rural U.S. conditions: different free-flow speed distributions for passenger cars, SUTs and TTs in simulation, as shown in Figure 8.1.
	Truck composition	30% SUT/70% TT, 50% SUT/50% TT, 70% SUT/30% TT
Driver behavior	Car following*	HCM 2016 conditions: *Wiedemann 99* model, default values. Western rural U.S. conditions: *Wiedemann 99* model, calibrated values for Nebraska traffic flow conditions (96).

	Lane changing strategy	Slow lane rules
Data collection	Aggregation interval	1 minute
	Location of detectors (Grade length)	0.25, 0.5, 0.75, 1, 1.5, 2.5 and 5 miles from the beginning point of the grade section, one detector for each lane
	Type of detectors	Data collection point detectors
	Valid data for analysis	1 hours data for each scenario at steady status
Simulation procedure	Simulation procedure	One-hour vehicle loading to achieve a steady-state. One-hour steady-state with constant volume, where moving bottlenecks could be observed. One-hour vehicle unloading.

Note: The item with * means there are difference in parameter settings between HCM 2016 and western rural U.S. conditions.

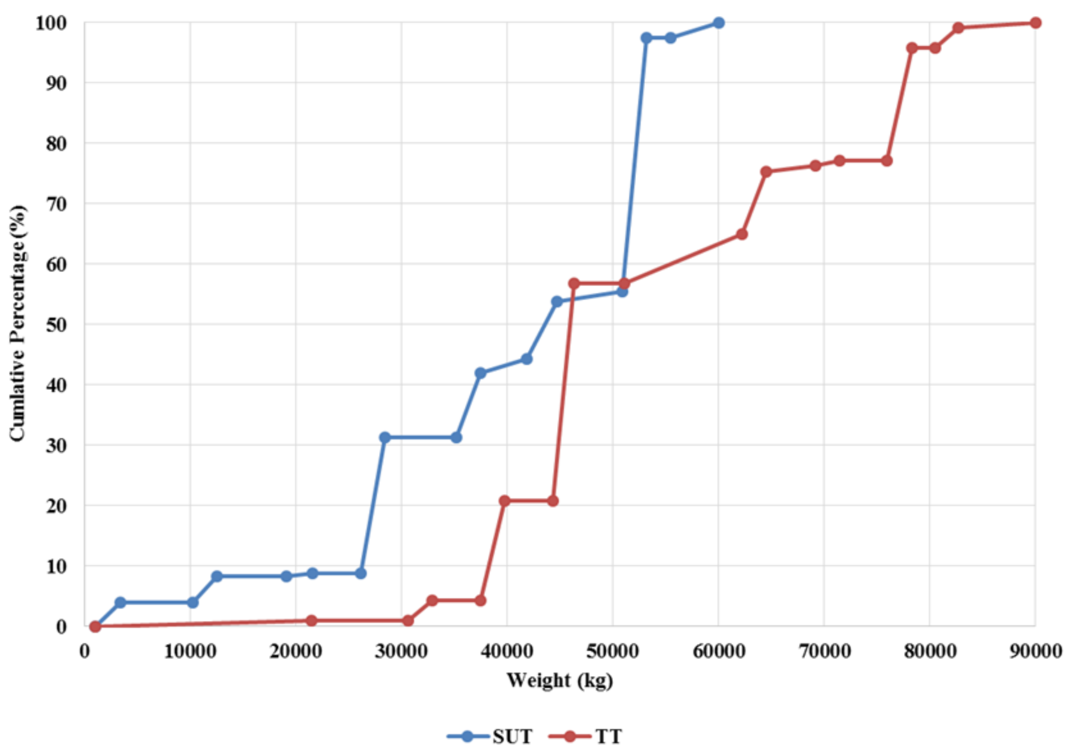


Figure 8.3 Weight distributions for SUTs and TTs for equal-capacity method

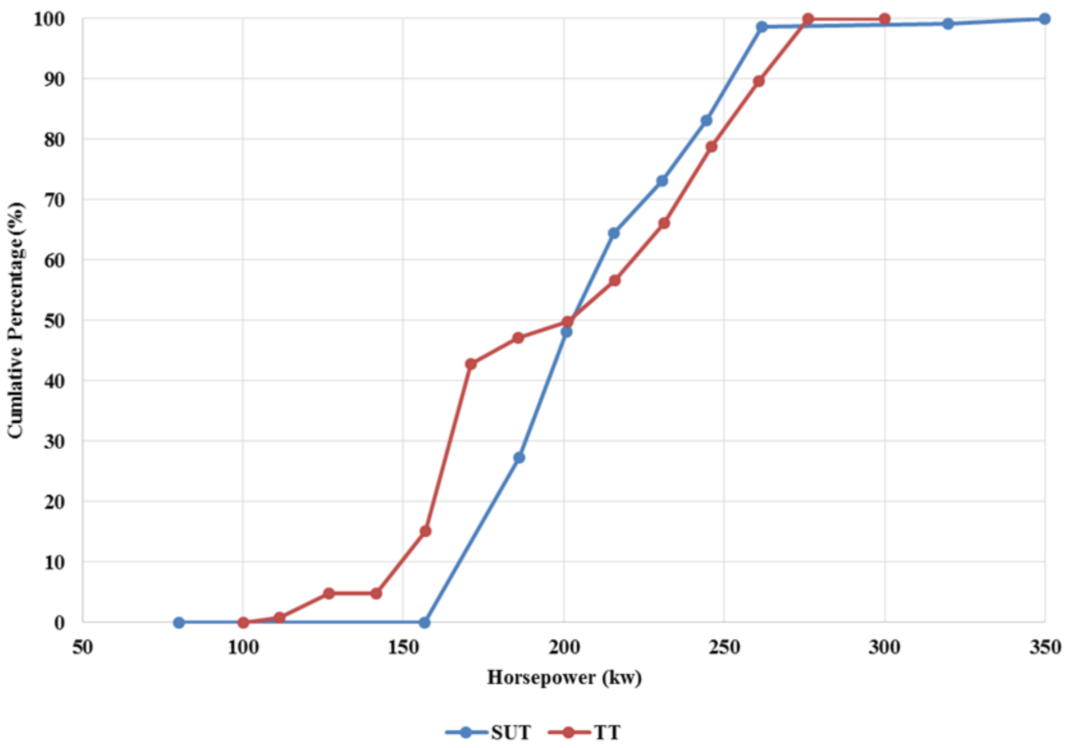


Figure 8.4 Horsepower distributions for SUTs and TTs for equal-capacity method

Figure 8.5 shows a schematic of the VISSIM simulation network that was used to create the PCE values in the 2016 HCM. It may be seen that there is an 8-mile level section followed by a 6-mile graded section followed by a one-mile level section. The data were collected on the upgrade at the point detectors locations (e.g. 0.25, 0.5, 0.75, 1.0, 1.5, 2.5 and 5 miles) at one-minute intervals. The simulation model was run for one hour for each scenario before any data was collected so that the network would reach steady state conditions before any data was collected. The data was then collected for one hour (e.g. 60 intervals of 1-minute length).

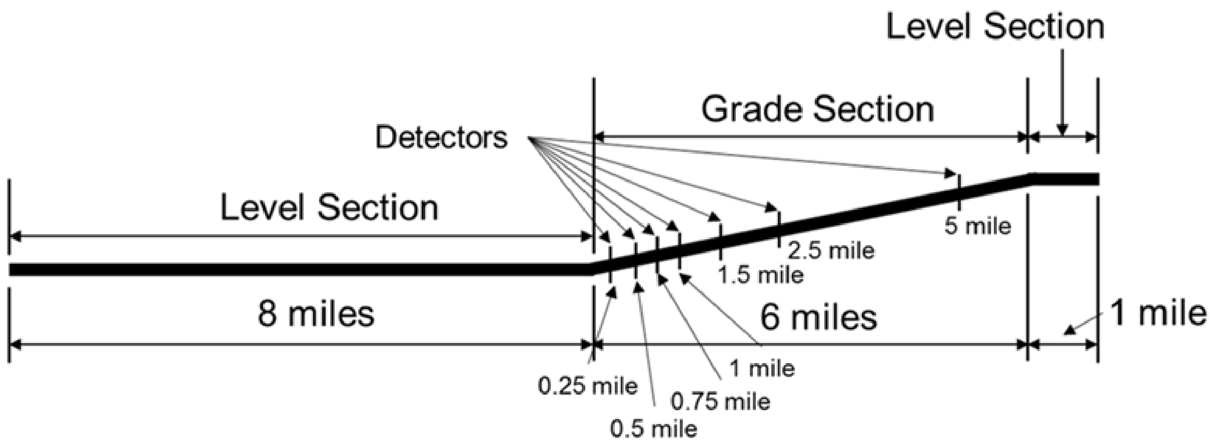


Figure 8.5 Schematic of the simulation model for equal-capacity PCE estimation

VISSIM 9.0 output data include detection of entering and exiting time, ordinal number, class, length, and speed for each vehicle (63). Similar to the process for ED_PCE calculation, it was also necessary to validate and calibrate the model. In the simulation model for equal-capacity method, parameters in VISSIM 9.0 are calibrated by Genetic Algorithm (GA). Steps in the calibration process and relative parameter settings are the same as that in section 7.1.2. The best MAER and best fitness at each generation (iteration) are shown in Figure 8.6 and Figure 8.7.

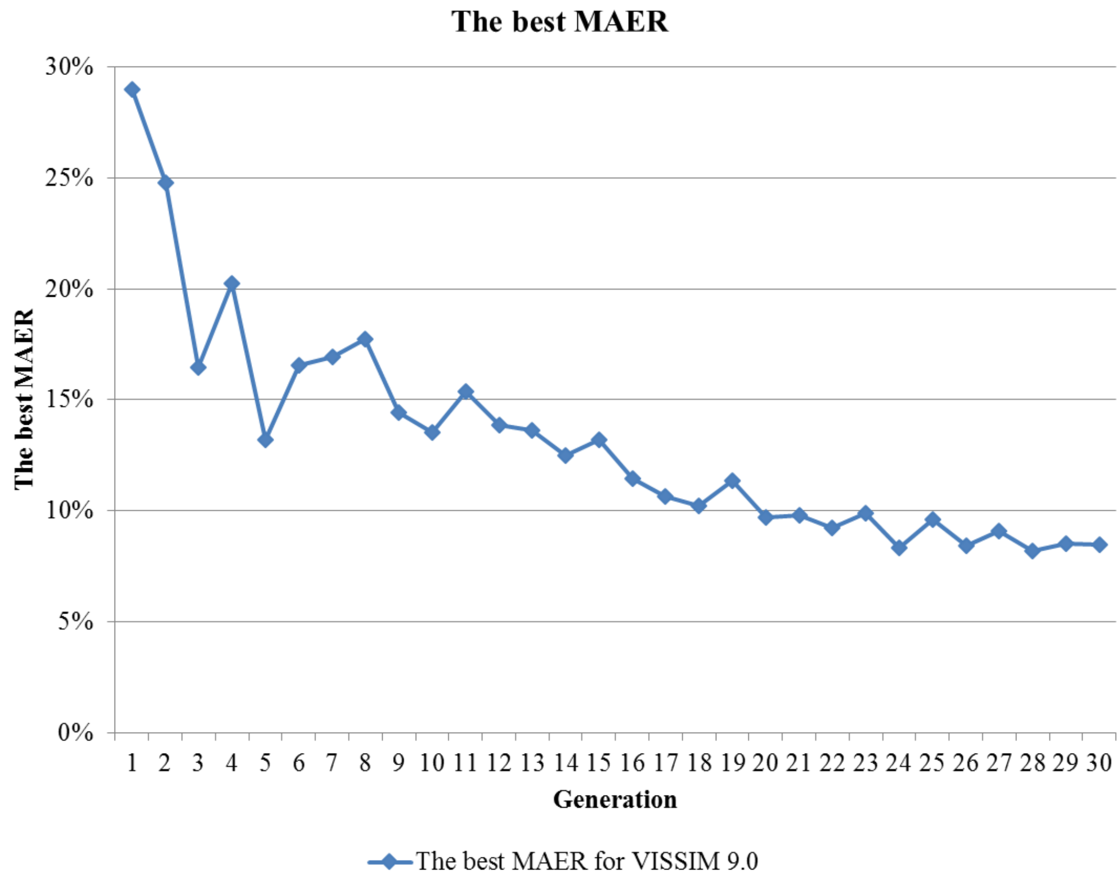


Figure 8.6 The best MAER at each generation for equal-capacity method

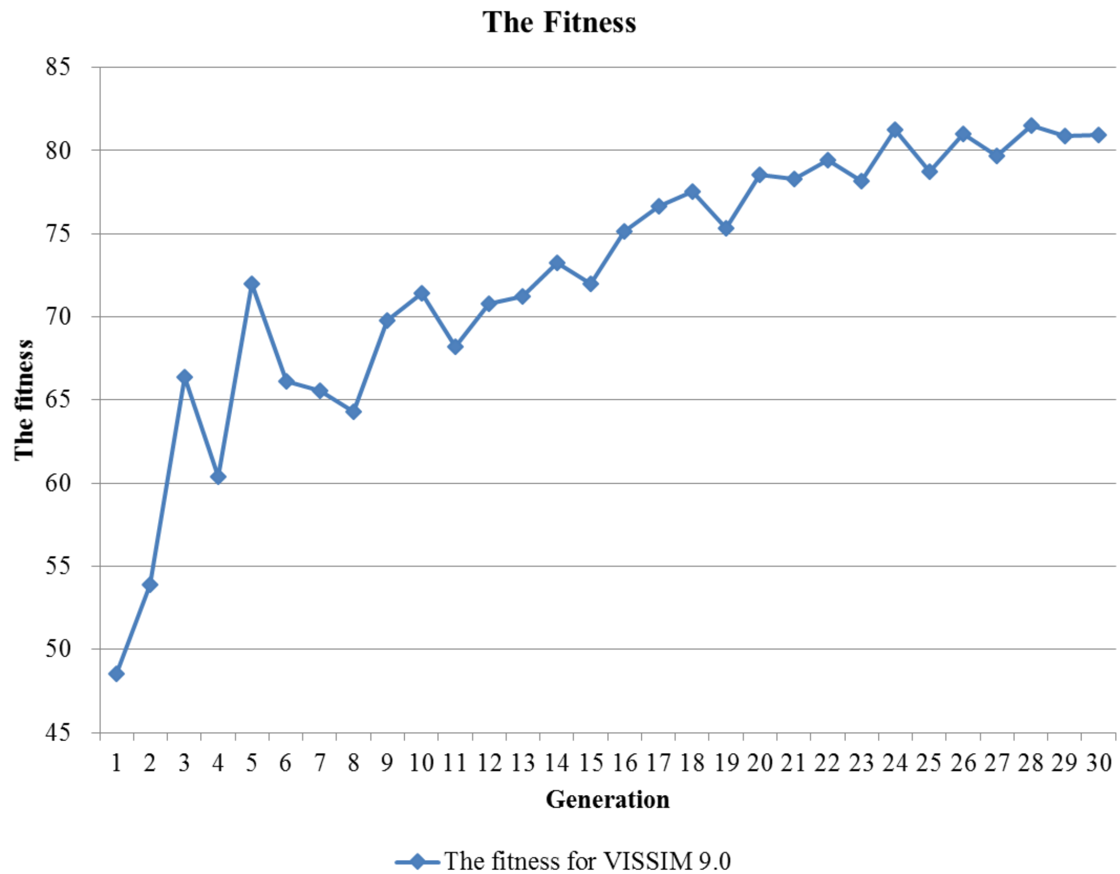


Figure 8.7 The best fitness at each generation for equal-capacity method

8.2 Procedure for PCE Estimation Based on Equal-capacity Method

The approach adopted here follows the HCM 2016 PCE estimation methodology, which was based on equivalent capacities (3) (39). This approach computes the PCE values based on obtaining equivalence in the capacity of the auto-only traffic flow and the capacity of mixed traffic flow. In this research the equal-capacity-based PCEs (EC_PCEs) are respectively estimated for two groups of data, which are based on

a) 1-minute flow-density data under the original research conditions (Group 1);

b) 1-minute flow-density data under western rural U.S. conditions (Group 2).

There are five steps necessary in order to calculate the EC-PCEs described below.

Step 1: Flow-density Plots Development for Auto-only and Mixed Flow

The first step is to simulate flow rate and density data for various pairs of flow rate and density values for the auto-only flow (passenger car only flow). These values are often published as scatter plots. One scatter plot is determined for each combination of grade and distance (grade length). Because there are 13 levels of grade and 7 levels of distance as described in Table 8.1, there are 91 scatter plots for auto-only flow in total.

Next, the flow-density scatter plots for the mixed flow (e.g. combined passenger car and truck flow) are developed. One scatter plot is determined for each "scenario", which is a combination of truck percentage, truck composition, grade, and distance. Because there were 13 levels of truck percentage, 3 levels of truck composition, 13 levels of grade, and 7 levels of distance, in the 2016 HCM procedure as described in Table 8.1, there are 3549 scatter plots for mixed flow in total.

To obtain a given scatter plot, the auto-only flow (or mixed flow, as appropriate) at each combination of truck percentage, truck composition, grade, and distance, needs to be simulated at 9 levels of flow rate, resulting in 9 simulation runs. In a given scatter plot, each point represents the relationship of flow-rate and density over a one minute period. Because there are 9 flow rate levels and the simulation data is collected for 60 1-minute periods, there will be 540 points on each scatter plot. The flow-rate at a given point ($q_{f,t,p,m,g,d,r}$) is estimated based on the 1-min interval traffic volume ($V_{f,t,p,m,g,d,r}$) recorded at the given detector using Equation 8.1:

$$q_{f,t,p,m,g,d,r} = V_{f,t,p,m,g,d,r} * 60 \tag{8.1}$$

$q_{f,t,p,m,g,d,r}$	Flow rate for the f flow type at t time interval, p truck percentage level, m truck composition level, g grade level, d distance level, and r simulation flow-rate level based on 1-min interval traffic volume recorded by the detector (veh/h/ln)
$V_{f,t,p,m,g,d,r}$	1-min interval traffic volume recorded by the detector for the f flow type at t time interval, p truck percentage level, m truck composition level, g grade level, d distance level, and r simulation flow-rate level (veh/min/ln)
f	Ordinal number of flow type, f = 1 for auto-only flow, f = 2 for mixed flow
t	Ordinal number of 1-min time interval, t = 1,2,…,T
T	Total numbers of 1-min time interval, T = 60
p	Ordinal number of truck percentage level. If f = 1, p = 0, means 0% truck percentage. If f = 2, p = 1,2,..., P, means 2% to 100% truck percentage.
P	Total levels of truck percentage, P = 13
m	Ordinal number of truck composition level, m = 0 for no trucks, m = 1 for 30% SUT/70% TT, m = 2 for 50% SUT/50% TT, m = 3 for 70% SUT/30% TT
g	Ordinal number of grade level, g = 1,2,…,G, means -6% to 6% grade

G	Total levels of grade, G = 13
d	Ordinal number of distance level (the level of detector location), d = 1,2,…,D, means 0.25 to 5 miles
D	Total levels of distance (detector location), D = 7
r	Ordinal number of simulation flow-rate level, r = 1,2,…,R, means 240 to 2400 veh/h/ln
R	Total levels of simulation flow-rate, R = 9

The density at a given point ($k_{f,t,p,m,g,d,r}$) is estimated by dividing the flow-rate at the given point ($q_{f,t,p,m,g,d,r}$) by the 1-min interval average space mean speed ($\bar{v}_{f,t,p,m,g,d,r}$) recorded by the detector (39) (40) at a given point, using Equation 8.2:

$$k_{f,t,p,m,g,d,r} = \frac{q_{f,t,p,m,g,d,r}}{\bar{v}_{f,t,p,m,g,d,r}} \tag{8.2}$$

$k_{f,t,p,m,g,d,r}$	Density for the f flow type at t time interval, p truck percentage level, m truck composition level, g grade level, d distance level, and r simulation flow-rate level (veh/mi/ln)
$q_{f,t,p,m,g,d,r}$	Flow rate for the f flow type at t time interval, p truck percentage level, m truck composition level, g grade level, d distance level, and r simulation flow-rate level based on 1-min interval traffic volume recorded by the detector (veh/h/ln)

$\bar{v}_{f,t,p,m,g,d,r}$	1-min interval space mean speed for the f flow type at t time interval, p truck percentage level, m truck composition level, g grade level, d distance level, and r simulation flow-rate level (mph)
f	Ordinal number of flow type, f = 1 for auto-only flow, f = 2 for mixed flow
t	Ordinal number of 1-min time interval, t = 1,2,…,T
T	Total numbers of 1-min time interval, T = 60
p	Ordinal number of truck percentage level. If f = 1, p = 0, means 0% truck percentage. If f = 2, p = 1,2,..., P, means 2% to 100% truck percentage.
P	Total levels of truck percentage, P = 13
m	Ordinal number of truck composition level, m = 0 for no trucks, m = 1 for 30% SUT/70% TT, m = 2 for 50% SUT/50% TT, m = 3 for 70% SUT/30% TT
g	Ordinal number of grade level, g = 1,2,…,G, means -6% to 6% grade
G	Total levels of grade, G = 13
d	Ordinal number of distance level (the level of detector location), d = 1,2,…,D, means 0.25 to 5 miles
D	Total levels of distance (detector location), D = 7

r	Ordinal number of simulation flow-rate level, r = 1,2,…,R, means 240 to 2400 veh/h/ln
R	Total levels of simulation flow-rate, R = 9

Step 2: Capacity Adjustment Factor (CAF) Calculation

In Step 2 the capacity adjustment factor (CAF) is computed for each scenario using the auto-only flow scatter plots and the mixed flow plots obtained in Step 1. The capacity adjustment factor for a given mixed flow scenario ($CAF_{2,p,m,g,d}$) is defined as the ratio of the capacity of the mixed flow for that scenario ($C_{2,p,m,g,d}$) to the capacity of auto-only flow at corresponding grade and distance ($C_{1,0,0,g,d}$) for that scenario, as shown in the Equation 8.3. By default the capacity adjustment factor for the auto-only flow ($CAF_{1,0,0,g,d}$) is equal to one as shown in Equation 8.4. There are 3549 estimated capacity adjustment factors ($CAF_{2,p,m,g,d}$) in total, one for each of the scatter plots.

$$CAF_{2,p,m,g,d} = \frac{C_{2,p,m,g,d}}{C_{1,0,0,g,d}} \tag{8.3}$$

$$CAF_{1,0,0,g,d} = \frac{C_{1,0,0,g,d}}{C_{1,0,0,g,d}} = 1 \tag{8.4}$$

$CAF_{2,p,m,g,d}$	Capacity adjustment factor for the mixed flow at p truck percentage level, m truck composition level, g grade level, d distance level
$CAF_{1,0,0,g,d}$	Capacity adjustment factor for the auto-only flow at g grade level, d distance level

$C_{2,p,m,g,d}$	Capacity for the mixed flow at p truck percentage level, m truck composition level, g grade level, d distance level (veh/h/ln)
$C_{1,0,0,g,d}$	Capacity for the auto-only flow at g grade level, d distance level (veh/h/ln)
f	Ordinal number of flow type, f = 1 for auto-only flow, f = 2 for mixed flow
p	Ordinal number of truck percentage level. If f = 1, p = 0, means 0% truck percentage. If f = 2, p = 1,2,..., P, means 2% to 100% truck percentage.
P	Total levels of truck percentage, P = 13
m	Ordinal number of truck composition level, m = 0 for no trucks, m = 1 for 30% SUT/70% TT, m = 2 for 50% SUT/50% TT, m = 3 for 70% SUT/30% TT
g	Ordinal number of grade level, g = 1,2,…,G, means -6% to 6% grade
G	Total levels of grade, G = 13
d	Ordinal number of distance level (the level of detector location), d = 1,2,…,D, means 0.25 to 5 miles
D	Total levels of distance (detector location), D = 7

The capacity of auto-only flow at g grade level and d distance level ($C_{1,0,0,g,d}$), and the capacity of the mixed flow at p truck percentage level, m truck composition level, g grade level, d distance level ($C_{2,p,m,g,d}$), is defined as the maximum flow-rate of the 540 points in the scatter plot and this is identified using Equation 8.5:

$$C_{f,p,m,g,d} = \max_{\substack{t=1\ to\ 60 \\ q=1\ to\ 9}} \{q_{f,t,p,m,g,d,r}\} \tag{8.5}$$

$C_{f,p,m,g,d}$	Capacity for the mixed flow at p truck percentage level, m truck composition level, g grade level, d distance level (veh/h/ln)
$q_{f,t,p,m,g,d,r}$	Flow rate for the f flow type at t time interval, p truck percentage level, m truck composition level, g grade level, d distance level, and r simulation flow-rate level based on 1-min interval traffic volume recorded by the detector (veh/h/ln)
f	Ordinal number of flow type, f = 1 for auto-only flow, f = 2 for mixed flow
t	Ordinal number of 1-min time interval, t = 1,2,…,T
T	Total numbers of 1-min time interval, T = 60
p	Ordinal number of truck percentage level. If f = 1, p = 0, means 0% truck percentage. If f = 2, p = 1,2,..., P, means 2% to 100% truck percentage.
P	Total levels of truck percentage, P = 13

m	Ordinal number of truck composition level, m = 0 for no trucks, m = 1 for 30% SUT/70% TT, m = 2 for 50% SUT/50% TT, m = 3 for 70% SUT/30% TT
g	Ordinal number of grade level, g = 1,2,…,G, means -6% to 6% grade
G	Total levels of grade, G = 13
d	Ordinal number of distance level (the level of detector location), d = 1,2,…,D, means 0.25 to 5 miles
D	Total levels of distance (detector location), D = 7
r	Ordinal number of simulation flow-rate level, r = 1,2,…,R, means 240 to 2400 veh/h/ln
R	Total levels of simulation flow-rate, R = 9

Step 3: Development of Regression Models for Capacity Adjustment Factor (CAF) Estimation

Once the CAFs are obtained a series of regression models were developed that relate the CAF to truck percentage, truck composition, grade and distance. It should be noted that for each of the three levels of truck composition (m), one model was developed. In the other words the models for 30% SUT/70% TT, 50% SUT/50% TT, and 70% SUT/30% TT were developed separately. The HCM 2016 models were estimated and calibrated by the research team at North Carolina State University (97) and these results are shown in Equation 8.6 through 8.10. At here the exact same model form was used. However, the parameters in

Equation 8.6 through 8.10 were estimated and calibrated using the VISSIM simulation data in this research. Three models, one for each truck composition, were calibrated for the HCM 2016 conditions and three models were calibrated for the western rural U.S. conditions.

$$CAF_{2,p,m,g,d} = CAF_{1,0,0,g,d} - CAF_{2,p,m}^{T_a} - CAF_{2,p,m,g,d}^{G_a} - CAF_{2,p,m}^{FFS_a} \tag{8.6}$$

$$CAF_{2,p,m}^{T_a} = \alpha_{2,m}^{T_a} * (p_s)_p^{\beta_{2,m}^{T_a}} \tag{8.7}$$

$$\rho_{2,p,m}^{G_a} = \begin{cases} \gamma_{2,m}^{G_a} * (p_s)_p & if\ (p_s)_p < p^* \\ \theta_{2,m}^{G_a} - \mu_{2,m}^{G_a} * (p_s)_p & if\ (p_s)_p \geq p^* \end{cases} \tag{8.8}$$

$$CAF_{2,p,m,g,d}^{G_a} = \rho_{2,p,m}^{G_a} * max\left\{0, \alpha_{2,m}^{G_a} * \left[e^{\phi_{2,m}^{G_a}*(g_s)_g} - \eta_{2,m}^{G_a}\right]\right\} * max\left\{0, \beta_{2,m}^{D_a} * \left[1 - \alpha_{2,m}^{D_a} * e^{\phi_{2,m}^{D_a}*(d_s)_d}\right]\right\} \tag{8.9}$$

$$CAF_{2,p,m}^{FFS_a} = \mu_{2,m}^{FFS_a} * \left[1 - \rho_{2,m}^{FFS_a} * (p_s)_p^{\beta_{2,m}^{FFS_a}}\right] * [(70 - FFS_1)/100]^{\phi_{2,m}^{FFS_a}} \tag{8.10}$$

$CAF_{2,p,m,g,d}$	Capacity adjustment factor for the mixed flow at p truck percentage level, m truck composition level, g grade level, d distance level
$CAF_{1,0,0,g,d}$	Capacity adjustment factor for the auto-only flow at g grade level, d distance level. The value is always 1.
$CAF_{2,p,m}^{T_a}$	Capacity adjustment factor for truck percentage effect for the mixed flow at p truck percentage level, m truck composition level

$CAF_{2,p,m,g,d}^{G_a}$	Capacity adjustment factor for grade effect for the mixed flow at p truck percentage level, m truck composition level, g grade level, d distance level
$CAF_{2,p,m}^{FFS_a}$	Capacity adjustment factor for free-flow speed effect for the mixed flow at p truck percentage level, m truck composition level
$\rho_{2,p,m}^{G_a}$	Coefficient for capacity adjustment factor for grade effect for the mixed flow at p truck percentage level, m truck composition level
f	Ordinal number of flow type, f = 1 for auto-only flow, f = 2 for mixed flow
p	Ordinal number of truck percentage level. If f = 1, p = 0, means 0% truck percentage. If f = 2, p = 1,2,..., P, means 2% to 100% truck percentage.
P	Total levels of truck percentage, P = 13
m	Ordinal number of truck composition level, m = 0 for no trucks, m = 1 for 30% SUT/70% TT, m = 2 for 50% SUT/50% TT, m = 3 for 70% SUT/30% TT
g	Ordinal number of grade level, g = 1,2,…,G, means -6% to 6% grade

G	Total levels of grade, G = 13
d	Ordinal number of distance level, d = 1,2,…,D, means 0.25 to 5 miles
D	Total levels of distance, D = 7
$(p_s)_p$	Truck percentage at p truck percentage level (between 0 to 1)
p^*	Threshold of truck percentage for calculating coefficient for capacity adjustment factor related to grade with default value 0.01
$(g_s)_g$	Grade at g grade level (between -1 to 1)
$(d_s)_d$	Distance of grade at d distance level (mile)
FFS_1	Free-flow speed for auto-only flow (mph)

At here the parameters $\alpha_{2,m}^{T_a}$, $\beta_{2,m}^{T_a}$, $\gamma_{2,m}^{G_a}$, $\theta_{2,m}^{G_a}$, $\mu_{2,m}^{G_a}$, $\alpha_{2,m}^{G_a}$, $\phi_{2,m}^{G_a}$, $\eta_{2,m}^{G_a}$, $\alpha_{2,m}^{D_a}$, $\beta_{2,m}^{D_a}$, $\phi_{2,m}^{D_a}$, $\mu_{2,m}^{FFS_a}$, $\rho_{2,m}^{FFS_a}$, $\beta_{2,m}^{FFS_a}$, $\phi_{2,m}^{FFS_a}$ from Equation 8.6 to 8.10 were calibrated with a non-linear regression procedure that minimized the error between and the estimated CAFs and the simulated CAFs (98)(99).

Step 4: Capacity Adjustment Factor (CAF) Estimation for Specific Conditions

In step 4 the CAF is estimated for the mixed flow scenarios at specific truck percentage p_s, truck composition m_s, grade g_s, and distance d_s (CAF_{2,p_s,m_s,g_s,d_s}) , under HCM 2016 and western rural U.S. conditions, using the models developed in Step 3.

Step 5: EC_PCE Estimation

In step 5 the EC-PCE at specific truck percentage p_s, truck composition m_s, grade g_s, and distance d_s (PCE_{p_s,m_s,g_s,d_s}) is calculated. The EC_PCE is calculated using the Equation 8.12, which is derived from the Equation 8.11:

$$C_{1,g_s,d_s} = C_{2,p_s,m_s,g_s,d_s} * p_s * PCE_{p_s,m_s,g_s,d_s} + C_{2,p_s,m_s,g_s,d_s} * (1 - p_s) \tag{8.11}$$

$$PCE_{p_s,m_s,g_s,d_s} = \frac{1}{p_s} * \left(\frac{C_{1,g_s,d_s}}{C_{2,p_s,m_s,g_s,d_s}} - 1\right) + 1 = \frac{C_{1,g_s,d_s} - (1 - p_s) * C_{2,p_s,m_s,g_s,d_s}}{p_s * C_{2,p_s,m_s,g_s,d_s}} \tag{8.12}$$

PCE_{p_s,m_s,g_s,d_s}	EC-PCE for the mixed flow at truck percentage p_s, truck composition m_s, grade g_s, and distance d_s
C_{1,g_s,d_s}	Capacity for the auto-only flow at grade g_s, and distance d_s (veh/h/ln)
C_{2,p_s,m_s,g_s,d_s}	Capacity for the mixed flow at truck percentage p_s, truck composition m_s, grade g_s, and distance d_s (veh/h/ln)
f	Ordinal number of flow type, f = 1 for auto-only flow, f = 2 for mixed flow
p_s	Truck percentage (between 0 to 1)
m_s	Truck composition (percentage of single-unit and heavy trucks)
g_s	Grade (between -1 to 1)
d_s	Distance of grade (mile)

The CAF for truck percentage p_s, truck composition m_s, grade g_s, and distance d_s is calculated using Equation 8.13:

$$CAF_{2,p_s,m_s,g_s,d_s} = C_{2,p_s,m_s,g_s,d_s}/C_{1,g_s,d_s} \tag{8.13}$$

Therefore, the equation for PCE_{p_s,m_s,g_s,d_s} can be estimated using Equation 8.14:

$$EC\_PCE_{p_s,m_s,g_s,d_s} = \frac{1-(1-p)*CAF_{2,p_s,m_s,g_s,d_s}}{p*CAF_{2,p_s,m_s,g_s,d_s}} \tag{8.14}$$

$EC\_PCE_{p_s,m_s,g_s,d_s}$	EC-PCE for the mixed flow at truck percentage p_s, truck composition m_s, grade g_s, and distance d_s
CAF_{2,p_s,m_s,g_s,d_s}	Capacity adjustment factor for the mixed flow at truck percentage p_s, truck composition m_s, grade g_s, and distance d_s
p_s	Truck percentage (between 0 to 1)
m_s	Truck composition (percentage of single-unit and heavy trucks)
g_s	Grade (between -1 to 1)
d_s	Distance of grade (mile)

The CAF_{2,p_s,m_s,g_s,d_s} obtained in Step 4 is substituted into the Equation 8.14 and the PCE_{p_s,m_s,g_s,d_s} is calculated.

8.3 PCE Based on Equal-Capacity Method

8.3.1 EC_PCE Estimation Results

This section shows the results of each step in the EC_PCE estimation procedure, as well as the final recommended estimated EC_PCE values.

Figure 8.8 shows an example result of flow-density scatter plot for the auto-only flow at 1% grade, at a location 1.5 miles from the beginning of the grade under the HCM 2016 conditions. Figure 8.9 shows an example of a flow-density scatter plot for the mixed flow at 10% truck percentage, 30% SUT/70% TT truck composition, 1% grade, at a location from the start of the grade 1.5 miles distance under the HCM 2016 conditions. As described previously there are 540 data points on each scatter plot. The two figures show that, the maximum flow rate of the mixed flow (e.g. 2394 veh/h/ln) is lower than the maximum flow rate of the auto-only (e.g. 2755 veh/h/ln) flow at the same grade and distance. This is not surprising given that the presence of trucks would be expected to reduce the observed maximum flow rate (e.g. capacity). The examples of flow-density scatter plots for the auto-only flow (or mixed flow, as appropriate) at each combination of truck percentage, truck composition, grade, and distance are shown in the appendix.

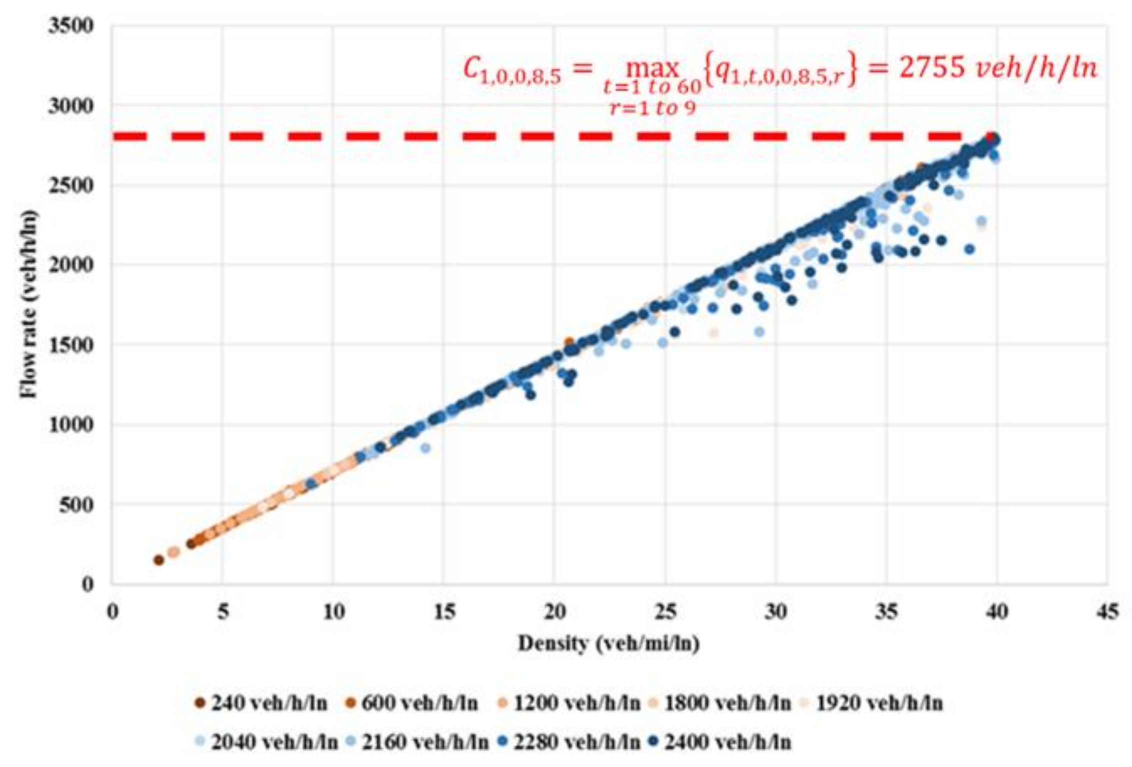


Figure 8.8 Flow-density scatter plots for the auto-only flow at 1% grade, 1.5 miles distance under HCM 2016 conditions

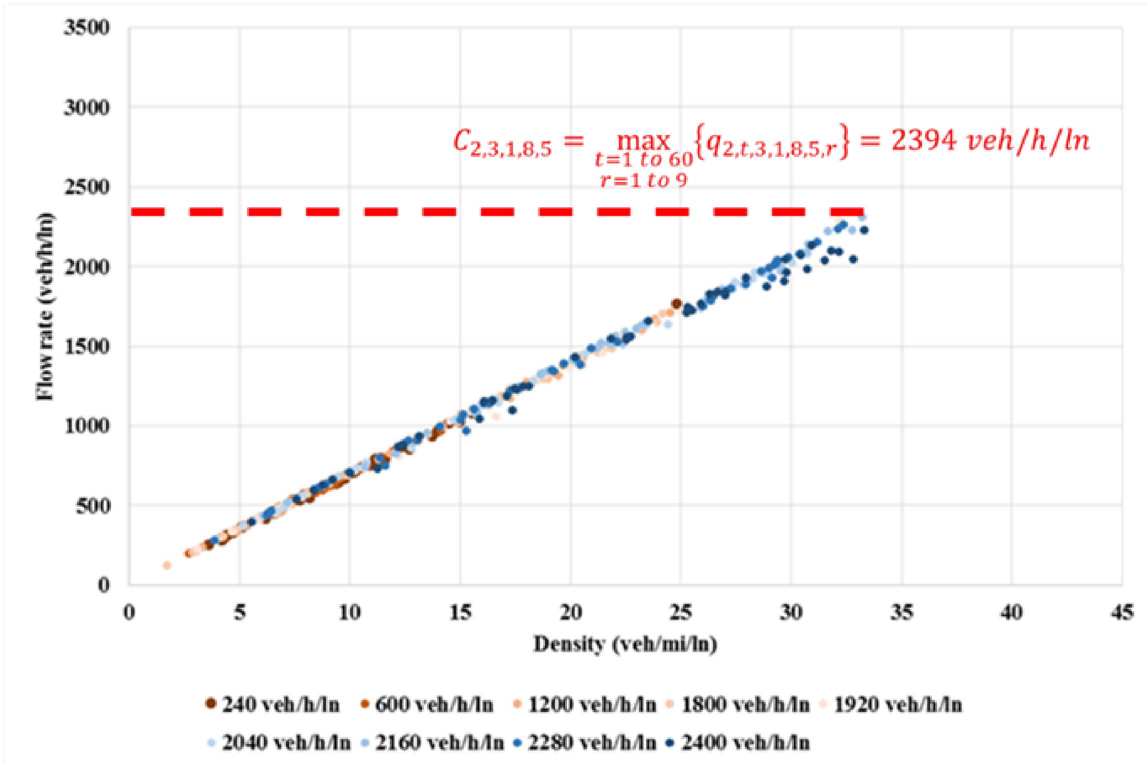


Figure 8.9 Flow-density scatter plots for the mixed flow at 10% truck percentage, 30% SUT/70% TT truck composition, 1% grade, 1.5 miles distance under HCM 2016 conditions

Using the example in Figure 8.8, under the HCM 2016 conditions, the capacity for the auto-only flow on the 1% grade, at a point 1.5 miles from the start of the grade is 2755 veh/h/ln ($C_{1,0,0,8,5}$ = 2755 veh/h/ln). As shown in Figure 8.9, under the HCM 2016 conditions, the capacity for the mixed flow at 10% truck percentage, 30% SUT/70% TT, on the 1% grade, at a location 1.5 miles from the start of the grade is 2394 veh/h/ln ($C_{2,3,1,8,5}$ = 2394 veh/h/ln). Thus, the CAF for this scenario $CAF_{2,3,1,8,5}$ is 0.869 (e.g. 2394/2755).

Figure 8.10, Figure 8.11, and Figure 8.12 show all 3549 computed simulated CAFs for SUT/TT truck percentage ratios of 30% SUT/70% TT, 50% SUT/50% TT, and 70% SUT/30% TT, respectively. The scenario number is calculated using the Equation 8.15. The sorting order for the scenario number is truck percentage, grade and distance. Note the CAFs for both the HCM 2016 and western rural U.S. conditions are shown. In general the CAFs under the HCM 2016 conditions are 30 percent higher than those under the western rural U.S. conditions.

$$n = 91 * p + (g - 1) * 7 + d \tag{8.15}$$

n	Scenario number
p	Ordinal number of truck percentage level, p = 1,2,..., P, means 2% to 100% truck percentage.
P	Total levels of truck percentage, P = 13
g	Ordinal number of grade level, g = 1,2,…,G, means -6% to 6% grade
G	Total levels of grade, G = 13
d	Ordinal number of distance level (the level of detector location), d = 1,2,…,D, means 0.25 to 5 miles
D	Total levels of distance (detector location), D = 7

For example, the scenario number for 2% truck percentage, -4% grade, 0.75 miles distance is 108 (e.g. 1*91+(3-1)*7+3). Note that the red lines in the Figure 8.10, Figure 8.11, and Figure 8.12 are the estimated CAFs using the estimation models developed in Step 3.

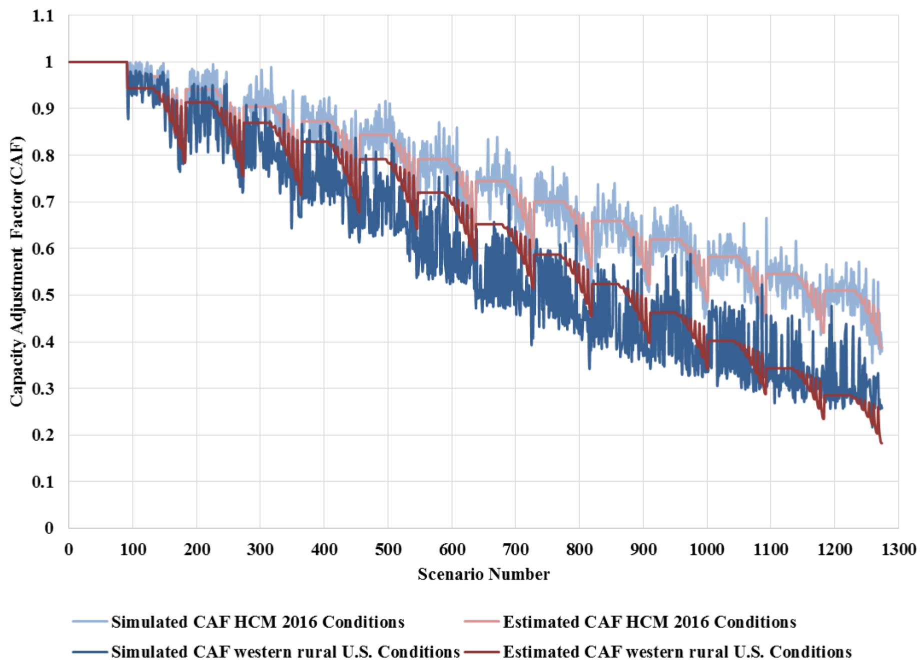


Figure 8.10 Simulated and estimated CAF for 30% SUT/70% TT truck compositions

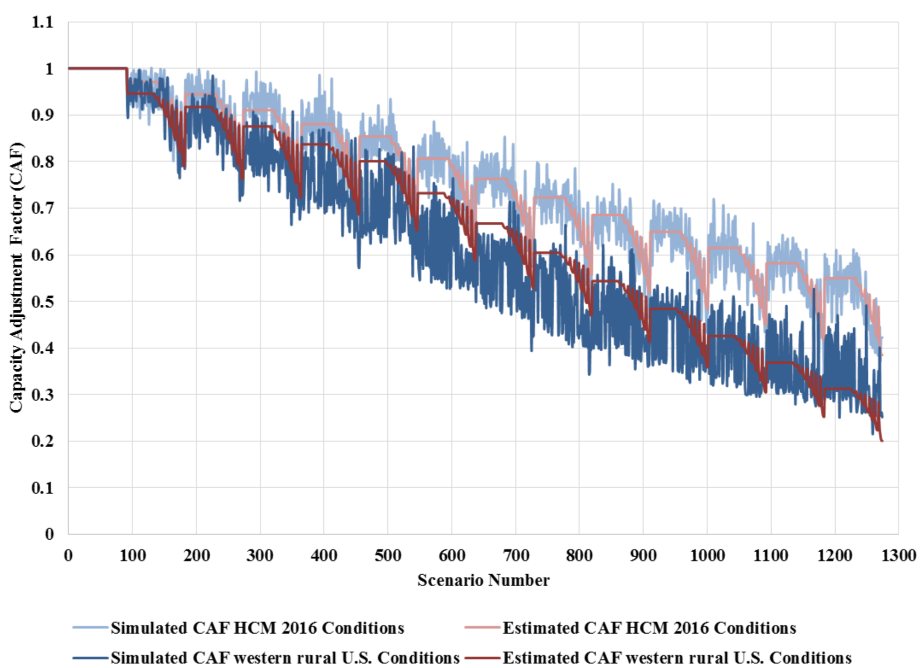


Figure 8.11 Simulated and estimated CAF for 50% SUT/50% TT truck compositions

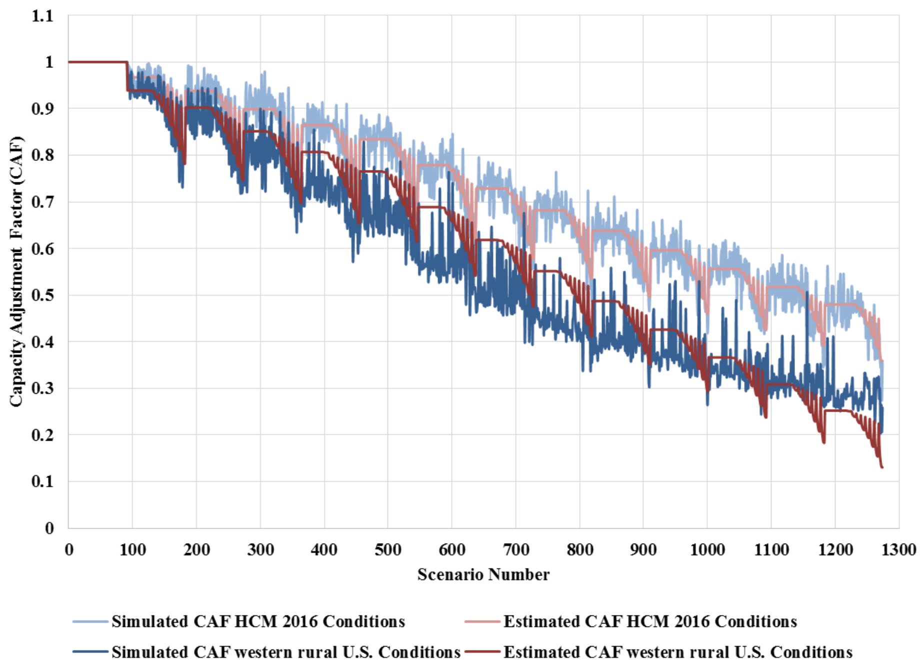


Figure 8.12 Simulated and estimated CAF for 70% SUT/30% TT truck compositions

Table 8.2 shows the estimated values of parameters, and it may be seen that the calibrated parameters based on VISSIM data under the HCM 2016 conditions were close to the HCM 2016 original parameters as evidenced by the fact that they were all within three percent of each other. In contrast the calibrated parameters under the western rural U.S. conditions were considerably different from the HCM 2016 original parameters as evidenced by the fact that they had, on average, a 48 percent difference.

Table 8.2 Parameters in the models for CAF estimation

Parameters	HCM 2016			Group 1 (HCM 2016 conditions)			Group 2 (Western rural U.S. conditions)		
	3S7T*	5S5T*	7S3T*	3S7T*	5S5T*	7S3T*	3S7T*	5S5T*	7S3T*
$\alpha_{2,m}^{T_a}$	0.530	0.490	0.470	0.522	0.496	0.474	0.747	0.674	0.644
$\beta_{2,m}^{T_a}$	0.720	0.710	0.730	0.707	0.701	0.723	0.700	0.849	0.856
$\gamma_{2,m}^{G_a}$	8.000	8.000	8.000	8.000	8.000	8.000	8.000	8.000	8.000
$\theta_{2,m}^{G_a}$	0.126	0.137	2.110	0.130	0.127	2.117	0.124	0.137	0.137
$\mu_{2,m}^{G_a}$	0.030	0.030	0.010	0.036	0.032	0.009	0.036	0.047	0.020
$\alpha_{2,m}^{G_a}$	0.690	0.590	0.160	0.622	0.583	0.151	0.753	0.712	0.725
$\phi_{2,m}^{G_a}$	12.900	13.460	13.600	13.672	13.330	13.623	11.580	11.829	11.343
$\eta_{2,m}^{G_a}$	1.000	1.030	1.000	1.025	0.975	0.986	0.831	0.923	0.923
$\alpha_{2,m}^{D_a}$	1.710	1.530	1.240	1.780	1.512	1.223	1.800	1.447	1.220
$\beta_{2,m}^{D_a}$	1.720	1.600	0.390	1.637	1.684	0.406	1.438	1.493	1.659
$\phi_{2,m}^{D_a}$	-3.160	-3.280	-2.800	-3.426	-3.267	-2.723	-2.851	-3.219	-2.554
$\mu_{2,m}^{FFS_a}$	0.250	0.250	0.250	0.250	0.250	0.250	-0.218	-0.388	-0.371
$\rho_{2,m}^{FFS_a}$	0.700	0.700	0.700	0.700	0.700	0.700	-0.145	-0.779	-1.052
$\beta_{2,m}^{FFS_a}$	1.000	1.000	1.000	1.000	1.000	1.000	0.200	0.288	1.404
$\phi_{2,m}^{FFS_a}$	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
R^2	0.970	0.960	0.950	0.966	0.951	0.938	0.932	0.878	0.859

*The 3S7T means 30% SUT/70% TT, 5S5T means 50% SUT/50% TT, 7S3T means 70% SUT/30% TT.

Because this research focus on level freeway segments with high truck percentage, the CAFs were estimated for 0% grade and 5% to 85% truck percentage. The CAF estimation results are shown in Table 8.3 to Table 8.5. The table shows the CAFs estimated by the developed models calibrated under HCM 2016 conditions were close to the HCM 2016 CAF values with an average difference of 0.7 percent. However, the CAFs estimated under Western rural U.S. conditions were considerably lower than the HCM 2016 CAF values, with an average difference of 17.4%. A t-test was conducted for each pair and the results were all statistically significant of the 95 percent level of confidence (95).

Table 8.3 Estimated CAFs for different truck percentage and distance at 0% grade at 30% SUT, 70% TT

Group	**Length (mi)**	**Percentage of Trucks (%)**								
		5%	15%	25%	35%	45%	55%	65%	75%	85%
HCM 2016	0.125	0.939	0.865	0.805	0.747	0.696	0.652	0.613	0.579	0.548
	0.375	0.939	0.865	0.805	0.747	0.696	0.652	0.613	0.579	0.548
	0.625	0.939	0.865	0.805	0.747	0.696	0.652	0.613	0.579	0.548
	0.875	0.939	0.865	0.805	0.747	0.696	0.652	0.613	0.579	0.548
	1.25	0.939	0.865	0.805	0.747	0.696	0.652	0.613	0.579	0.548
	1.5	0.939	0.865	0.805	0.747	0.696	0.652	0.613	0.579	0.548
Group 1 (HCM 2016 conditions)	0.125	0.937	0.863	0.804	0.751	0.703	0.658	0.615	0.574	0.535
	0.375	0.937	0.863	0.804	0.751	0.703	0.658	0.615	0.574	0.535
	0.625	0.937	0.863	0.804	0.751	0.703	0.658	0.615	0.574	0.535
	0.875	0.937	0.863	0.804	0.751	0.703	0.658	0.615	0.574	0.535
	1.25	0.937	0.863	0.804	0.751	0.703	0.658	0.615	0.574	0.535
	1.5	0.937	0.863	0.804	0.751	0.703	0.658	0.615	0.574	0.535
Group 2 (Western rural U.S. conditions)	0.125	0.896	0.790	0.705	0.630	0.561	0.496	0.435	0.377	0.321
	0.375	0.888	0.782	0.697	0.622	0.553	0.489	0.428	0.370	0.314
	0.625	0.881	0.775	0.690	0.615	0.547	0.483	0.422	0.364	0.309
	0.875	0.877	0.772	0.687	0.612	0.544	0.480	0.419	0.362	0.306
	1.25	0.875	0.769	0.685	0.610	0.542	0.478	0.418	0.360	0.305
	1.5	0.875	0.769	0.684	0.610	0.541	0.478	0.417	0.360	0.304

Table 8.4 Estimated CAFs for different truck percentage and distance at 0% grade at 50% SUT, 50% TT

Group	Length (mi)	Percentage of Trucks (%)								
		5%	15%	25%	35%	45%	55%	65%	75%	85%
HCM 2016	0.125	0.939	0.867	0.811	0.754	0.705	0.662	0.623	0.589	0.559
	0.375	0.939	0.867	0.811	0.754	0.705	0.662	0.623	0.589	0.559
	0.625	0.939	0.867	0.811	0.754	0.705	0.662	0.623	0.589	0.559
	0.875	0.939	0.867	0.811	0.754	0.705	0.662	0.623	0.589	0.559
	1.25	0.939	0.867	0.811	0.754	0.705	0.662	0.623	0.589	0.559
	1.5	0.939	0.867	0.811	0.754	0.705	0.662	0.623	0.589	0.559
Group 1 (HCM 2016 conditions)	0.125	0.939	0.869	0.812	0.762	0.717	0.674	0.633	0.595	0.557
	0.375	0.938	0.867	0.811	0.761	0.715	0.672	0.632	0.593	0.556
	0.625	0.937	0.866	0.810	0.760	0.714	0.672	0.631	0.593	0.555
	0.875	0.936	0.866	0.810	0.760	0.714	0.671	0.631	0.592	0.555
	1.25	0.936	0.866	0.809	0.760	0.714	0.671	0.631	0.592	0.555
	1.5	0.936	0.866	0.809	0.760	0.714	0.671	0.631	0.592	0.555
Group 2 (Western rural U.S. conditions)	0.125	0.921	0.837	0.762	0.693	0.626	0.562	0.499	0.438	0.379
	0.375	0.915	0.831	0.757	0.687	0.621	0.557	0.495	0.434	0.375
	0.625	0.912	0.829	0.754	0.685	0.619	0.555	0.493	0.432	0.373
	0.875	0.911	0.827	0.753	0.684	0.618	0.554	0.492	0.431	0.372
	1.25	0.911	0.827	0.753	0.683	0.617	0.553	0.491	0.431	0.371
	1.5	0.910	0.827	0.753	0.683	0.617	0.553	0.491	0.431	0.371

Table 8.5 Estimated CAFs for different truck percentage and distance at 0% grade at 70% SUT, 30% TT

Group	Length (mi)	Percentage of Trucks (%)								
		5%	15%	25%	35%	45%	55%	65%	75%	85%
HCM 2016	0.125	0.947	0.882	0.828	0.775	0.728	0.687	0.650	0.616	0.586
	0.375	0.947	0.882	0.828	0.775	0.728	0.687	0.650	0.616	0.586
	0.625	0.947	0.882	0.828	0.775	0.728	0.687	0.650	0.616	0.586
	0.875	0.947	0.882	0.828	0.775	0.728	0.687	0.650	0.616	0.586
	1.25	0.947	0.882	0.828	0.775	0.728	0.687	0.650	0.616	0.586
	1.5	0.947	0.882	0.828	0.775	0.728	0.687	0.650	0.616	0.586
Group 1 (HCM 2016 conditions)	0.125	0.945	0.880	0.826	0.778	0.734	0.692	0.653	0.615	0.578
	0.375	0.945	0.879	0.825	0.777	0.733	0.691	0.652	0.614	0.578
	0.625	0.944	0.878	0.825	0.777	0.732	0.691	0.651	0.614	0.577
	0.875	0.944	0.878	0.824	0.776	0.732	0.691	0.651	0.613	0.577
	1.25	0.944	0.878	0.824	0.776	0.732	0.691	0.651	0.613	0.577
	1.5	0.944	0.878	0.824	0.776	0.732	0.691	0.651	0.613	0.577
Group 2 (Western rural U.S. conditions)	0.125	0.930	0.852	0.781	0.713	0.649	0.586	0.524	0.464	0.404
	0.375	0.925	0.847	0.776	0.708	0.644	0.581	0.519	0.459	0.400
	0.625	0.922	0.844	0.773	0.706	0.641	0.578	0.517	0.456	0.397
	0.875	0.921	0.842	0.771	0.704	0.640	0.577	0.515	0.455	0.396
	1.25	0.920	0.841	0.770	0.703	0.639	0.576	0.514	0.454	0.395
	1.5	0.919	0.841	0.770	0.703	0.638	0.576	0.514	0.454	0.395

The CAF_{2,p_s,m_s,g_s,d_s} in Table 8.3 to Table 8.5 is substituted into the Equation 8.14 and the PCE_{p_s,m_s,g_s,d_s} is calculated. The EC_PCE estimation results at different truck percentage, truck composition and distance at 0% grade for each group are shown in Table 8.6, Table 8.7, and Table 8.8 for the 30% SUT/70% TT, 50% SUT/70% TT, and 70% SUT/30% TT, respectively. For ease of analysis these tables are also recreated in Figure 8.13, Figure 8.14, Figure 8.15, respectively.

Table 8.6 Equal-capacity-based PCE estimation results at level freeway segments (0% Grade) with 30% SUT, 70% TT

Group	**Length (mi)**	**Percentage of Trucks (%)**								
		5%	15%	25%	35%	45%	55%	65%	75%	85%
HCM 2016	0.125	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	0.375	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	0.625	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	0.875	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	1.25	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	1.5	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
Group 1 (Original Research Conditions)	0.125	2.34	2.05	1.97	1.94	1.94	1.95	1.96	1.99	2.02
	0.375	2.34	2.05	1.97	1.94	1.94	1.95	1.96	1.99	2.02
	0.625	2.34	2.05	1.97	1.94	1.94	1.95	1.96	1.99	2.02
	0.875	2.34	2.05	1.97	1.94	1.94	1.95	1.96	1.99	2.02
	1.25	2.34	2.05	1.97	1.94	1.94	1.95	1.96	1.99	2.02
	1.5	2.34	2.05	1.97	1.94	1.94	1.95	1.96	1.99	2.02
Group 2 (Western rural U.S. conditions)	0.125	3.31	2.77	2.68	2.68	2.74	2.85	3.00	3.20	3.49
	0.375	3.52	2.86	2.74	2.74	2.80	2.90	3.06	3.27	3.57
	0.625	3.70	2.94	2.80	2.79	2.84	2.95	3.10	3.32	3.63
	0.875	3.79	2.97	2.82	2.81	2.86	2.97	3.13	3.35	3.66
	1.25	3.85	3.00	2.84	2.82	2.88	2.99	3.15	3.37	3.68
	1.5	3.87	3.00	2.85	2.83	2.88	2.99	3.15	3.37	3.69

Table 8.7 Equal-capacity-based PCE estimation results at level freeway segments (0% Grade) with 50% SUT, 50% TT

Group	**Length (mi)**	**Percentage of Trucks (%)**								
		5%	15%	25%	35%	45%	55%	65%	75%	85%
HCM 2016	0.125	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	0.375	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	0.625	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	0.875	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	1.25	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	1.5	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
Group 1 (Original Research Conditions)	0.125	2.29	2.01	1.92	1.89	1.88	1.88	1.89	1.91	1.93
	0.375	2.33	2.02	1.93	1.90	1.89	1.89	1.90	1.91	1.94
	0.625	2.35	2.03	1.94	1.90	1.89	1.89	1.90	1.92	1.94
	0.875	2.36	2.03	1.94	1.90	1.89	1.89	1.90	1.92	1.94
	1.25	2.36	2.03	1.94	1.90	1.89	1.89	1.90	1.92	1.94
	1.5	2.36	2.03	1.94	1.90	1.89	1.89	1.90	1.92	1.94
Group 2 (Western rural U.S. conditions)	0.125	2.72	2.30	2.25	2.27	2.33	2.42	2.54	2.71	2.93
	0.375	2.86	2.35	2.28	2.30	2.36	2.45	2.57	2.74	2.96
	0.625	2.92	2.38	2.30	2.31	2.37	2.46	2.58	2.75	2.98
	0.875	2.95	2.39	2.31	2.32	2.37	2.46	2.59	2.76	2.99
	1.25	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	1.5	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99

Table 8.8 Equal-capacity-based PCE estimation results at level freeway segments (0% Grade) with 70% SUT, 30% TT

Group	**Length (mi)**	**Percentage of Trucks (%)**								
		5%	15%	25%	35%	45%	55%	65%	75%	85%
HCM 2016	0.125	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
	0.375	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
	0.625	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
	0.875	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
	1.25	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
	1.5	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
Group 1 (Original Research Conditions)	0.125	2.15	1.91	1.84	1.82	1.81	1.81	1.82	1.84	1.86
	0.375	2.17	1.92	1.85	1.82	1.81	1.81	1.82	1.84	1.86
	0.625	2.18	1.92	1.85	1.82	1.81	1.81	1.82	1.84	1.86
	0.875	2.19	1.93	1.85	1.82	1.81	1.81	1.82	1.84	1.86
	1.25	2.19	1.93	1.85	1.82	1.81	1.81	1.82	1.84	1.86
	1.5	2.19	1.93	1.85	1.82	1.81	1.81	1.82	1.84	1.86
Group 2 (Western rural U.S. conditions)	0.125	2.50	2.16	2.12	2.15	2.20	2.29	2.40	2.54	2.73
	0.375	2.62	2.21	2.16	2.18	2.23	2.31	2.42	2.57	2.77
	0.625	2.69	2.23	2.18	2.19	2.24	2.33	2.44	2.59	2.79
	0.875	2.72	2.25	2.18	2.20	2.25	2.33	2.45	2.60	2.80
	1.25	2.75	2.26	2.19	2.20	2.26	2.34	2.45	2.60	2.80
	1.5	2.76	2.26	2.19	2.21	2.26	2.34	2.45	2.60	2.80

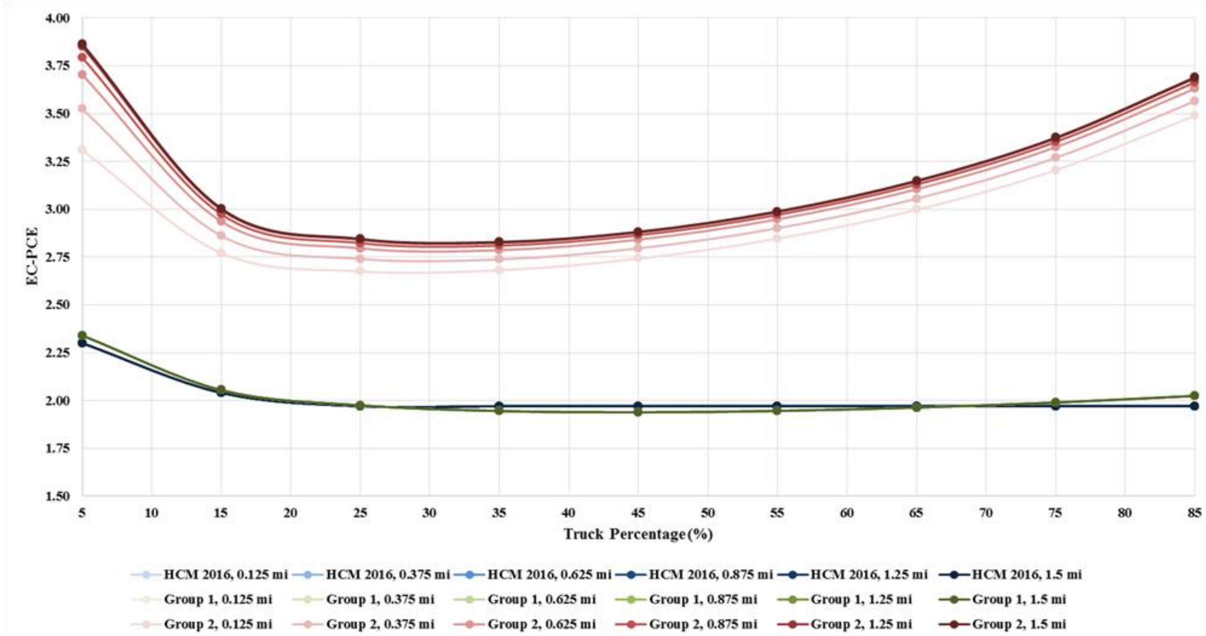


Figure 8.13 Equal-capacity-based PCE estimation results for 30% SUT, 70% TT, 0% grade

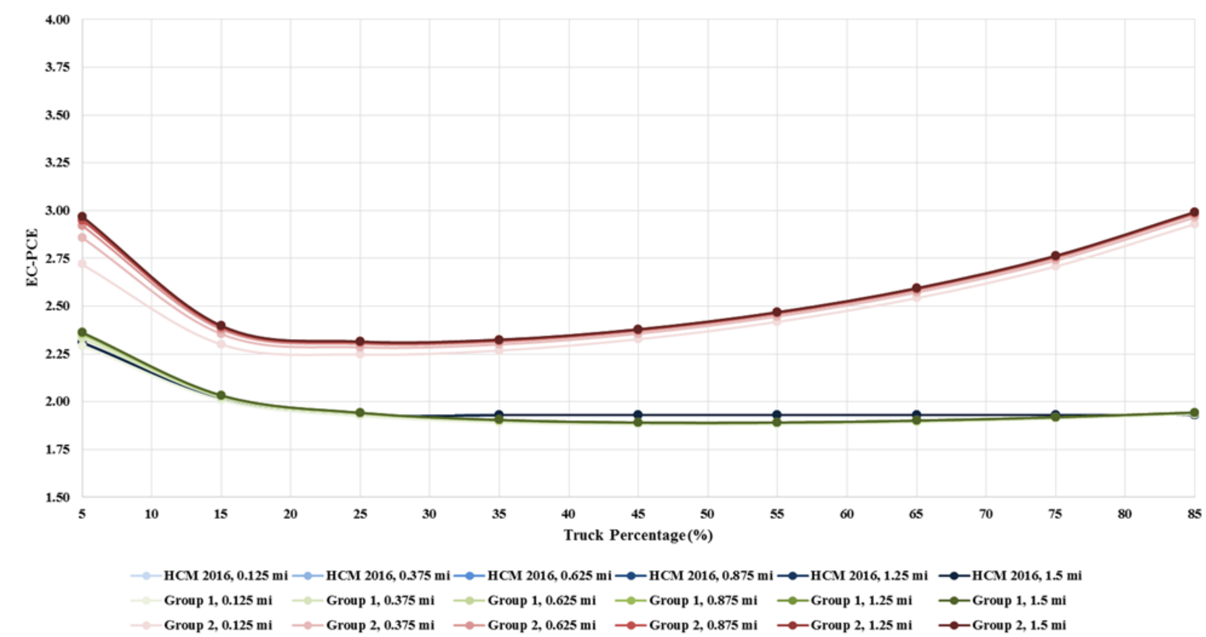


Figure 8.14 Equal-capacity-based PCE estimation results for 50% SUT, 50% TT, 0% grade

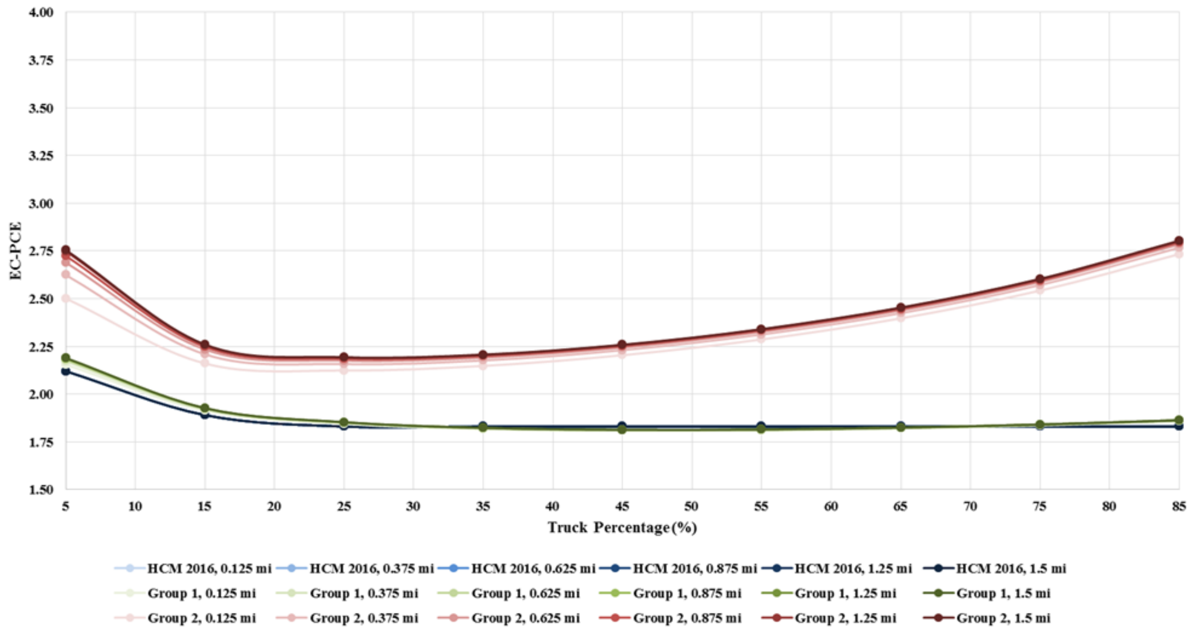


Figure 8.15 Equal-capacity-based PCE estimation results for 70% SUT, 30% TT, 0% grade

Table 8.6 and Figure 8.13 show that, at 30% SUT/70% TT, the EC_PCEs range from 1.94 to 2.34 at the HCM 2016 condition, ranging from 2.68 to 3.87 at the western rural U.S. condition. Table 8.7 and Figure 8.14 show that, at 50% SUT/50% TT, the EC_PCEs range from 1.88 to 2.36 at the HCM 2016 condition, ranging from 2.25 to 2.99 at the western rural U.S. condition. Table 8.8 and Figure 8.15 show that, at 70% SUT/30% TT, the EC_PCEs range from 1.81 to 2.19 at the HCM 2016 condition, ranging from 2.12 to 2.80 at the western rural U.S. condition. The three tables and figures show that, the EC_PCEs at all compositions of single-unit trucks (SUT) and heavy trucks (TT):

1) The EC_PCE values at the western rural U.S. conditions are 20 to 70 percent higher than that in HCM 2016 conditions;
2) The EC_PCEs increase with the percentage of heavy trucks;
3) Under western rural U.S. conditions, the EC_PCEs increase with grade length;

4) Under western rural U.S. conditions, the EC_PCEs decrease with the truck percentage up to 25 percent, then slightly increase with the truck percentage.

8.3.2 Influencing Factor Analysis

According to Table 8.6 to Table 8.8, it was found that the EC_PCEs under western rural U.S. conditions were 20 to 70 percent higher than the HCM 2016 conditions. These results were found to be statistically significant based on the t-tests at the 95 percent level of confidence (95). It is hypothesized that these differences are as a result of:

1) The differential free-flow speeds distributions between passenger cars and trucks on the level terrain cause the vehicles tending to form moving bottlenecks;
2) The freeway segments with two lanes per direction under the western rural U.S. conditions make vehicles passing more difficult than the freeway segments with three lanes per direction; and
3) The truck lengths under the western rural U.S. conditions are longer than the HCM 2016 conditions.

The results also show that under either the HCM 2016 or western rural U.S. conditions, the EC_PCEs increase with the percentage of heavy trucks. This is consistent with the HCM 2016. It should be noted that the heavy trucks generally have higher weight-to-horsepower ratios than the single-unit trucks and this may result in heavy trucks affecting the capacity more than the single-unit trucks. It is also found that, under the western rural U.S. conditions, the EC_PCEs increase with the grade length, decrease with the truck percentage up to 25 percent then slightly increase with the truck percentage. The results indicate that the capacity may be affected by the differential free-flow speeds between trucks and passenger cars on long segments with high truck percentage.

8.4 Comparison between EC_PCEs and the Recommended PCEs in HCM 2016

As shown in the Table 8.6 to Table 8.8 and Figure 8.13 to Figure 8.15, using the 2016 HCM PCE estimation methodology and VISSIM 9.0, the HCM 2016 PCE values were replicated. It may be seen that there was, on average, a 3% difference in the results. It is hypothesized that this difference is a result of the use of different VISSIM models (e.g. version 9.0 v.s. version 4.4) or different random seeds number.

It is hypothesized that the EC_PCEs calculated in this paper better capture the relationship between trucks and passenger cars under moving-bottleneck conditions. For each of the data collection sites it is found that the observed speeds were 0 to 20 percent lower than that which is to be expected from the HCM 2016 for the same flow-rate (pcu/h/ln), if 1) the speed-flow curve for FFS = 75 mph (Exhibit 12-7 in HCM 2016) is selected for analysis and 2) the PCEs recommended in HCM 2016 are used for converting the mixed flow to the passenger-car-only flow, as shown in Figure 8.16. It may be seen in Figure 8.16 that the observed speeds correspond to the LOS C or D section on the curve.

If the larger EC_PCEs proposed in this study (Group 2) are used, the estimated LOS will change, as shown in Figure 8.17. The comparison of LOS results between these two conditions is shown in Figure 8.18. It can be seen that if the HCM 2016 recommended PCEs are used, the results would be 40 percent LOS A, 43 percent LOS B, and 17 percent LOS C. However, if the proposed EC_PCEs (Group 2) are used, with the same operational analysis procedures, the results would be 20 percent LOS A, 27 percent LOS B, 29 percent LOS C, 22 percent LOS D, and one percent LOS E, which means that, under moving bottlenecks conditions, the drivers will feel that the roadway is much more congested than would be expected under current HCM procedures.

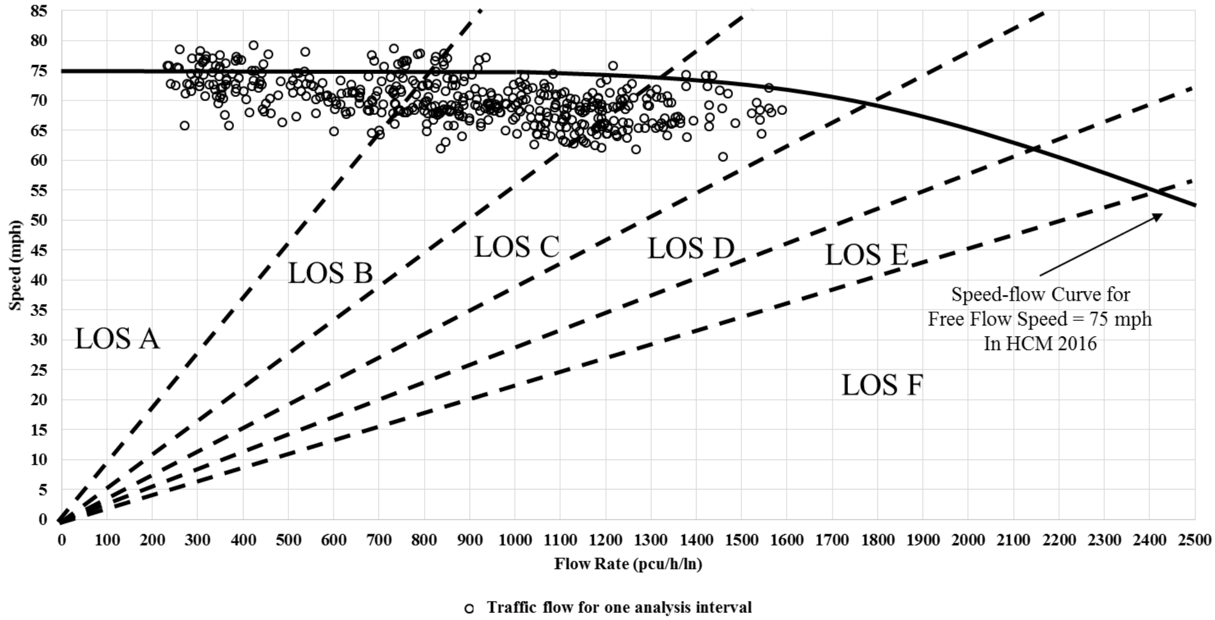


Figure 8.16 The observed traffic flow, FFS = 75 mph speed-flow curve in HCM 2016, and the operational analysis results with PCE for level freeway segment in HCM 2016

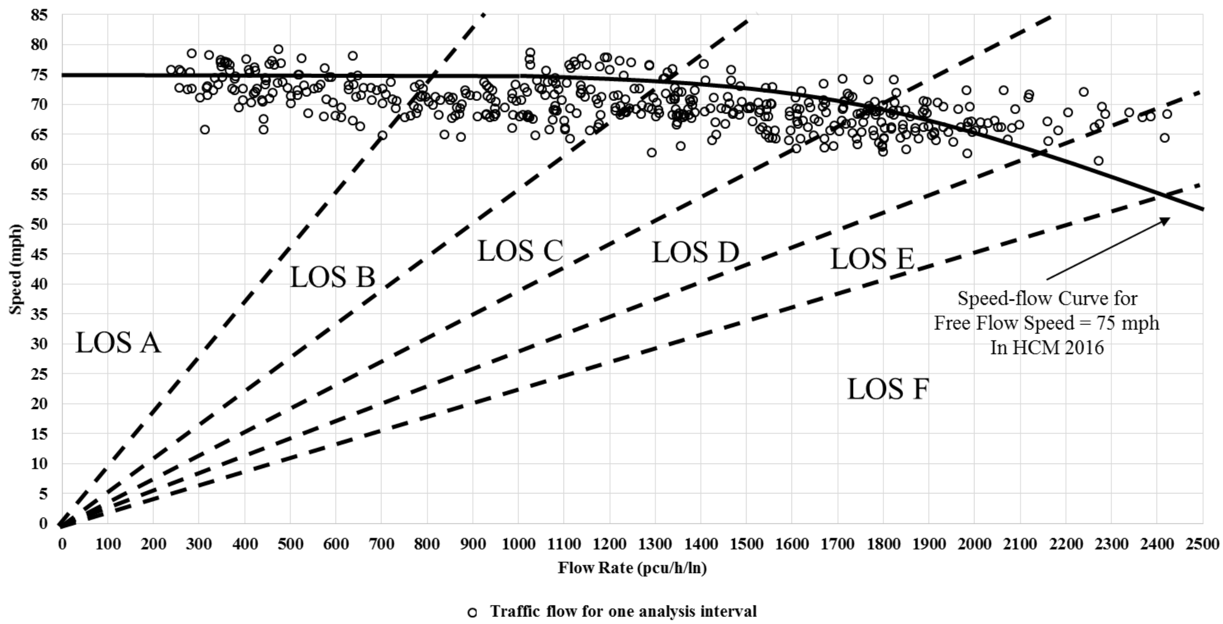


Figure 8.17 The observed traffic flow, FFS = 75 mph speed-flow curve in HCM 2016, and the operational analysis results with proposed EC_PCEs (western rural U.S. conditions)

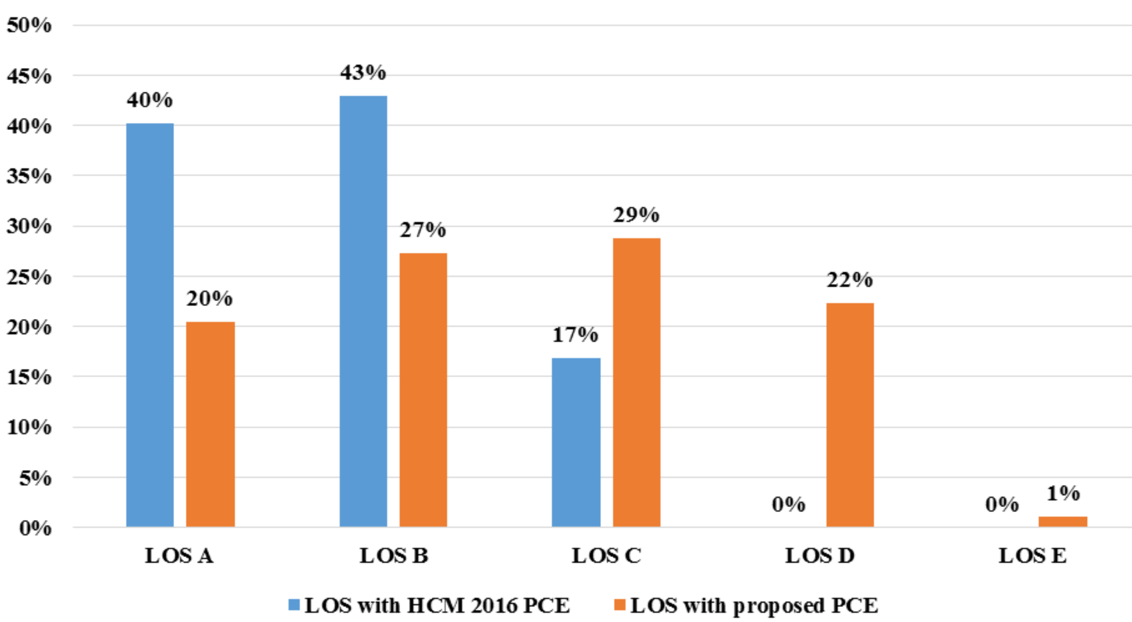


Figure 8.18 Comparison of LOS results between using recommended PCEs in HCM 2016 and proposed EC_PCEs (one-minute interval, western rural U.S. condition)

8.5 Applicable Conditions for EC_PCEs and the Recommended PCEs in HCM 2016

According to the analyses above, the EC_PCEs under western rural U.S. conditions are significantly higher than EC_PCEs under HCM 2016 conditions and the recommended PCEs in HCM 2016, due to the localized congestions caused by moving bottlenecks. Because the HCM 2016 implies all analyses for basic freeway segments are "applied to segments with uniform characteristics", it is hypothesized the HCM 2016 recommended PCEs may not be applicable to the conditions with localized congestion but the EC_PCEs estimated in this chapter may be appropriate. Also, because the PCEs are latest updated in HCM 2016, it is practically meaningful to explore the applicable conditions for HCM 2016 recommended PCEs. In section 6.4 a metric "Density uniformity factor" (DUF) is proposed to measure the localized congestion. This section analyzes the difference in localized congestion between HCM 2016 and western rural U.S. conditions, and discusses the applicable conditions for the HCM 2016 recommended PCEs and the new estimated EC_PCEs based on DUF using simulation data.

The VISSIM 9.0 simulation model developed for EC_PCEs estimation in section 8.1 is used for capturing localized congestion as the following and illustrated by Figure 8.19:

1) The simulation road network is a 15-mile uni-direction simulation freeway consisting of three sections, which are an 8-mile section used for loading vehicles followed by a 6-mile section used for data collection and an 1 mile section;
2) The simulation is running with 0% grade, since this research focuses on the moving bottlenecks and localized congestions on level freeway segments;

3) 24 point detectors are deployed at 6-mile data collection sections with equal distance (Δs_j = 0.25 miles), beginning from 0 mileposts and ending at 5.75 mileposts, used for collecting raw data (e.g. traffic volume, vehicle speeds, vehicle types, and timestamps of vehicle entering and leaving detectors, etc.);
4) Two simulation conditions: HCM 2016 conditions and western rural U.S. conditions. For these two conditions, the different parameters are lane numbers, truck lengths, desired speed distributions and car following parameters, which have the same settings as in Table 8.1;
5) The other parameters for geometrics, vehicles, traffic flows, driver behaviors, data collection and simulation procedure under the HCM 2016 conditions have the same values as western rural U.S. conditions. The details can be found in Table 8.1.
6) The simulation running lasts three hours. The first hour (0 to 3600s) aims at loading vehicles into the road network. The second hour (3600s to 7200s) aims at data collection. The last hour (7200s to 10800s) aims at unloading vehicles.

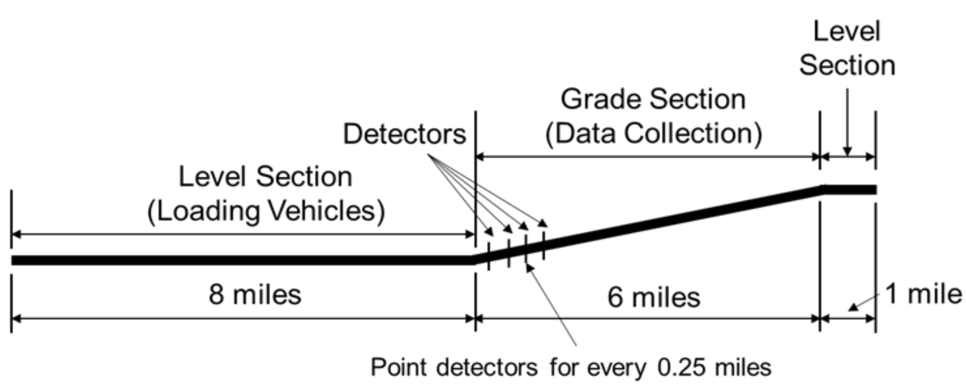


Figure 8.19 Simulation model for capturing localized congestion

To prove the difference in localized congestions between the HCM 2016 conditions and western rural U.S. conditions, a traffic flow with 1200 veh/h/ln traffic flow rate, 60% truck percentage, 30% single-unit trucks and 70% heavy trucks, and 6 miles distance is simulated for the two conditions. After data collection, the traffic volume ($V_{i,j}$) and average speed ($\bar{v}_{i,j}$) in each time period i at freeway segment j can be obtained from the raw data of point detectors. The traffic flow rate ($q_{i,j}$) in each time period i at freeway segment j is calculated by Equation 6.8. The 6-mile data collection freeway segment is divided into 24 segments due to 0.25 miles distance between detectors (Δs_j = 0.25 miles), which means there are 24 levels (S = 24) for freeway segments in total. The number of freeway segment ranges between 1 and 24 (j = 1, 2 to 24), corresponding to 0-0.25 miles to 5.75-6 miles with 0.25 miles as interval. The traffic flow rate ($q_{i,j}$) and average speed ($\bar{v}_{i,j}$) are analyzed based on 1-minute interval data (Δt_i = 60 seconds). Thus, there are 60 levels for time intervals in total (T = 60). The number of time interval ranges between 1 and 60 (i = 1, 2, to 60), corresponding to 0-1 minute to 59-60 minute with 1 minute as interval.

Combining the number of levels for freeway segments and time intervals, 1,440 traffic flow rates and average speeds can be obtained. Then, the density ($k_{i,j}$) at time period i at freeway segment j is calculated using Equation 6.9.

The heat map for the density at time period i at freeway segment j ($k_{i,j}$) under HCM 2016 conditions and western rural U.S. conditions are shown in Figure 8.20 and Figure 8.21. The x-axis and y-axis represent the time period i and freeway segment j in the heat map. The color in each cell represents the value of density. A dark cell represents a high density. The figures show that, under HCM 2016 conditions, the densities in all cells are lower than 20 veh/mi/ln with standard deviation 1.95. Because there is no obvious change in density over all time periods and freeway segments, it may indicate there is no obvious localized congestion observed. However, under western rural U.S. conditions, the density ranges between 5 veh/mi/ln and 46 veh/mi/ln with standard variance 6.22. There are three "strips" with high density values observed over several time periods and freeway sections in both the heat and contour maps. It expands from 1) time interval 21 to 26 with freeway segment 1 to 24, 2) time interval 34 to 40 with freeway segment 1 to 24, and 3) time interval 49 to 55 with freeway segment 1 to 24 may indicate there is localized congestion over the 6-mile freeway section.

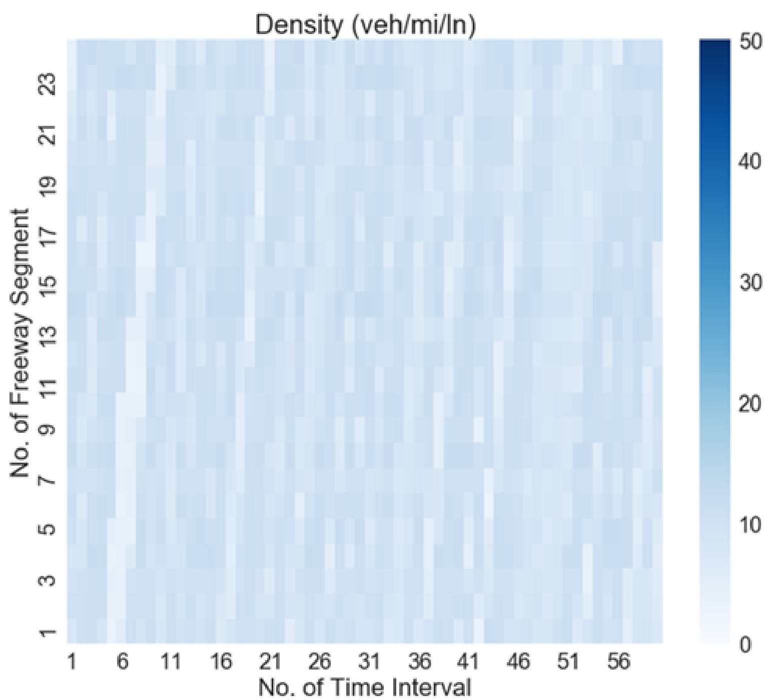


Figure 8.20 Heat map for the density at time period i at freeway segment j under HCM 2016 conditions

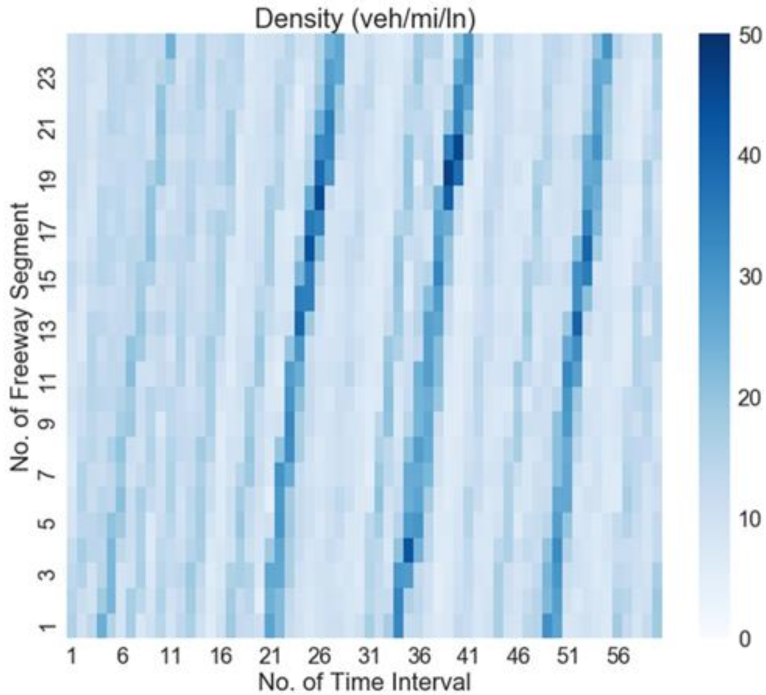


Figure 8.21 Heat map and for the density at time period i at freeway segment j under western rural U.S. conditions

The DUF_j over 60 consecutive time periods at freeway segments j under HCM 2016 conditions range between 0.69 and 0.82. However, under western rural U.S. conditions, it ranges between 0.32 and 0.47, which is on average 46 percent lower than under the HCM 2016 conditions, as shown in Figure 8.22. For the DUF_i over 24 consecutive freeway segments at time period i, under the HCM 2016 conditions, it ranges between 0.67 and 0.88. However, under the western rural U.S. conditions, it ranges between 0.21 and 0.66, which is on average 42 percent lower than the HCM 2016 conditions, as shown in Figure 8.23.

The DUF over 60 consecutive time periods and 24 consecutive freeway segments under HCM 2016 and western rural U.S. conditions are 0.79 and 0.42 respectively.

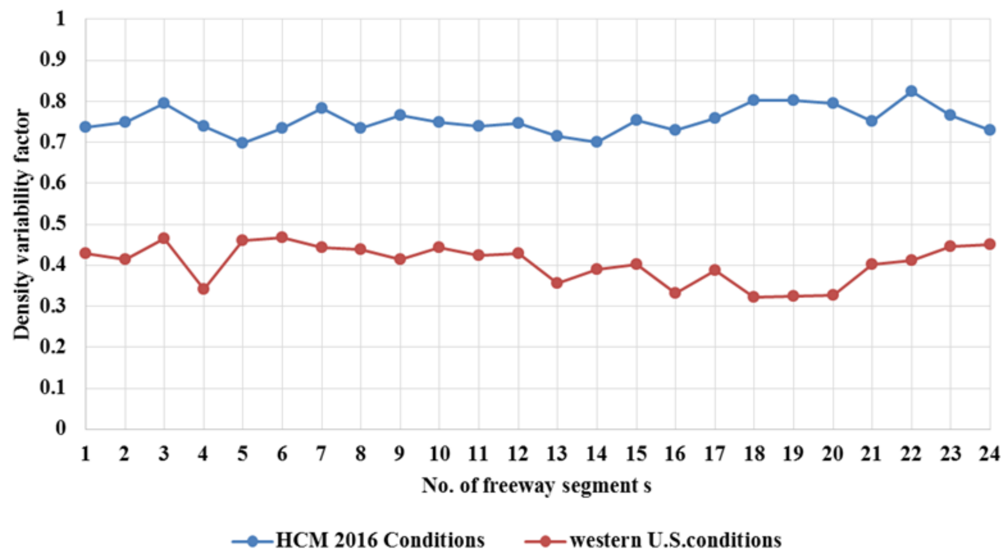


Figure 8.22 Density uniformity factor over 60 consecutive time periods at freeway segments j (DUF_j) under HCM 2016 and western rural U.S. conditions

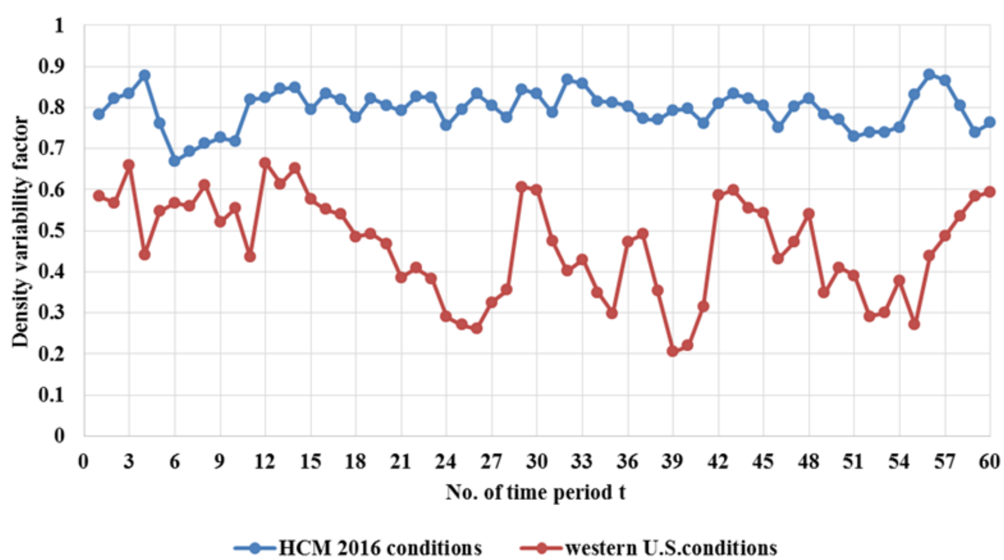


Figure 8.23 Density uniformity factor over 24 consecutive freeway segments at time period i under HCM 2016 and western rural U.S. conditions (DUF_i)

A statistical test is given to verify the density of the 6-mile freeway section under western rural U.S. conditions is significantly more "non-uniform" than under the HCM 2016 conditions based on the value DUF_j:

Null hypothesis: H_0: $\overline{DUF}_{j,1} = \overline{DUF}_{j,2}$

Alternative hypothesis: H_a: $\overline{DUF}_{j,2} < \overline{DUF}_{j,1}$

Where the $\overline{DUF}_{j,m}$ represents the average density uniformity factor over all freeway segments under m condition (1 for HCM 2016 conditions, 2 for western rural U.S. conditions). The $\overline{DUF}_{j,m}$ is calculated by the following equation:

$$\overline{DUF}_{j,m} = \frac{\sum_{j=1}^{S} DUF_{j,m}}{S} \tag{6.13}$$

$\overline{DUF}_{j,m}$	Average density uniformity factor over all freeway segments under m analyzing condition
j	Indicator for freeway segment, j = 1, 2, …, S
S	Total number of freeway segments, S = 24
m	Indicator for analyzing conditions, 1: HCM 2016 conditions, 2: western rural U.S. conditions
$DUF_{j,m}$	Density uniformity factor over T consecutive time periods at freeway segment j under m analyzing condition, range between 0 and 1, range between 0 and 1

The test is carried out by a one-tail t-test. The calculated t-value is -29.7, lower than the critical value $-t_{23,0.05}$ = -1.71 at 95% significance level, which means the null hypothesis is rejected. Thus, the density of the 6-mile freeway section under western rural U.S. conditions is significantly more “non-uniform” than under the HCM 2016 conditions.

Similarly, there is also a statistical test to show the density of the 6-mile freeway section under western rural U.S. conditions is significantly more “non-uniform” than under HCM 2016 conditions based on the value DUF_i:

Null hypothesis: H_0: $\overline{DUF}_{i,1} = \overline{DUF}_{i,2}$

Alternative hypothesis: H_a: $\overline{DUF}_{i,2} < \overline{DUF}_{i,1}$

Where the $\overline{DUF}_{i,m}$ represents the average density uniformity factor over all time periods under m condition (1 for HCM 2016 conditions, 2 for western Nebraska conditions). The $\overline{DUF}_{i,m}$ is calculated by the following equation:

$$\overline{DUF}_{i,m} = \frac{\sum_{i=1}^{T} DUF_{i,m}}{T} \tag{6.14}$$

$\overline{DUF}_{i,m}$	Average density uniformity factor over all time periods under m analyzing condition
i	Indicator for freeway segment, i = 1, 2, …, T
T	Total number of freeway segments, T = 60
m	Indicator for analyzing conditions, 1: HCM 2016 conditions, 2: western rural U.S. conditions

$DUF_{i,m}$	Density uniformity factor over S consecutive freeway segment at time period i under m analyzing condition, range between 0 and 1

The test is also carried out by a one-tail t-test. The results show that the calculated t-value is -19.95, lower than the critical value $-t_{59,0.05}$ = -1.68 at 95% significance level, which means the null hypothesis is rejected. Thus, the density of the 6-mile freeway section under western rural U.S. conditions is significantly more “non-uniform” than under the HCM 2016 conditions.

In conclusion, under the western rural U.S. conditions, the density of the 6-mile freeway section is considered much more “non-uniform” than under the HCM 2016 conditions. The result is consistent with the fact that the severe localized congestions caused by large numbers of moving bottlenecks are observed under western rural U.S. conditions.

To explore the applicable conditions for HCM 2016 recommended PCEs and western rural U.S. EC_PCEs, the relationship between EC_PCEs and DUF is developed, as shown in Figure 8.24. The DUF is calculated for each scenario for EC_PCE estimation in the equal-capacity method procedure. The DUF for each scenario is calculated under two conditions, the HCM 2016 conditions and western rural U.S. conditions. One scenario is a combination of truck percentage, truck composition, grade and distance. In this paper only the DUFs for 0% grade are calculated. There are 13 levels for truck percentage (e.g. 2%, 5%, 10%, 15%, 20%, 30%, 40%, 50%, 60%, 70%, 80%, 90%, 100%), three levels for truck composition (e.g. 30% SUT/70% TT, 50% SUT/50% TT, and 70% SUT/30% TT), and seven levels for distance (e.g. 0.25, 0.5, 0.75, 1, 1.5, 2.5 and 5 miles), which are same as parameters settings in Table 8.1. The results show that under western rural U.S. conditions, the DUFs range between 0.23 and 0.68 with average 0.51. Under the HCM 2016 conditions, the DUFs range between 0.57 and 0.91 with average 0.77. The DUFs under western rural U.S. conditions are found significantly lower than the HCM 2016 conditions with 35 percent on average with t-test at 95 level of confidence.

Comprehensively considering the DUF, DUF_i and DUF_j values, this paper suggests the HCM 2016 recommended PCEs are applicable to the traffic flows with DUF value higher than 0.7. If the DUF value of the traffic flow is no greater than 0.7, the western rural U.S. EC_PCEs are recommended to use. It should be noted the suggested thresholds here are determined based on the data from the simulation model calibrated by Nebraska I-80 empirical data. For different locations the threshold may be changed with the same methodology.

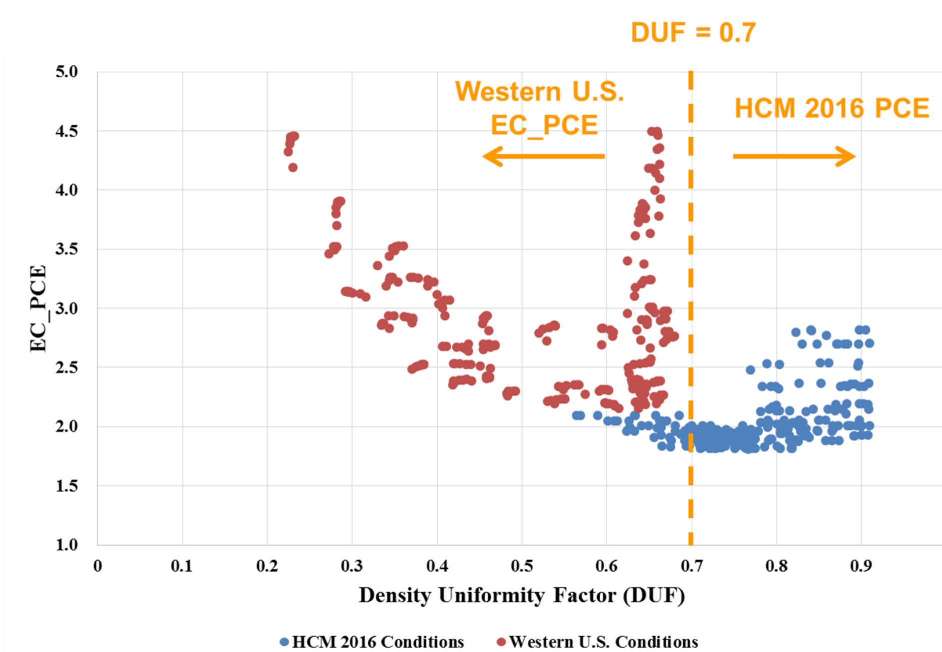


Figure 8.24 The relationship between EC_PCEs and DUFs for each simulation scenario

8.6 Summary

This research estimated truck PCEs for four-lane level freeway segments that experience high truck percentages and large speed differentials between heavy trucks and passenger cars, using the HCM 2016 equal-capacity method. Two conditions were studied - the HCM 2016 conditions and the western rural U.S. conditions. Following the HCM methodology the one-minute flow-density data was simulated using VISSIM 9.0. It was found that at level freeway segment (0% grade):

1) The EC_PCE values estimated under the western rural U.S. conditions are 20 to 70 percent higher than those estimated under the HCM 2016 conditions, and the PCEs recommended in HCM 2016;
2) The EC_PCE values increase with the percentage of heavy trucks under both the HCM 2016 and western rural U.S. conditions; and
3) The EC_PCEs increase with the grade length, decrease with the truck percentage up to 25 percent then slightly increase with the truck percentage under the western rural U.S. conditions.

It is hypothesized that the PCEs estimated under western rural U.S. conditions (Group 2) better capture the interaction between trucks and passenger cars on freeway segments in the western rural U.S. rural areas. This is because it simulated four-lane freeway segments that predominated in western rural U.S., and it used empirical free-flow speeds distributions from western rural U.S., and the empirical truck lengths.

Additionally, the applicable conditions for HCM 2016 recommended PCEs and the EC_PCEs estimated in this chapter are discussed based on the metric DUF proposed in

Chapter 6. The density under western rural U.S. conditions is proved much more "non-uniform" than under HCM 2016 conditions based on simulation data, which means the localized congestions under western rural U.S. conditions are proved to be much more severe than that under HCM 2016 conditions. Comprehensively considering the DUF values, the HCM 2016 recommended PCEs are suggested applicable to the traffic flows with DUF higher than 0.7, otherwise the estimated EC_PCEs in this chapter are suggested for using.

CHAPTER 9 SENSITIVITY ANALYSIS FOR WESTERN RURAL U.S. EC_PCES

This chapter examines a number of issues related to capacity and EC_PCE estimation under western rural U.S. conditions. The chapter first examines the effect of speed limit on EC_PCE values. Secondly, the effects of passing restrictions on EC_PCE values are examined. It is hypothesized that by restricting truck passing the number of moving bottlenecks will be reduced, thus reducing the EC_PCE value. Lastly, the 2016 HCM EC_PCE values were based on simulation data that was obtained at one minute intervals. The effects of using different data aggregation period (e.g. fifteen minutes that commonly used within the HCM), on EC_PCE values, are analyzed. It should be noted only the EC_PCEs under western rural U.S. conditions are examined in this chapter.

9.1 Sensitivity Analysis for EC_PCEs Based on Speed Limit Changes

9.1.1 Overview of Speed Limit

The speed limits of rural freeways vary among states in U.S. The speed limit of rural freeway segments for each state is shown in Table 9.1. The maximum-posted daytime speed limits on rural freeways for passenger cars in most of the states west of Mississippi River (except California, Oregon, Minnesota, Iowa and Missouri) and Michigan, Maine, are no lower than 75 mph. In general, these are higher than found in eastern states. In addition, in many states (e.g. Montana, Michigan, etc.), the speed limits for passenger cars and trucks are set separately. In these situations, the speed limits of trucks are 5 to 15 mph lower than that for passenger cars. Large differences in free-flow speed between passenger cars and trucks occur in states with high speed limits (no lower than 80 mph), like Texas, Montana, South Dakota, Utah, Wyoming, and Nevada. Recently the Nebraska government plans to raise speed limits on I-80 to 80 mph from the current 75 mph to make the transportation system more effective, efficient and customer-focused (86).

Table 9.1 Speed limit of rural freeway segments for each state in U.S.

Standard Federal Regions	**State**	**Speed Limit for Rural Freeway (mph) (Car)**	**Speed Limit for Rural Freeway (mph) (Truck)**
I (New England)	CT	65	65
	ME	75	75
	MA	65	65
	NH	65-70	65-70
	RI	65	65
	VT	65	65
II (New York Metropolitan)	NJ	65	65
	NY	65	65
III (Mid-Atlantic)	DE	55	55
	DC	-	-
	MD	70	70
	PA	65-70	65-70
	VA	70	70
	WV	70	70
IV (Southeast)	AL	70	70
	FL	70	70
	GA	70	70
	KY	65-70	65-70
	MS	70	70
	NC	70	70
	SC	70	70
	TN	70	70
V (North Central)	IL	70	70
	IN	70	65
	MI	70-75	65
	MN	70	70
	OH	70	70
	WI	70	70
VI (South Central)	AR	70-75	70-75
	LA	75	75
	NM	75	75
	OK	75	75

	TX	75-85	75-85
VII (Midwest)	IA	70	70
	KS	75	75
	MO	70	70
	NE	75	75
VIII (Mountains and Plains)	CO	75	75
	MT	80	65
	ND	75	75
	SD	80	80
	UT	75-80	75-80
	WY	75-80	75-80
IX (Pacific Southwest)	AZ	75	75
	CA	70	55
	HI	60	60
	NV	80	80
X (Pacific Northwest)	AK	65	65
	ID	70-80	70
	OR	65-70	55-65
	WA	70-75	60

This section discusses the impact of speed limits on EC_PCEs under western rural U.S. conditions at four-lane level freeway segments. There are four speed limits levels, which are 70 mph, 75 mph, 80 mph, and 85 mph. It is assumed that because the majority of trucks in western rural U.S. are equipped with speed limiters that the changes in speed limits will not affect their desired free flow speeds. As an aside speed limiters are applied to heavy trucks to increase safety and reduce fuel usage (81). Therefore, the empirical truck free flow speed profiles that were input to the VISSIM model were not changed as the speed limit was changed. It was observed that the empirical average passenger car free flow speed was close to the 75 mph speed limit. It was assumed in this paper that this would be true for other speed limit values. Therefore, the mean speed for the passenger cars was set to the speed limit.

It was also assumed the empirical variability in free flow speed observed at 75 mph would not change as the speed limit changed. For example, the desired free flow speed distribution simulated for passenger cars at 80 mph was modeled using the empirical free flow speed distribution for 75 mph with the exception that the distribution was "shifted" 5 mph to account for the higher speed limit. The free-flow speed distributions for different vehicle types at speed limit 70 mph, 75 mph, 80 mph, and 85 mph are shown in Figure 9.1, Figure 9.2, Figure 9.3, and Figure 9.4 respectively. The EC_PCEs are calculated for each level of speed limit, under no truck passing restriction conditions with 1-minute interval data. The impacts of speed limits on EC_PCEs are statistically analyzed.

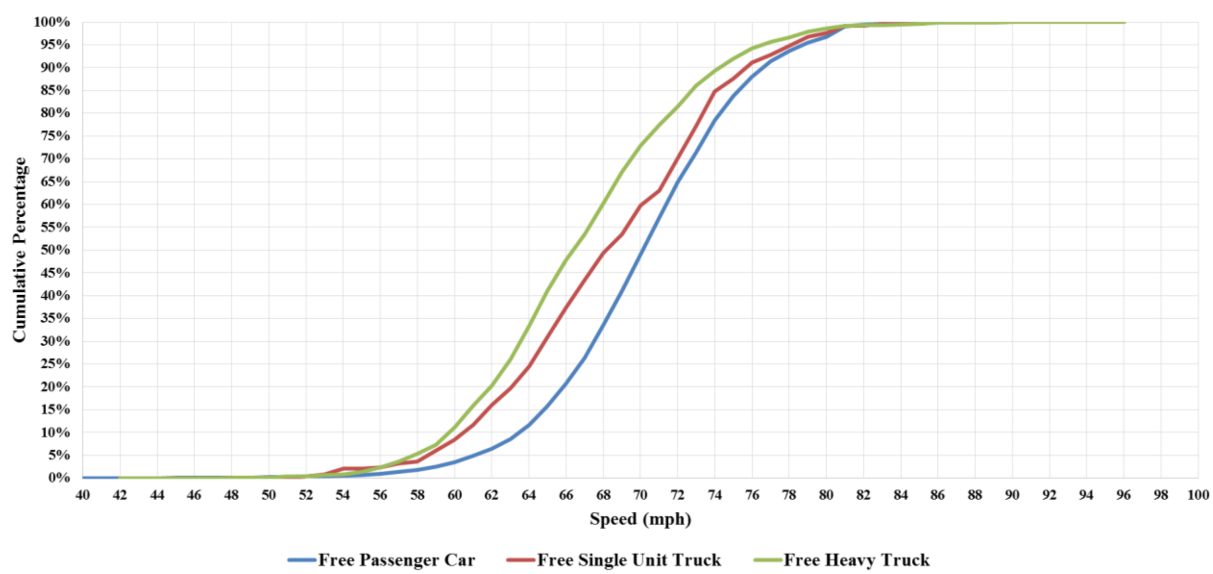


Figure 9.1 Free-flow speed distributions for different vehicle types at speed limit 70 mph

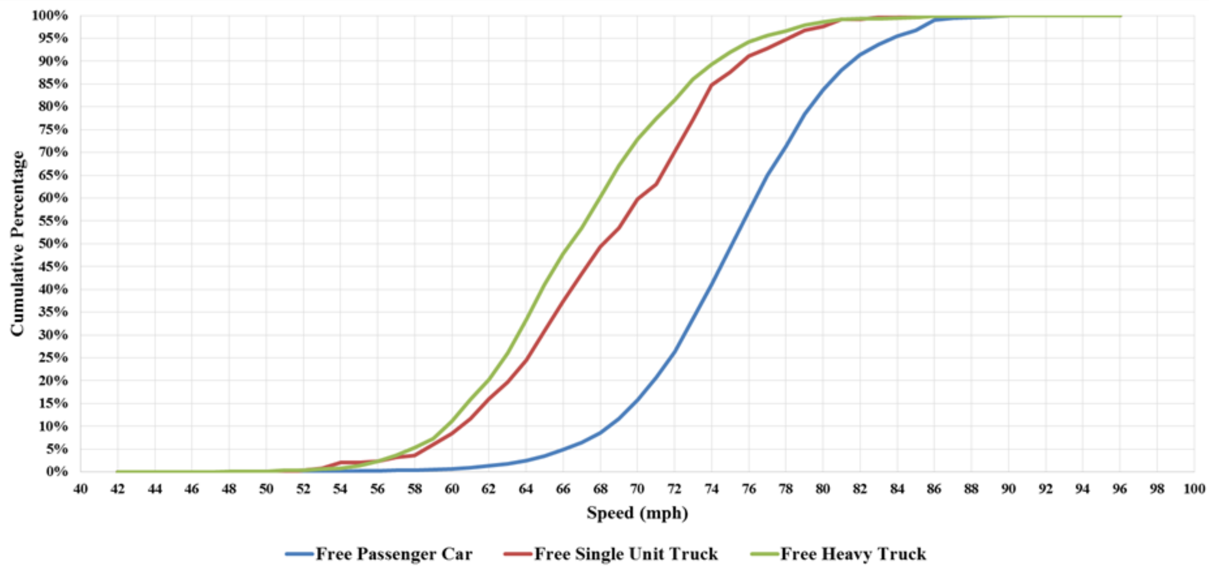


Figure 9.2 Free-flow speed distributions for different vehicle types at speed limit 75 mph

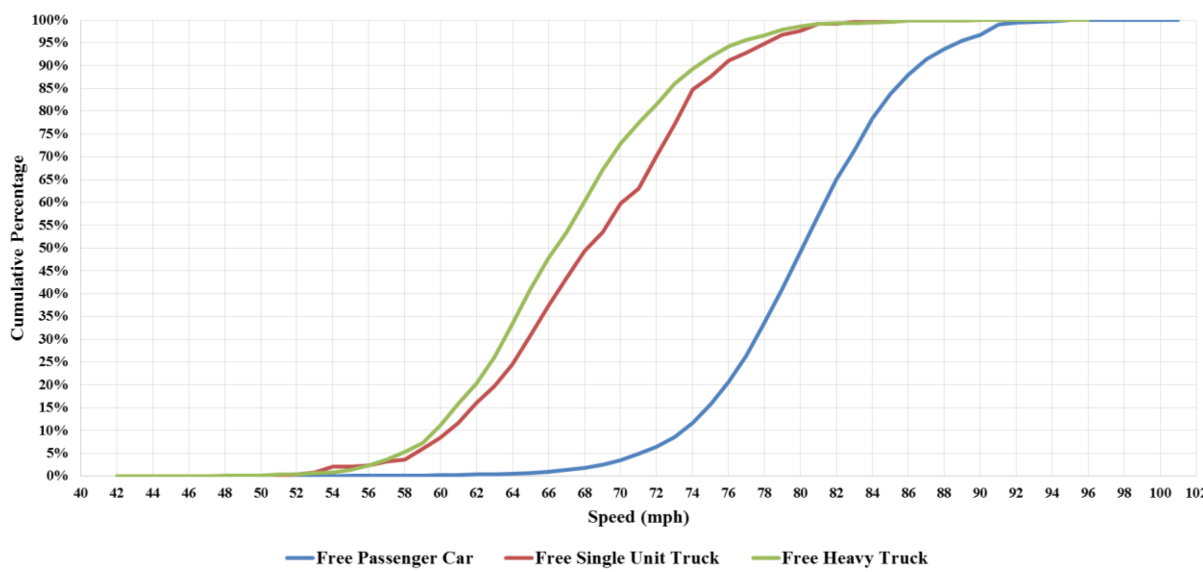


Figure 9.3 Free-flow speed distributions for different vehicle types at speed limit 80 mph

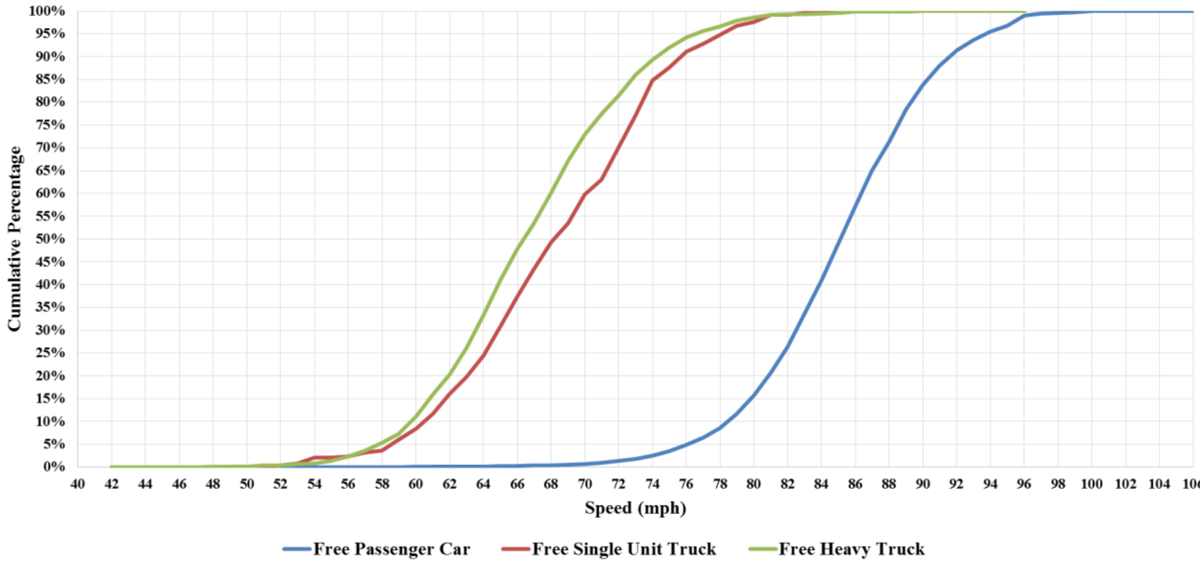


Figure 9.4 Free-flow speed distributions for different vehicle types at speed limit 85 mph

9.1.2 EC_PCEs at Different Speed Limit Level

The CAF estimation models were developed for speed limit values of 70 mph, 75 mph, 80 mph and 85 mph assuming no truck passing restrictions and using a 1-minute data aggregation level, as shown in Table 9.2 and Table 9.3. The R-square values for these models vary from 0.838 to 0.932, which show good correlations between the CAFs and the independent variables. It should be noted that the original form of the HCM CAF models were not changed – only the parameters were calibrated (97).

Table 9.2 Parameters in the models for CAF estimation at 70 mph and 75 mph speed limit

Parameters	**70 mph speed limit**			**75 mph speed limit**		
	3S7T*	5S5T*	7S3T*	3S7T*	5S5T*	7S3T*
$\alpha_{2,m}^{T_a}$	0.715	0.668	0.608	0.747	0.674	0.644
$\beta_{2,m}^{T_a}$	0.749	0.790	0.790	0.700	0.849	0.856
$\gamma_{2,m}^{G_a}$	8.000	8.000	8.000	8.000	8.000	8.000
$\theta_{2,m}^{G_a}$	0.135	0.148	0.132	0.124	0.137	0.137
$\mu_{2,m}^{G_a}$	0.016	0.049	0.041	0.036	0.047	0.020
$\alpha_{2,m}^{G_a}$	0.784	0.757	0.732	0.753	0.712	0.725
$\phi_{2,m}^{G_a}$	11.051	11.568	11.568	11.580	11.829	11.343
$\eta_{2,m}^{G_a}$	0.869	0.932	0.912	0.831	0.923	0.923
$\alpha_{2,m}^{D_a}$	1.532	1.450	1.350	1.800	1.447	1.220
$\beta_{2,m}^{D_a}$	1.382	1.448	1.490	1.438	1.493	1.659
$\phi_{2,m}^{D_a}$	-2.628	-2.487	-2.467	-2.851	-3.219	-2.554
$\mu_{2,m}^{FFS_a}$	0.250	0.250	0.250	-0.218	-0.388	-0.371
$\rho_{2,m}^{FFS_a}$	0.700	0.700	0.700	-0.145	-0.779	-1.052
$\beta_{2,m}^{FFS_a}$	1.000	1.000	1.000	0.200	0.288	1.404
$\phi_{2,m}^{FFS_a}$	1.000	1.000	1.000	1.000	1.000	1.000
R^2	0.907	0.874	0.838	0.932	0.878	0.859

*The 3S7T means 30% SUT/70% TT, 5S5T means 50% SUT/50% TT, 7S3T means 70% SUT/30% TT. The interpretations of parameters are same as Equation 8.6 to 8.10.

Table 9.3 Parameters in the models for CAF estimation at 80 mph and 85 mph speed limit

Parameters	**80 mph speed limit**			**85 mph speed limit**		
	3S7T*	5S5T*	7S3T*	3S7T*	5S5T*	7S3T*
$\alpha_{2,m}^{T_a}$	0.714	0.709	0.665	0.717	0.706	0.663
$\beta_{2,m}^{T_a}$	0.719	0.785	0.787	0.713	0.780	0.793
$\gamma_{2,m}^{G_a}$	8.000	8.000	8.000	8.000	8.000	8.000
$\theta_{2,m}^{G_a}$	0.136	0.149	0.141	0.137	0.145	0.139
$\mu_{2,m}^{G_a}$	0.016	0.005	0.035	0.018	0.050	0.045
$\alpha_{2,m}^{G_a}$	0.785	0.758	0.728	0.778	0.758	0.725
$\phi_{2,m}^{G_a}$	11.430	11.895	11.614	11.411	11.836	11.584
$\eta_{2,m}^{G_a}$	0.886	0.937	0.909	0.906	0.942	0.909
$\alpha_{2,m}^{D_a}$	1.538	1.429	1.381	1.506	1.451	1.370
$\beta_{2,m}^{D_a}$	1.173	1.453	1.501	1.260	1.435	1.472
$\phi_{2,m}^{D_a}$	-2.691	-2.502	-2.463	-2.651	-2.455	-2.511
$\mu_{2,m}^{FFS_a}$	-0.356	-0.414	-0.419	-0.335	-0.425	-0.450
$\rho_{2,m}^{FFS_a}$	-0.863	-0.750	-0.891	-0.887	-0.841	-0.909
$\beta_{2,m}^{FFS_a}$	0.190	0.252	0.308	0.232	0.270	0.306
$\phi_{2,m}^{FFS_a}$	1.000	1.000	1.000	1.000	1.000	1.000
R^2	0.918	0.910	0.838	0.912	0.912	0.909

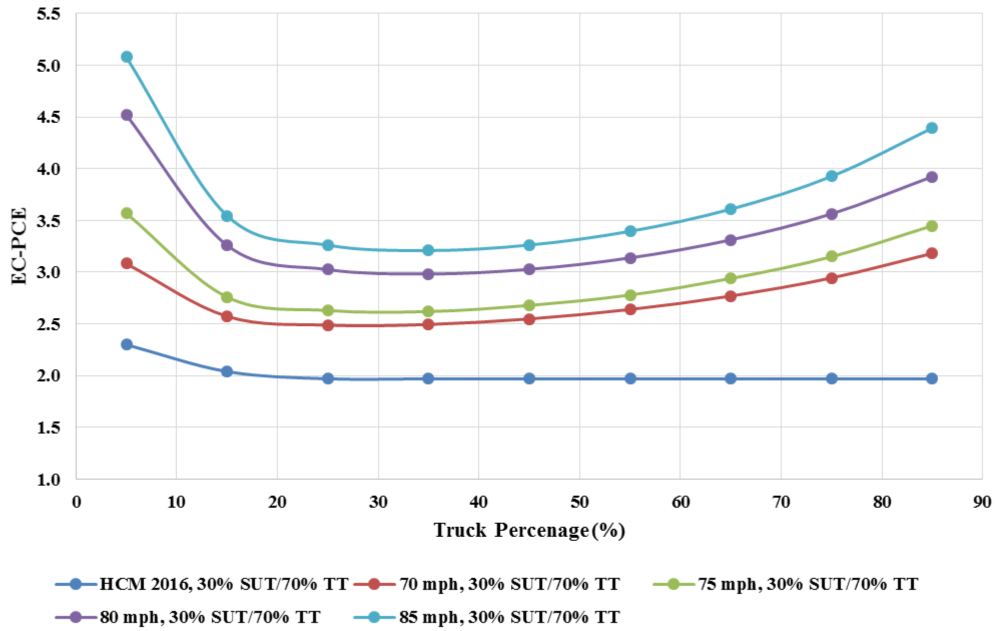
*The 3S7T means 30% SUT/70% TT, 5S5T means 50% SUT/50% TT, 7S3T means 70% SUT/30% TT. The interpretations of parameters are same as Equation 8.6 to 8.10.

Once the CAF models are calibrated the EC_PCEs are estimated for truck percentages ranging from 5% to 85% with 10% interval, six grade length 0.125, 0.375, 0.625, 0.875, 1.25, and 1.5 miles, and three truck composition 30% SUT/70% TT, 50% SUT/50% TT, 70% SUT/30% TT, at 0% grade. Because the results show that at different grade length level, the EC_PCEs show the same changing patterns with truck percentage, truck composition, and speed limit level, and also because of the limited space in the text, at here only the EC_PCEs at 1.5 miles grade length are shown. The EC_PCEs at other grade length are shown in appendix. The results of EC_PCEs are shown in the Table 9.4. The comparisons of EC_PCEs at each speed limit level with speed limit 75 mph, which is the empirical condition, are shown in Figure 9.5. It may be seen that as speed limit increases so too does the estimated EC_PCEs. This implies that the impacts of trucks on traffic flow become more severe as the speed limit increases, all else being equal. It may also be seen that the EC_PCE relationship with truck percentage is “u” shaped and that the lowest EC_PCE values occur around 25 percent. Not surprisingly, the more heavy vehicles (e.g. TT) in a given truck percentage the higher the EC_PCE value. Lastly, even when the lowest speed limit is used (e.g. 70 mph) the estimated EC_PCE values are, on average, 26% higher than the values recommended in HCM 2016. It is hypothesized that the speed limiters for trucks and lower number of lanes result in higher EC_PCE values and that the higher the speed limit, and the greater the difference in free flow speeds between trucks and passenger cars, the greater the effect. It is also easy to hypothesize that raising speed limits in the western rural U.S., without adopting mitigation strategies, may actually make traffic operations worse for all vehicle types.

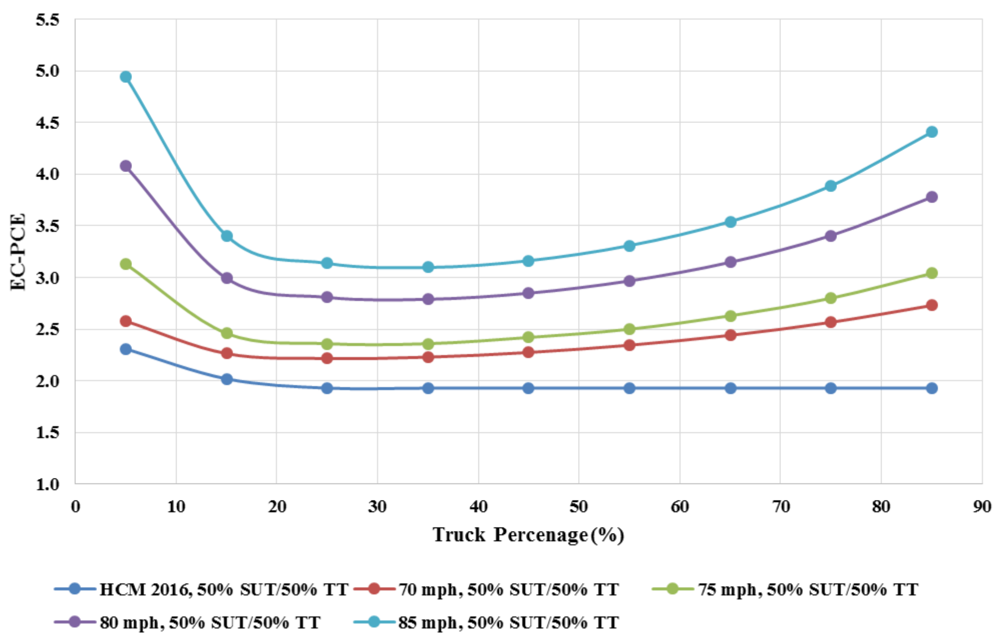
Table 9.4 EC_PCE results as a function of speed limit level (0% grade and 1.5 mile grade length)

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.87	3.00	2.85	2.83	2.88	2.99	3.15	3.37	3.69
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.76	2.26	2.19	2.21	2.26	2.34	2.45	2.60	2.80
70 mph, 0%, 1 min	30% SUT, 70% TT	3.08	2.57	2.49	2.50	2.55	2.64	2.77	2.95	3.18
	50% SUT, 50% TT	2.58	2.26	2.22	2.23	2.27	2.35	2.44	2.57	2.73
	70% SUT, 30% TT	2.48	2.15	2.09	2.09	2.11	2.16	2.22	2.30	2.40
80 mph, 0%, 1 min	30% SUT, 70% TT	4.52	3.26	3.03	2.98	3.03	3.14	3.31	3.56	3.92
	50% SUT, 50% TT	4.08	2.99	2.81	2.79	2.85	2.97	3.15	3.41	3.78
	70% SUT, 30% TT	4.07	2.94	2.73	2.70	2.73	2.82	2.96	3.15	3.42
85 mph, 0%, 1 min	30% SUT, 70% TT	5.08	3.55	3.26	3.21	3.26	3.40	3.61	3.93	4.39
	50% SUT, 50% TT	4.94	3.40	3.14	3.10	3.16	3.31	3.54	3.89	4.41
	70% SUT, 30% TT	5.00	3.36	3.07	3.00	3.04	3.15	3.33	3.60	4.00

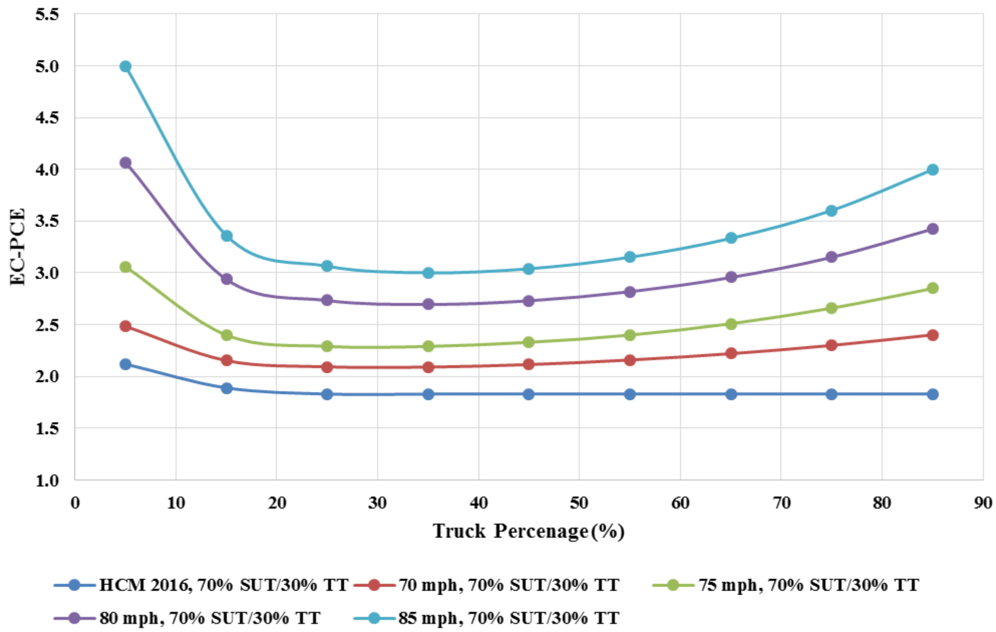
*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.



(a) Comparison of EC_PCEs among different speed limits at 30% SUT/70% TT



(b) Comparison of EC_PCEs among different speed limits at 50% SUT/50% TT



(c) Comparison of EC_PCEs among different speed limits at 70% SUT/30% TT

Figure 9.5 Comparison of EC_PCEs among different speed limits at (a) 30% SUT/70% TT (b) 50% SUT/50% TT and (c) 70% SUT/30% TT with no truck passing restriction, 1-minute data aggregated interval, 0% grade, and 1.5 miles grade length

9.1.3 Impacts of Speed Limits on EC_PCEs

To explore the impacts of changes in speed limits on EC_PCEs, a series of ANOVA and comparison tests are conducted:

1) The differences in EC_PCEs among different truck percentage levels;
2) The differences in EC_PCEs among different truck composition levels;
3) The differences in EC_PCEs among different speed limit levels;
4) The differences in EC_PCEs between each pair of speed limit levels (e.g. 70 mph v.s. 75 mph).

The ANOVA tests above are based on F-tests at the 95 percent level of confidence. The comparison tests above are based on t-tests at 95 percent level of confidence. The results for ANOVA analysis are shown in Table 9.5.

Table 9.5 Results of ANOVA and comparison test for EC_PCEs related to speed limit

Group	Test	Test method	Test results	P-value
ANOVA	Different truck percentage	F-test	5.97	< 0.05
	Different truck composition	F-test	5.20	0.007
	Different speed limit	F-test	32.9	< 0.05
Comparison	70 mph v.s. 75 mph speed limit	t-test	-2.65	0.005
	70 mph v.s. 80 mph speed limit	t-test	-7.08	< 0.05
	70 mph v.s. 85 mph speed limit	t-test	-8.88	< 0.05
	75 mph v.s. 80 mph speed limit	t-test	-3.96	< 0.05
	75 mph v.s. 85 mph speed limit	t-test	-6.17	< 0.05
	80 mph v.s. 85 mph speed limit	t-test	-2.70	0.005

*Note: The tests in this table are based on the EC_PCEs at 70, 75, 80, 85 speed limits, no truck passing restriction, 1-minute interval data, 0% grade, 1.5 miles grade length.

The analyses results show that:

1) The EC_PCEs vary with truck percentage;
2) The EC_PCEs increase with the percentage of heavy trucks;
3) The EC_PCEs vary with speed limit;
4) The EC_PCEs at 70 mph speed limits are lower than 75 mph speed limits, while the EC_PCEs at 80 mph and 85 mph speed limits are higher than 75 mph speed limits, which means the EC_PCEs also increase with the speed limit.

The differences are statistically significant at 95% level of confidence. Therefore, it can be concluded from the sensitivity analysis results that the speed limit has impacts on

EC-PCEs when the traffic flow experiencing moving bottlenecks. It can be explained as that, with the increase in speed limit, there is increase in the speed differentials between passenger cars and trucks, if it is assumed that when the trucks operate with speed limiters the free-flow speed distributions of trucks do not vary with speed limit. Because of the increase in speed differentials between passenger cars and trucks, when a truck would like to pass another truck at low speed differentials, the following passenger cars with speed higher than the overtaking trucks will experience more speed reduction and delay compared with in the free flows with passenger car only. Compounding the issues above, the moving bottlenecks will be more easily tending to form with the increase in speed limit, resulting in the localized congestion will be more apparently recognized by the drivers. Thus, at level four-lane freeway segments, when trucks operate with speed limiters, if and only if a higher speed limit is used, with all else being equal, the impacts of trucks on passenger cars will become more severe, which is reflected by the EC-PCEs significantly increasing with the speed limit.

9.2 Sensitivity Analysis for EC_PCEs based on Truck Passing Restriction

9.2.1 Overview of Truck Passing Restriction

In many locations, truck restriction strategies have been implemented to mitigate localized congestion, improve highway operations, and improve safety (101). These include lane restrictions, route restrictions, time-of-day restrictions, and speed restrictions. This paper focuses on lane restrictions. When there is a lane restriction all trucks, or trucks of a specific size, weight, or axle configuration, are restricted from traveling in specified lanes (102). Lane restrictions have been implemented in Germany and other European countries, where trucks are often restricted to the right lane (85). Because many of the freeways in Europe have two lanes in each direction, the lane restrictions combined with a lower truck speed limit restricts trucks to the right lane and passing opportunities are constrained. Lane restrictions have also been implemented in many of U.S. states (e.g. Texas, Illinois, Nevada, etc.). This is particularly true in urban freeway segments, where trucks are restricted to the right two lanes when there are three or more than three lanes per direction, or restricted to the right lane when there are two lanes per direction (103). The lane restriction will cause truck passing restriction, which means the trucks on the shoulder lane are not permitted to use the median lane to pass the vehicles on the shoulder lane. To date there has been few studies on the impacts of truck passing restriction rules on PCEs.

This section discusses the impact of truck passing restrictions on EC_PCEs under western rural U.S. conditions at four-lane level freeway segments. Because this research focused on the rural sections of interstate highways, it is assumed that the truck passing restriction would be on the median lane of the freeways and that trucks would only use the rightmost or shoulder lane. This is readily accomplished in VISSIM by using the “blockage” function. The truck passing restrictions were modeled for various levels (e.g. 0, 25, 50, 75 and 100 percent) of the simulated freeways. For example, a value of 75 percent implies that truck passing restrictions are in effect for 75 percent of the freeway network. The EC_PCEs are calculated for different truck passing restriction levels are under the speed limit of 75 mph with 1-minute interval data. The impacts of truck passing restrictions on EC_PCEs are statistically analyzed.

9.2.2 EC_PCEs at Different Truck Passing Restriction Levels

Similar to the previous sensitivity analysis, the first step is to develop CAF estimation models. In this case they are for truck passing restrictions of 0%, 25%, 50%, 75% and 100% of the simulated freeway miles, as shown in Table 9.6 and Table 9.7. As before a 75 mph speed limit and 1-minute data aggregation level, which corresponds to that used in the base case, were used. The R-square values for these models vary from 0.825 to 0.932, which show good correlations between CAFs and independent variables. It should be noted that the original form of the HCM CAF models were not changed – only the parameters were calibrated (97).

Table 9.6 Parameters in the models for CAF estimation at 0%, 25% and 50% truck passing restriction

Parameters	0% truck passing restriction			25% truck passing restriction			50% truck passing restriction		
	3S7T*	5S5T*	7S3T*	3S7T*	5S5T*	7S3T*	3S7T*	5S5T*	7S3T*
$\alpha_{2,m}^{T_a}$	0.747	0.674	0.644	0.653	0.608	0.573	0.595	0.552	0.474
$\beta_{2,m}^{T_a}$	0.700	0.849	0.856	0.740	0.833	0.926	0.874	0.948	0.981
$\gamma_{2,m}^{G_a}$	8.000	8.000	8.000	8.000	8.000	8.000	8.000	8.000	8.000
$\theta_{2,m}^{G_a}$	0.124	0.137	0.137	0.139	0.136	0.144	0.138	0.140	0.135
$\mu_{2,m}^{G_a}$	0.036	0.047	0.020	0.020	0.043	0.040	0.007	0.053	0.020
$\alpha_{2,m}^{G_a}$	0.753	0.712	0.725	0.775	0.741	0.729	0.793	0.752	0.768
$\phi_{2,m}^{G_a}$	11.580	11.829	11.343	11.398	11.802	11.617	11.599	11.904	11.611
$\eta_{2,m}^{G_a}$	0.831	0.923	0.923	0.886	0.920	0.911	0.892	0.983	0.950
$\alpha_{2,m}^{D_a}$	1.800	1.447	1.220	1.503	1.410	1.383	1.504	1.433	1.398
$\beta_{2,m}^{D_a}$	1.438	1.493	1.659	1.403	1.476	1.468	1.441	1.453	1.442
$\phi_{2,m}^{D_a}$	-2.851	-3.219	-2.554	-2.610	-2.575	-2.509	-2.618	-2.486	-2.529
$\mu_{2,m}^{FFS_a}$	-0.218	-0.388	-0.371	-0.246	-0.258	-0.383	-0.357	-0.386	-0.365
$\rho_{2,m}^{FFS_a}$	-0.145	-0.779	-1.052	-0.680	-0.976	-0.915	-1.088	-1.592	-1.227
$\beta_{2,m}^{FFS_a}$	0.200	0.288	1.404	0.155	0.082	0.033	0.245	0.222	0.078
$\phi_{2,m}^{FFS_a}$	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
R^2	0.932	0.878	0.859	0.883	0.837	0.854	0.825	0.829	0.847

*The 3S7T means 30% SUT/70% TT, 5S5T means 50% SUT/50% TT, 7S3T means 70% SUT/30% TT. The interpretations of parameters are same as Equation 8.6 to 8.10.

Table 9.7 Parameters in the models for CAF estimation at 50% and 75% truck passing restriction

Parameters	**75% truck passing restriction**			**100% truck passing restriction**		
	3S7T*	5S5T*	7S3T*	3S7T*	5S5T*	7S3T*
$\alpha_{2,m}^{T_a}$	0.553	0.496	0.410	0.516	0.437	0.362
$\beta_{2,m}^{T_a}$	0.872	0.947	0.940	0.872	0.885	0.986
$\gamma_{2,m}^{G_a}$	8.000	8.000	8.000	8.000	8.000	8.000
$\theta_{2,m}^{G_a}$	0.137	0.144	0.138	0.138	0.144	0.135
$\mu_{2,m}^{G_a}$	0.011	0.051	0.044	0.016	0.051	0.040
$\alpha_{2,m}^{G_a}$	0.793	0.752	0.732	0.792	0.748	0.733
$\phi_{2,m}^{G_a}$	11.593	11.940	11.578	11.556	11.892	11.269
$\eta_{2,m}^{G_a}$	0.860	0.976	0.912	0.881	0.995	0.910
$\alpha_{2,m}^{D_a}$	1.490	1.431	1.413	1.508	1.450	1.332
$\beta_{2,m}^{D_a}$	1.430	1.451	1.497	1.439	1.452	1.465
$\phi_{2,m}^{D_a}$	-2.595	-2.486	-2.546	-2.611	-2.480	-2.540
$\mu_{2,m}^{FFS_a}$	-0.330	-0.391	-0.395	-0.326	-0.388	-0.365
$\rho_{2,m}^{FFS_a}$	-0.980	-0.708	-0.945	-0.866	-0.717	-1.190
$\beta_{2,m}^{FFS_a}$	0.180	0.073	0.146	0.140	0.091	0.084
$\phi_{2,m}^{FFS_a}$	1.000	1.000	1.000	1.000	1.000	1.000
R^2	0.848	0.868	0.879	0.829	0.817	0.811

*The 3S7T means 30% SUT/70% TT, 5S5T means 50% SUT/50% TT, 7S3T means 70% SUT/30% TT. The interpretations of parameters are same as Equation 8.6 to 8.10.

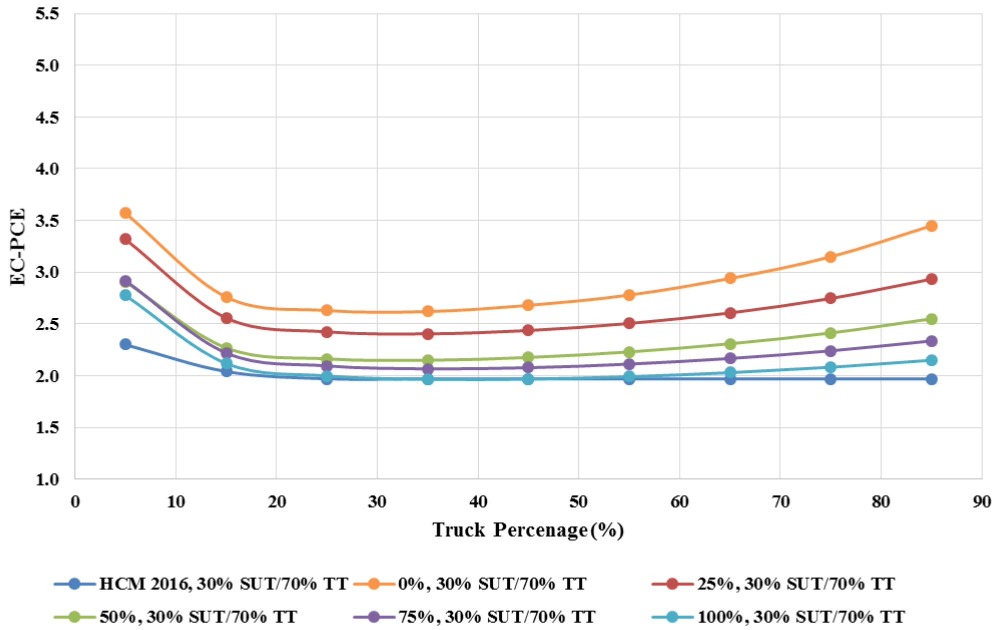
Similar to above, once the CAF models are calibrated the EC_PCEs are estimated for truck percentages ranging from 5% to 85% with 10% interval, six grade length 0.125, 0.375, 0.625, 0.875, 1.25 and 1.5 miles, and three truck composition 30% SUT/70% TT, 50% SUT/50% TT, 70% SUT/30% TT, at 0% grade. Because the results show that at different grade length level, the EC_PCEs show the same changing patterns with truck percentage, truck composition, and truck passing restriction levels, and also because of the limited space in the text, at here only the EC_PCEs at 1.5 miles grade length are shown. The

EC_PCEs at other grade length are shown in appendix. The EC_PCEs results are shown in Table 9.8. For ease of use the EC-PCEs for all three SUT/TT ratios are shown in Figure 9.6. It may be seen that as the truck passing restriction percentage increases the EC_PCE decreases. This would be expected because the truck passing restriction limits the amount of passing opportunities for trucks and hence reduces the number of moving bottlenecks. At the extreme (e.g. 100 percent truck passing restriction) the EC_PCEs are, on average, 22% lower than the EC-PCEs scenario with no truck passing restrictions. It is hypothesized that adding truck passing restrictions, which are a fairly inexpensive operational countermeasure, may be more cost effective than constructing additional lanes when capacity becomes an issue on western rural U.S. highways.

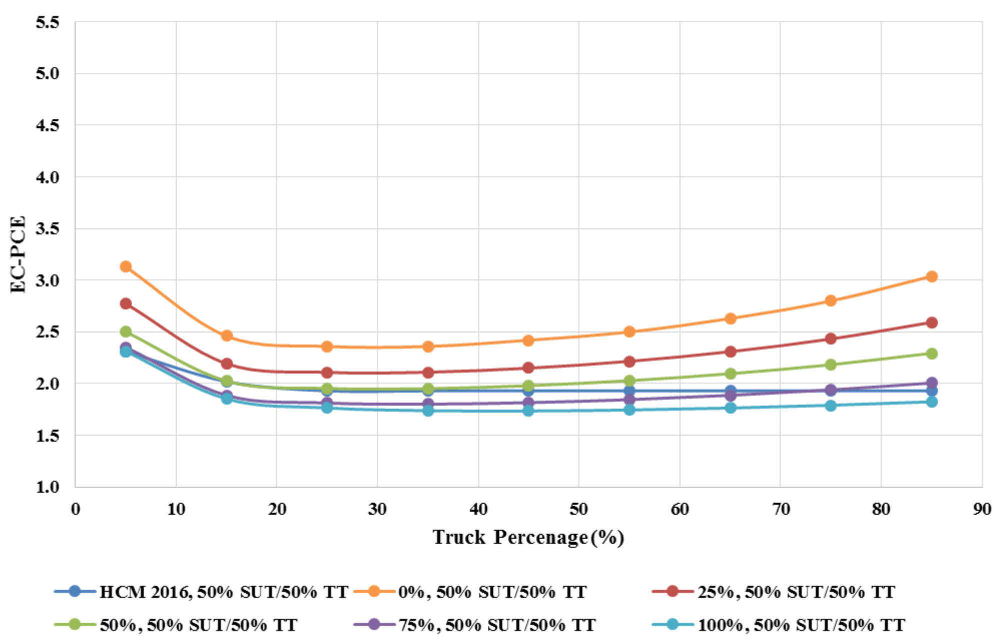
Table 9.8 EC_PCE results as a function of truck passing restriction level (0% grade and 1.5 mile grade length)

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.87	3.00	2.85	2.83	2.88	2.99	3.15	3.37	3.69
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.76	2.26	2.19	2.21	2.26	2.34	2.45	2.60	2.80
75 mph, 25%, 1 min	30% SUT, 70% TT	3.35	2.62	2.47	2.44	2.46	2.52	2.60	2.72	2.87
	50% SUT, 50% TT	2.84	2.27	2.17	2.16	2.18	2.23	2.30	2.40	2.53
	70% SUT, 30% TT	2.83	2.15	2.04	2.03	2.05	2.10	2.17	2.26	2.38
75 mph, 50%, 1 min	30% SUT, 70% TT	2.91	2.27	2.16	2.15	2.18	2.23	2.31	2.41	2.55
	50% SUT, 50% TT	2.50	2.02	1.95	1.95	1.98	2.03	2.10	2.18	2.29
	70% SUT, 30% TT	2.47	1.89	1.80	1.78	1.79	1.82	1.85	1.90	1.97
75 mph, 75%, 1 min	30% SUT, 70% TT	2.92	2.22	2.09	2.07	2.08	2.11	2.17	2.24	2.34
	50% SUT, 50% TT	2.35	1.89	1.81	1.8	1.82	1.85	1.89	1.94	2.01
	70% SUT, 30% TT	2.48	1.87	1.75	1.71	1.71	1.71	1.73	1.75	1.78
75 mph, 100%, 1 min	30% SUT, 70% TT	2.77	2.12	2.00	1.97	1.97	1.99	2.03	2.08	2.15
	50% SUT, 50% TT	2.31	1.85	1.76	1.74	1.73	1.74	1.76	1.79	1.82
	70% SUT, 30% TT	2.43	1.78	1.66	1.62	1.61	1.61	1.62	1.63	1.65

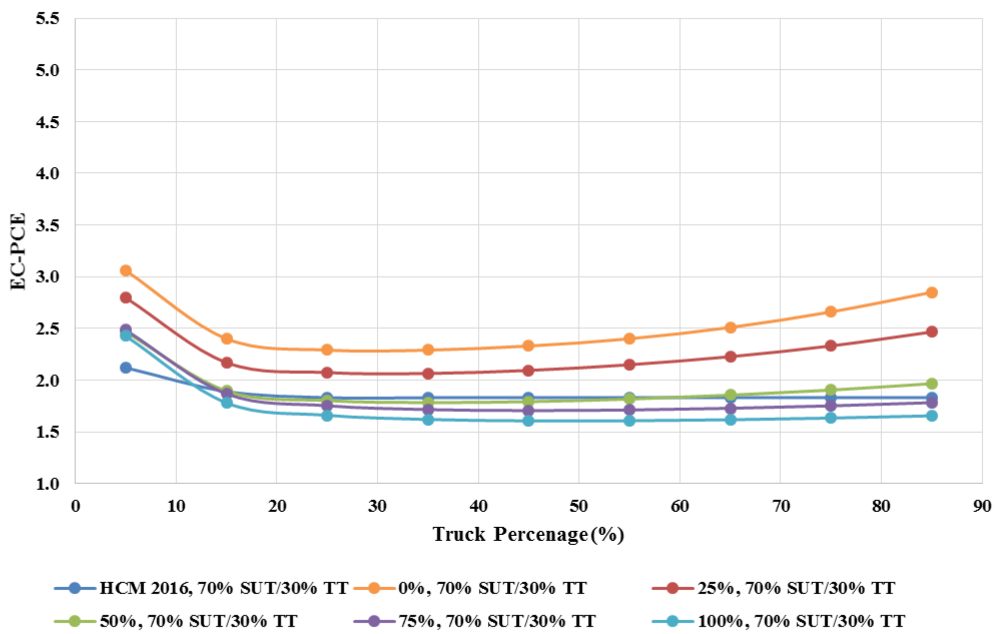
*Note: The marks mean for this group the EC_PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.



(a) Comparison of EC_PCEs among different percentages of truck passing restriction at 30% SUT/70% TT



(b) Comparison of EC_PCEs among different percentages of truck passing restriction at 50% SUT/50% TT



(c) Comparison of EC_PCEs among different percentages of truck passing restriction at 70% SUT/30% TT

Figure 9.6 Comparison of EC_PCEs among different percentages of truck passing restriction at (a) 30% SUT/70% TT (b) 50% SUT/50% TT and (c) 70% SUT/30% TT with 75 mph speed limit, 1-minute data aggregated interval, 0% grade, and 1.5 miles grade length.

9.2.3 Impacts of Truck Passing Restriction on PCEs

To explore the impacts of truck passing restrictions on EC_PCEs, a series of ANOVA tests and comparison tests are conducted based on F-tests and t-tests respectively at the 95 percent level of confidence, which are

1) The differences in EC_PCEs among different truck percentage levels;
2) The differences in EC_PCEs among different truck composition levels;
3) The differences in EC_PCEs among different truck passing restriction levels;

4) The differences in EC_PCEs between pairs of different truck passing restriction levels (e.g. no truck passing restriction vs. 25% truck passing restriction).

The results for ANOVA and comparison tests are shown in Table 9.9.

Table 9.9 Results of ANOVA and comparison tests for EC_PCEs related to truck passing restriction

Group	Test	Test method	Test results	P-value
ANOVA	Different truck percentage	F-test	5.24	< 0.05
	Different truck composition	F-test	18.52	< 0.05
	Different truck passing restriction	F-test	29.0	< 0.05
Comparison	0% v.s. 25% truck passing restriction	t-test	3.04	0.002
	0% v.s. 50% truck passing restriction	t-test	6.01	< 0.05
	0% v.s. 75% truck passing restriction	t-test	7.17	< 0.05
	0% v.s. 100% truck passing restriction	t-test	8.28	< 0.05
	25% v.s. 50% truck passing restriction	t-test	3.54	< 0.05
	25% v.s. 75% truck passing restriction	t-test	4.99	< 0.05
	25% v.s. 100% truck passing restriction	t-test	6.36	< 0.05
	50% v.s. 75% truck passing restriction	t-test	1.62	0.055
	50% v.s. 100% truck passing restriction	t-test	3.05	0.002
	75% v.s. 100% truck passing restriction	t-test	1.39	0.085

*Note: The tests in this table are based on the EC_PCEs at 0%, 25%, 50%, 75%, 100% truck passing restriction, 75 mph speed limit, 15-minute interval data, 0% grade, 1.5 miles.

The analyses results show that:

1) The EC_PCEs vary with the truck percentage;
2) The EC_PCEs increase with the percentage of heavy trucks;
3) The EC_PCEs vary with truck passing restriction;
4) The EC_PCEs with truck passing restriction conditions are lower than the EC_PCEs with no truck passing restriction conditions, also, the EC_PCEs decrease with the increase in truck passing restriction percentage.

The differences are statistically significant at 95% level of confidence. Therefore, it can be concluded from the sensitivity analysis that the truck passing restriction has impacts on EC-PCEs when traffic flow experiences moving bottlenecks. This can be explained as there is decrease in the distance of freeway segments that permit a truck changing lanes with the increase in the percentage of total distance having truck passing restriction. This may cause a decrease in the amount of a truck passing another truck at low speed differentials, resulting in passenger cars with speeds higher than the trucks having less chance to reduce speed to follow the overtaking trucks, which means there will be less delay for the passenger cars compared with no truck passing restriction conditions. Compounding the issues above, the amount of moving bottlenecks will be less likely to form with the increase in truck passing restriction percentage, resulting in the localized congestion will be less recognized by the drivers. Thus, at level four-lane freeway segments, when trucks operate with speed limiters, if and only if trucks are restricted at a high percentage of total distance, with all else being equal, the impacts of trucks on passenger car operation will decrease, which is reflected by the EC_PCEs significantly decreasing with the percentage of truck passing restriction.

9.3 Sensitivity Analysis for EC_PCEs Based on Data Aggregation Interval

9.3.1 Overview of Data Aggregation Interval

The HCM 2016 EC_PCEs are estimated based on the simulation data aggregated using a 1-minute interval (3 and 39). However, in the HCM 2016 the freeway capacity represents a maximum hourly flow rate for a 15-min interval (3). In addition, in the level-of-service analysis for basic freeway segments the analysis period is generally the peak 15-min period within the peak hour (3). It would be expected that there would be more variability, and hence a higher capacity, when a 1-minute-based hourly flow rate is used compared to the situation when a 15-minute-based hourly flow rate is used (104). To date, there has been no research conducted on how the aggregation level for the data used in the 2016 HCM PCE estimation methodology affects PCE estimates.

This section discusses the impact of data aggregation interval on EC_PCEs under western rural U.S. conditions at four-lane level freeway segments. The parameter settings in VISSIM 9.0 simulation models used here are the same as Chapter 8. The HCM 2016 EC_PCE estimation procedures in section 8.2 are used here. The only difference in the EC_PCE estimation procedure between this section and section 8.2 is the data are aggregated by 1, 5, 10, 15 and 20 minute intervals. The difference among different data aggregation intervals are reflected in Step 1. For example, using the 15 minute interval implies that, in the flow-density scatter plots developed for auto-only and mixed flow in the Step 1, each point represents the relationship of flow-rate and density in 15 minutes. After that, the capacity of auto-only flow and mixed flow based on 15-minute-interval data can be obtained to calculate the capacity adjustment factors (CAFs). The EC_PCEs calculated for different data aggregation interval levels are under the speed limit of 75 mph and no truck passing restriction. The impacts of data aggregation intervals on EC_PCEs are statistically analyzed.

9.3.2 EC_PCEs at Different Data Aggregation Interval

Similar to the two previous sensitivity analyses the first step is to develop CAF estimation models using data aggregated into 1, 5, 10, 15 and 20 minutes intervals, as shown in Table 9.10 and Table 9.11. As before, it was assumed there was a 75 mph speed limit and no truck passing restrictions (e.g. similar to base case). The R-square values for these models vary from 0.824 to 0.932, which show good correlations between the CAFs and the independent variables. It should be noted that the original form of the HCM CAF models were not changed – only the parameters were calibrated (97).

Table 9.10 Parameters in the models for CAF estimation at 1, 5 and 10 minutes intervals

Parameters	**1-minute interval**			**5-minute interval**			**10-minute interval**		
	3S7T*	5S5T*	7S3T*	3S7T	5S5T	7S3T	3S7T	5S5T	7S3T
$\alpha_{2,m}^{T_a}$	0.747	0.674	0.644	0.725	0.662	0.627	0.720	0.657	0.625
$\beta_{2,m}^{T_a}$	0.700	0.849	0.856	0.774	0.864	0.859	0.776	0.864	0.861
$\gamma_{2,m}^{G_a}$	8.000	8.000	8.000	8.000	8.000	8.000	8.000	8.000	8.000
$\theta_{2,m}^{G_a}$	0.124	0.137	0.137	0.133	0.159	0.134	0.139	0.186	0.133
$\mu_{2,m}^{G_a}$	0.036	0.047	0.020	0.034	0.048	0.029	0.032	0.048	0.027
$\alpha_{2,m}^{G_a}$	0.753	0.712	0.725	0.759	0.703	0.747	0.763	0.672	0.736
$\phi_{2,m}^{G_a}$	11.580	11.829	11.343	11.589	11.605	11.119	11.590	11.300	11.104
$\eta_{2,m}^{G_a}$	0.831	0.923	0.923	0.862	0.924	0.911	0.916	0.919	0.908
$\alpha_{2,m}^{D_a}$	1.800	1.447	1.220	1.806	1.392	1.715	1.429	1.261	1.725
$\beta_{2,m}^{D_a}$	1.438	1.493	1.659	1.431	1.805	1.173	1.807	1.824	1.143
$\phi_{2,m}^{D_a}$	-2.851	-3.219	-2.554	-2.846	-3.309	-2.176	-2.848	-3.424	-2.181
$\mu_{2,m}^{FFS_a}$	-0.218	-0.388	-0.371	-0.228	-0.374	-0.319	-0.230	-0.355	-0.283
$\rho_{2,m}^{FFS_a}$	-0.145	-0.779	-1.052	-0.146	-0.584	-1.004	-0.148	-0.553	-1.089
$\beta_{2,m}^{FFS_a}$	0.200	0.288	1.404	0.115	0.504	1.002	0.101	0.527	0.96
$\phi_{2,m}^{FFS_a}$	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
R^2	0.932	0.878	0.859	0.906	0.865	0.852	0.915	0.885	0.844

*The 3S7T means 30% SUT/70% TT, 5S5T means 50% SUT/50% TT, 7S3T means 70% SUT/30% TT. The interpretations of parameters are same as Equation 8.6 to 8.10.

Table 9.11 Parameters in the models for CAF estimation at 15 and 20 minutes intervals

Parameters	**15-minute interval**			**20-minute interval**		
	3S7T*	5S5T*	7S3T*	3S7T	5S5T	7S3T
$\alpha_{2,m}^{T_a}$	0.720	0.665	0.623	0.707	0.644	0.615
$\beta_{2,m}^{T_a}$	0.778	0.862	0.868	0.783	0.875	0.861
$\gamma_{2,m}^{G_a}$	8.000	8.000	8.000	8.000	8.000	8.000
$\theta_{2,m}^{G_a}$	0.141	0.196	0.133	0.144	0.202	0.129
$\mu_{2,m}^{G_a}$	0.034	0.048	0.025	0.022	0.042	0.042
$\alpha_{2,m}^{G_a}$	0.765	0.670	0.729	0.732	0.655	0.713
$\phi_{2,m}^{G_a}$	11.592	11.119	11.807	11.397	11.085	11.578
$\eta_{2,m}^{G_a}$	0.932	0.928	0.911	0.918	0.938	0.915
$\alpha_{2,m}^{D_a}$	1.807	1.853	1.133	1.421	1.227	1.569
$\beta_{2,m}^{D_a}$	1.428	1.216	1.736	1.782	1.837	1.262
$\phi_{2,m}^{D_a}$	-2.846	-3.494	-2.185	-2.805	-3.476	-2.659
$\mu_{2,m}^{FFS_a}$	-0.231	-0.349	-0.268	-0.225	-0.360	-0.228
$\rho_{2,m}^{FFS_a}$	-0.148	-0.508	-1.094	-0.144	-0.453	-1.031
$\beta_{2,m}^{FFS_a}$	0.098	0.582	0.790	0.084	0.569	0.512
$\phi_{2,m}^{FFS_a}$	1.000	1.000	1.000	1.000	1.000	1.000
R^2	0.906	0.863	0.858	0.899	0.844	0.838

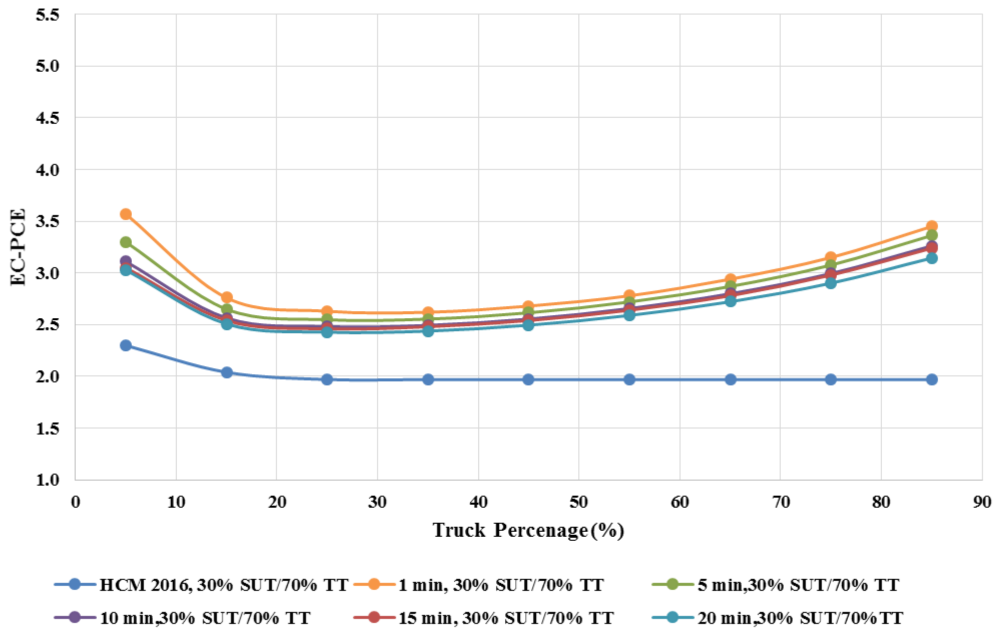
*The 3S7T means 30% SUT/70% TT, 5S5T means 50% SUT/50% TT, 7S3T means 70% SUT/30% TT. The interpretations of parameters are same as Equation 8.6 to 8.10.

Similar to above, once the CAF models are calibrated the EC_PCEs are estimated for truck percentages ranging from 5% to 85% with 10% intervals, six grade length 0.125, 0.375, 0.625, 0.875, 1.25, and 1.5 miles, and three truck compositions 30% SUT/70% TT, 50% SUT/50% TT, 70% SUT/30% TT, at 0% grade. Because the results show that at different grade length level, the EC_PCEs show the same changing patterns with truck percentage, truck composition, and data aggregation level, and also because of the limited space in the text, at here only the EC_PCEs at 1.5 miles grade length are shown. The EC_PCEs at other grade length are shown in the appendix. The EC_PCEs results are shown in Table 9.12. For ease of comparison they are also plotted in Figure 9.7. Not surprisingly the EC-PCEs decrease with the increase in the length of data aggregation interval. This is not surprising because higher aggregation intervals tend to reduce variability and result in lower capacity values. It is easy to hypothesize that if a larger aggregation interval was used, then the EC-PCEs reported in the 2016 HCM would be reduced.

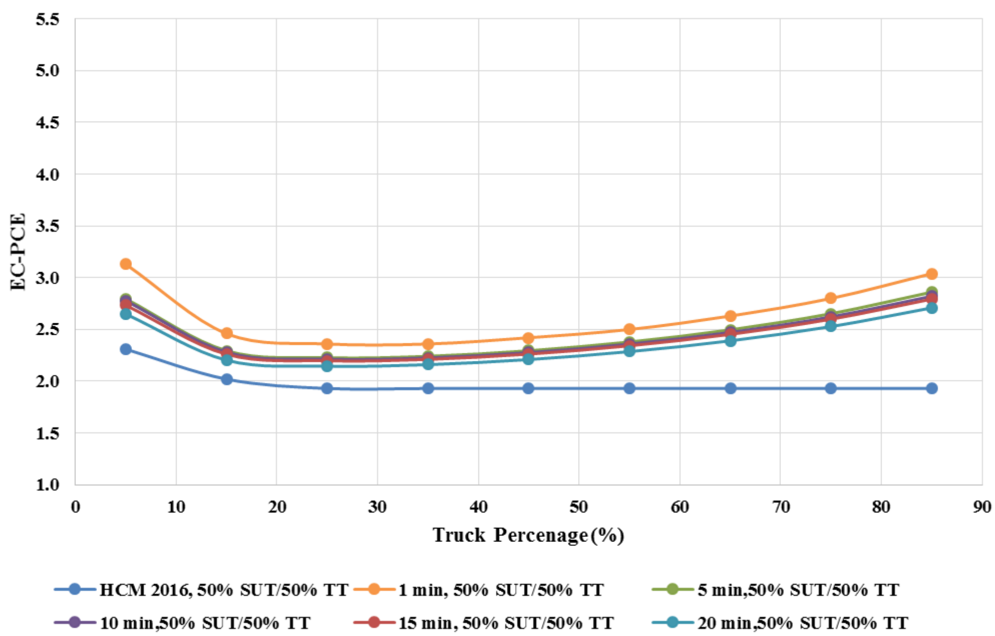
Table 9.12 EC_PCE results as a function of data aggregation interval level (0% grade and 1.5 mile grade length)

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.87	3.00	2.85	2.83	2.88	2.99	3.15	3.37	3.69
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.76	2.26	2.19	2.21	2.26	2.34	2.45	2.60	2.80
75 mph, 0%, 5 min	30% SUT, 70% TT	3.30	2.65	2.55	2.55	2.62	2.72	2.87	3.08	3.36
	50% SUT, 50% TT	2.80	2.29	2.23	2.24	2.29	2.38	2.50	2.65	2.86
	70% SUT, 30% TT	2.72	2.23	2.16	2.16	2.21	2.28	2.38	2.50	2.67
75 mph, 0%, 10 min	30% SUT, 70% TT	3.11	2.56	2.48	2.49	2.55	2.66	2.80	2.99	3.26
	50% SUT, 50% TT	2.78	2.28	2.21	2.23	2.28	2.36	2.47	2.62	2.82
	70% SUT, 30% TT	2.67	2.21	2.14	2.15	2.19	2.26	2.36	2.48	2.64
75 mph, 0%, 15 min	30% SUT, 70% TT	3.05	2.54	2.46	2.48	2.54	2.64	2.78	2.98	3.24
	50% SUT, 50% TT	2.73	2.26	2.20	2.21	2.26	2.34	2.45	2.60	2.79
	70% SUT, 30% TT	2.63	2.18	2.12	2.13	2.17	2.24	2.34	2.46	2.62
75 mph, 0%, 20 min	30% SUT, 70% TT	3.03	2.51	2.43	2.44	2.50	2.59	2.72	2.90	3.14
	50% SUT, 50% TT	2.65	2.20	2.15	2.16	2.21	2.29	2.39	2.53	2.71
	70% SUT, 30% TT	2.56	2.15	2.09	2.10	2.14	2.20	2.28	2.39	2.53

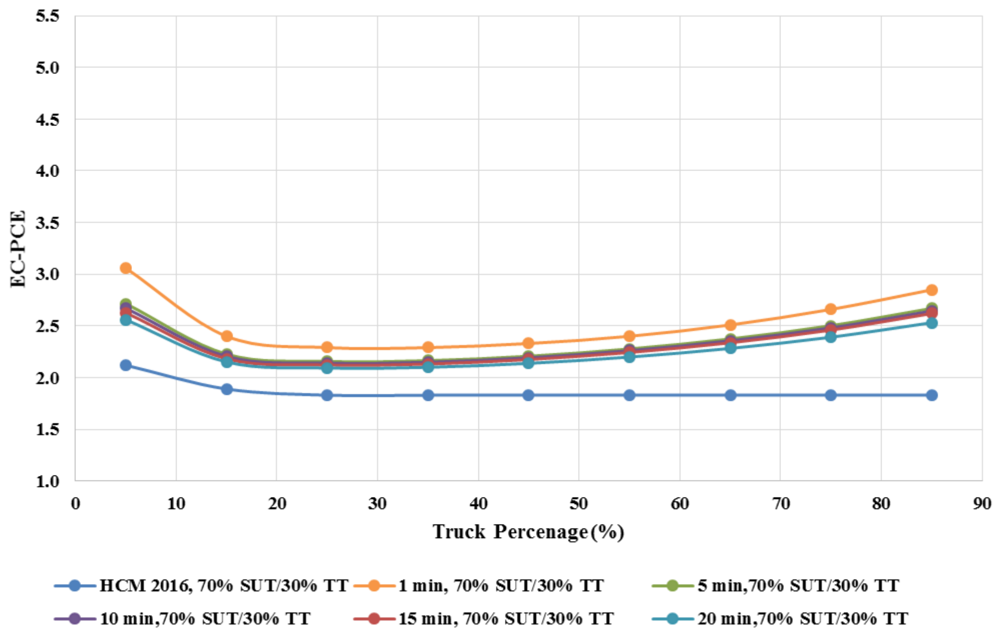
*Note: The marks mean for this group the EC_PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.



(a) Comparison of EC_PCEs among different data aggregation interval at 30% SUT/70% TT



(b) Comparison of EC_PCEs among different data aggregation interval at 50% SUT/50% TT



(c) Comparison of EC_PCEs among different data aggregation interval at 70% SUT/30% TT

Figure 9.7 Comparison of EC_PCEs among different data aggregation interval at (a) 30% SUT/70% TT (b) 50% SUT/50% TT and (c) 70% SUT/30% TT with 75 mph speed limit, no truck passing restriction, 0% grade, and 1.5 miles grade length

9.3.3 Impacts of Data Aggregation Interval on EC_PCEs

To explore the impacts of data aggregation intervals on EC_PCEs, a series of ANOVA and comparison tests are conducted based on F-tests and t-tests at the 95 percent level of confidence:

1) The differences in EC_PCEs among different truck percentage levels;
2) The differences in EC_PCEs among different truck composition levels;
3) The differences in EC_PCEs among different data aggregation interval levels;

4) The differences in EC_PCEs between pairs of different data aggregation interval levels (e.g. 1-minute interval vs. 15-minute interval).

The results for ANOVA and comparison tests are shown in Table 9.13.

Table 9.13 Results of ANOVA and comparison tests for EC_PCEs related to data aggregation interval

Group	Test	Test method	Test results	P-value
ANOVA	Different truck percentage	F-test	14.28	< 0.05
	Different truck composition	F-test	42.98	< 0.05
	Different data aggregation interval	F-test	2.83	0.027
Comparison	1-min v.s. 5-min interval	t-test	1.56	0.063
	1-min v.s. 10-min interval	t-test	2.03	0.024
	1-min v.s. 15-min interval	t-test	2.26	0.014
	1-min v.s. 20-min interval	t-test	2.83	0.003
	5-min v.s. 10-min interval	t-test	0.51	0.31
	5-min v.s. 15-min interval	t-test	0.78	0.22
	5-min v.s. 20-min interval	t-test	1.43	0.08
	10-min v.s. 15-min interval	t-test	0.27	0.39
	10-min v.s. 20-min interval	t-test	0.96	0.17
	15-min v.s. 20-min interval	t-test	0.69	0.25

*Note: The tests in this table are based on the EC_PCEs at 1-min, 5-min, 10-min, 15-min and 20-min aggregation interval data, 75 mph speed limit, no truck passing restriction, 0% grade, 1.5 miles.

It is found that with respect to the EC_PCEs values:

1) The EC_PCEs vary with truck percentage;

2) The EC_PCEs increase with percentage of heavy trucks;

3) The EC_PCEs vary with data aggregation interval;

4) The EC_PCEs with data aggregation intervals longer than one minute are lower than the EC_PCEs with one minute data aggregation intervals, which implies the EC_PCEs decrease with the increase in the length of data aggregation interval.

The differences are statistically significant at 95% level of confidence. The difference in EC_PCEs among using different data aggregation intervals can be attributed to, for the same dataset for one-hour traffic flow, if the data aggregation interval decreases, there will be more results and generally be more dispersed in measuring the hourly traffic flow. These might lead the difference in measuring the capacity, capacity adjustment factor (CAF) and then affect the EC_PCE estimation results.

9.4 Summary

This chapter conducted three sensitivity analyses to explore the impact that speed limit, truck passing restriction, and data aggregation level had on EC_PCEs estimated using the 2016 HCM PCE methodology under western rural U.S. conditions. It was assumed the simulated test freeway was on level terrain, had four lanes (e.g. two in each direction), had a considerable free flow speed differential between passenger cars and trucks because of speed limiters on the trucks, and experienced high truck percentages. These conditions are found across the western rural U.S. The study utilized data from the VISSIM 9.0 simulation model, which was calibrated for Nebraska traffic flow conditions based on I-80 empirical data.

The EC_PCEs were examined at four speed limit levels (e.g. 70, 75, 80 and 85 mph), five truck passing restriction levels (e.g. 0%, 25%, 50%, 75% and 100% of the roadway network), and five data aggregation levels (e.g. 1, 5, 10, 15 and 20 minutes). It was found that, the EC_PCEs increase with the speed limit, decrease with the increase in the percentage of truck passing restriction, and decrease as data aggregation level increases. All of these relationships were found to be statistically significant at the 95 percent level of confidence.

In summary, the results indicate that for western rural U.S. conditions:

1) If the speed limit is increased then the trucks will affect the passenger cars more severely, all else being equal;
2) If truck passing restrictions are implemented, similar to what occurs in Europe, the negative effects of trucks can be mitigated; and
3) The data aggregation level has an effect on the PCE values, all else being equal.

CHAPTER 10 CONCLUDING REMARKS

10.1 Conclusions

The freeway PCEs in HCM 2010 and HCM 2016 may not be appropriate for four-lane level freeway segments in western rural U.S. due to the existence of moving bottlenecks. These occur when trucks pass other trucks at low speed differentials, on freeway segments with two lanes per direction, high truck percentages and large speed differences between passenger cars and trucks (e.g. Table 1.1). This dissertation aims at estimating new PCEs for four-lane level freeway segments under western rural U.S. traffic conditions with localized congestion caused by moving bottlenecks. The conclusions are summarized in the following sections.

1. Moving bottlenecks identification

In this dissertation, on four-lane level rural U.S. freeway segments, the critical leading and lagging headways are used to categorize whether a vehicle is in a moving bottleneck or not. Vehicles on all lanes in one direction with leading headways less than or equal to the critical leading headways, or with lagging headways less than or equal to critical lagging headways, are considered to belong to the same moving bottleneck. Critical headways are determined by the rules that 1) vehicles with headways smaller than the critical headway experience lower speed, and 2) there is a linear relationship between speed and headway among those slower vehicles. This dissertation assumes that there is not a single critical headway value. It was found the critical headway is a function of vehicle type and their relative positions (e.g. car following truck or truck following truck). The values of critical headways range from 3.0s to 8.0s. The lowest value (3.0s) is the car-following-car leading critical headway. The highest value (8.0s) is the car-following-truck leading critical headway. Results of moving bottleneck identification analysis show that 87.1% of vehicles are impeded in moving bottlenecks. Based on moving bottleneck identification, the average speeds for both cars and trucks as impeded vehicles are significantly lower than as non-impeded vehicles. Also, the amount and percentage of impeded vehicles, as well as percentage of time impeding, increase with traffic volume.

2. The effects of moving bottlenecks on traffic flow

To explore the effects of moving bottlenecks on traffic flow, three categories of moving bottleneck characteristics are proposed:

1) Speed-related moving bottleneck characteristics, including the speed of impeded and non-impeded vehicles, the difference in speed of impeded and non-impeded vehicles, and the ratio of impeded vehicle speed to free flow speed;
2) Number and density of impeded vehicles in moving bottleneck; and
3) Characteristics related to moving bottleneck existence, including moving bottleneck existence time, moving bottleneck existence distance, and moving bottleneck-caused-delay experienced by passenger cars with 75 mph free flow speed.

The analyses showed that, on average, vehicles impeded in moving bottlenecks led by two trucks (Type IV moving bottleneck) experience the highest speed reduction, degree of congestion, and moving bottleneck delay. It was also found that moving bottlenecks led by two trucks have the longest moving bottleneck length, existence time, and distance.

Additionally, this dissertation proposed the metric “Density uniformity factor” (DUF) to measure the localized congestion caused by moving-bottlenecks on four-lane freeway segments. A high value of DUF means the density tends to be uniform and there is no obvious localized congestion.

3. PCEs under western rural U.S. traffic flow conditions with moving bottlenecks

In this dissertation, the equal-density-based PCEs (ED_PCEs) are calculated using simulation data from CORSIM 6.3 and VISSIM 9.0. The ED_PCE is based on the HCM 2010 methodology. Six levels of traffic volume are simulated beginning at 500 veh/h/ln and increasing to 1500 veh/h/ln. Nine levels of truck percentage are simulated beginning at 5 percent, and increasing to 85 percent at 10 percent increments. The simulation models are calibrated by genetic algorithms based on Nebraska empirical data. The results of ED_PCEs show:

1) With CORSIM 6.3 data under a single free-flow speed condition, the ED_PCEs are marginally higher than the recommended values in HCM 2010;
2) With VISSIM 9.0 data under a single free-flow speed distribution condition, the ED_PCEs are marginally higher than the HCM 2010 values and similar to the values from the CORSIM 6.3 analysis;
3) With VISSIM 9.0 data under empirical free-flow speed distributions condition, the ED_PCEs are higher than the HCM 2010 values. Not surprisingly they are higher than the CORSIM 6.3 analysis and VISSIM 9.0 single free-flow speed analysis value. All the differences were statistically significant at the 95% level of confidence.
4) On rural western U.S. four-lane level freeway segments, the ED_PCEs increase with traffic volume but do not vary appreciably with truck percentage.

In this research the higher calculated ED_PCE values better capture the relationship between trucks and passenger cars on this corridor, because the operational analysis results show the LOS based on the new ED_PCEs are lower than that based on the recommended values in HCM 2010.

The EC_PCEs are calculated by simulation data from VISSIM 9.0 under the HCM 2016 and western rural U.S. conditions. The EC_PCEs are calculated using the same methodology that was used to calculate the PCEs in the 2016 HCM. The simulation models are calibrated using the same procedures as were used to calculate the ED_PCEs. It was found that for western rural U.S. four-lane level freeway segments, the EC_PCEs increase as the percentage of heavy trucks increase. Under western rural U.S. conditions, the EC_PCEs increase with the grade length, and decrease with the truck percentage up to 25 percent then slightly increase with the truck percentage. Also, the EC_PCEs under western rural U.S. conditions are higher than the HCM 2016 conditions and the HCM 2016 recommended values. These differences are statistically significant at the 95% level of confidence. It is argued that the higher proposed EC_PCEs better capture the relationship between trucks and passenger cars for western rural U.S. freeways as compared with the HCM 2016 PCE values.

The density under western rural U.S. conditions were found to be much more “non-uniform” than under HCM 2016 conditions based on simulation data. This implies that for western rural U.S. conditions, localized congestion is much more severe than for HCM 2016 conditions. Based on the analyses in this dissertation, the HCM 2016 PCEs are recommended for use when the DUF is 0.7 or higher.

4. The impacts of speed limits, truck passing restriction and data aggregation intervals on EC_PCEs

Four speed limit levels were examined in this dissertation (70 mph, 75 mph, 80 mph, and 85 mph). It was assumed that only the speeds of passenger cars are affected by speed limits. It was found that the EC_PCEs vary with truck percentage, increase with the percentage of heavy trucks, and increase with the speed limit. All of these differences were found to be statistically significant at the 95% level of confidence.

Five truck passing restriction levels were examined in this dissertation (0%, 25%, 50%, 75%, and 100% of total distance). The truck passing restrictions are implemented by “lane restrictions”, which means trucks are restricted to the rightmost lanes and may not pass. It was found that the EC_PCEs vary with the truck percentage for all truck passing restriction levels, increase with the percentage of heavy trucks for all truck passing restriction levels, and decrease with the truck passing restriction percentage. All of these differences were found to be statistically significant at the 95% level of confidence.

Five data aggregation levels were examined in this dissertation (1, 5, 10, 15, and 20 minute interval). The EC-PCE analyses show the EC_PCEs vary with the truck percentage for all data aggregation interval levels, increase with the percentage of heavy trucks for all data aggregation interval levels, and decrease with the length of data aggregation interval. All of these differences were found to be statistically significant at the 95% level of confidence.

10.2 Contributions

The contributions of this research include:

1) Proposing new moving bottlenecks identification methods for four-lane freeways;
2) Proposing new metrics for analyzing moving bottlenecks impacts, and proposing "density uniformity factor" (DUF) to measure localized congestions;
3) Developing VISSIM simulation models for level freeway segments, calibrated to western rural U.S. conditions to capture moving bottlenecks;
4) Using a calibrated VISSIM model to examine the impacts of moving bottlenecks, including:
   a) Calculating PCEs using equal-density (2) and equal-capacity (3) methods under western rural U.S. traffic conditions;
   b) Exploring the applicable conditions of HCM 2016 recommended PCEs based on DUF; and
   c) Exploring the impacts of speed limits, truck passing restrictions and data aggregation intervals on PCEs.

The results will be helpful in understanding how trucks affect passenger cars and how moving bottlenecks affect traffic flow on four-lane level freeway segments.

10.3 Recommendations for Future Research

Recommended improvements for future research to get more accurate results include:

1) In critical headway determination, a non-linear regression model for speed-headway relationships need to be developed to get a more accurate critical headway value to replace the integer formats in this research;
2) During the simulation process in this research, the vehicles were classified into

passenger cars, single-unit trucks, and heavy trucks, because of the limitations of the video detector software. In future research, a more elaborated vehicle classification (e,g. Class 2-13) based on the number of axles and weight-to-horsepower ratio may be beneficial;

3) In the ED_PCEs and EC_PCEs estimation, only the PCEs at level freeway segments are estimated. In future research, the impacts of grade or other possible influencing factors on PCEs may be discussed. Also, the PCEs based on other existing methodologies (e.g. delay, headway, etc.) might be estimated;

4) For the impacts of truck restriction, in this research only the impacts of “lane restriction” options, which result in truck passing restrictions, are discussed. In the future, the impacts of other restriction options on PCEs, such as time or route restrictions, may be discussed.

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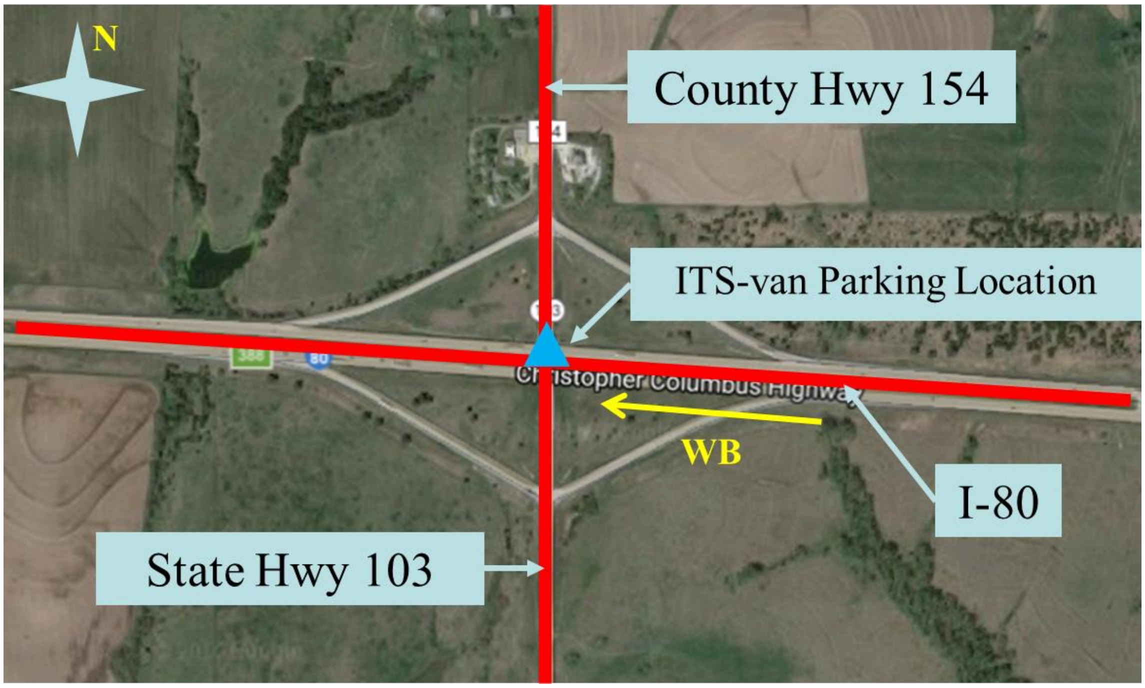
APPENDIX A DETAILS OF WESTERN RURAL U.S. FREEWAY SEGMENTS

State	Interstate Number	Segments*
AR	I-40	Oklahoma-Arkansas border to Exit 147 (Little Rock City)
AZ	I-8	California-Arizona border to Exit 178A (Arizona City)
	I-10	California-Arizona border to Exit 112 (Phoenix)
	I-10	Exit 279 (Tucson) to Arizona-New Mexico border
	I-15	Utah-Arizona border to Arizona-Nevada border
	I-17	Exit 337 (Flagstaff) to Exit 232 (Phoenix) to
	I-40	California-Arizona border to Arizona-New Mexico border
CO	I-25	Exit 128 (Colorado Springs) to Colorado-Oklahoma border
	I-70	Utah-Colorado border to Exit 240 (Idaho Springs)
	I-70	Exit 304 (Denver) to Colorado-Kansas border
	I-76	Exit 20 (Denver) to Colorado-Nebraska border
ID	I-15	Montana-Idaho border to Idaho-Utah border
	I-84	Oregon-Idaho border to Idaho-Utah border
	I-86	I-84 & I-86 interchange to Exit 58 (Pocatello)
	I-90	Washington-Idaho border to Idaho-Montana border
KS	I-35	Exit 210 (Kansas City) to Kansas-Oklahoma border
	I-70	Colorado-Kansas border to Exit 353 (Topeka)
	I-135	Exit 88 (Salina) to Exit 19 (Wichita)
LA	I-20	Texas-Louisiana border to Exit 85 (Monroe)
MT	I-15	Canada-U.S. border to Montana-Idaho border
	I-90	Idaho-Montana border to Montana-Wyoming border
	I-94	I-90 & I-94 interchange (Billings) to Montana-North Dakota border
ND	I-29	Canada-U.S. border to North Dakota-South Dakota border
	I-94	Montana-North Dakota border to North Dakota-Minnesota border
NE	I-80	Wyoming-Nebraska border to Exit 397 (Lincoln)
NM	I-10	Arizona-New Mexico border to Exit 132 (Las Cruces)
	I-25	Colorado-New Mexico border Exit 9 (Las Cruces)
	I-40	Arizona-New Mexico border to New Mexico-Texas border
NV	I-15	Exit 58 (Las Vegas) to Nevada-Arizona border
	I-80	California-Nevada border to Nevada-Utah border
OK	I-35	Kansas-Oklahoma border to Oklahoma-Texas border

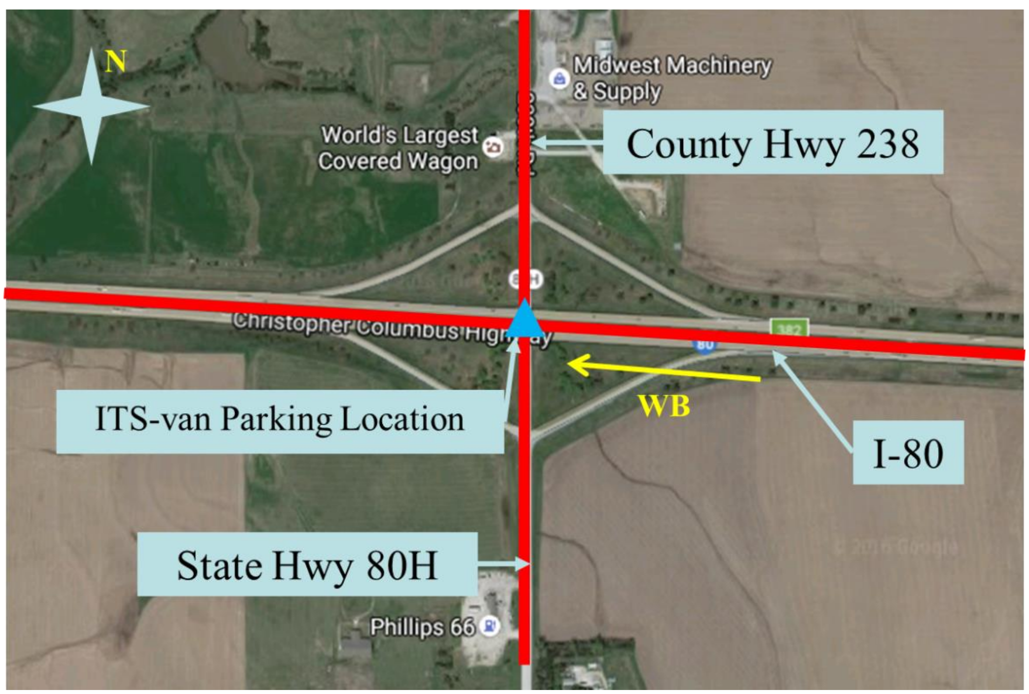
	I-40	Texas-Oklahoma border to Oklahoma-Arkansas border
	I-44	Exit 158 (Oklahoma City) to Oklahoma-Illinois border
SD	I-29	North Dakota-South Dakota border to South Dakota-Iowa border
	I-90	Wyoming-South Dakota border to South Dakota-Minnesota border
TX	I-10	New Mexico border to Exit 540 (San Antonio)
	I-10	Exit 591 (San Antonio) to Exit 729 (Houston)
	I-10	Exit 829 (Winnie) to Texas-Louisiana border
	I-20	I-10 & I-20 interchange to Texas-Louisiana border
	I-30	Exit 77A (Dallas) to Texas-Arkansas border
	I-35	Exit 140 (San Antonio) to U.S.-Mexico border
	I-37	Exit 125 (San Antonio) to Exit 17 (Corpus Christi)
	I-40	New Mexico-Texas border to Texas-Oklahoma border
	I-45	Exit 231 (Corsicana) to Exit 116 (Huntsville)
UT	I-15	Idaho-Utah border to Exit 362 (Ogden)
	I-15	Exit 242 (Provo) to Utah-Arizona border
	I-70	I-15 & I-70 interchange to Utah-Colorado border
	I-80	Nevada-Utah border to Utah-Wyoming border
	I-84	Idaho-Utah border to I-15 & I-84 interchange (Tremonton)
WY	I-25	Montana-Wyoming border to Wyoming-Colorado border
	I-80	Utah-Wyoming border to Wyoming-Nebraska border
	I-90	Exit 56B (Buffalo) to Wyoming-South Dakota border

*Parts of the segments listed here may locate in urban area. The freeway segments in urban area are not considered as western rural U.S. freeway segments.

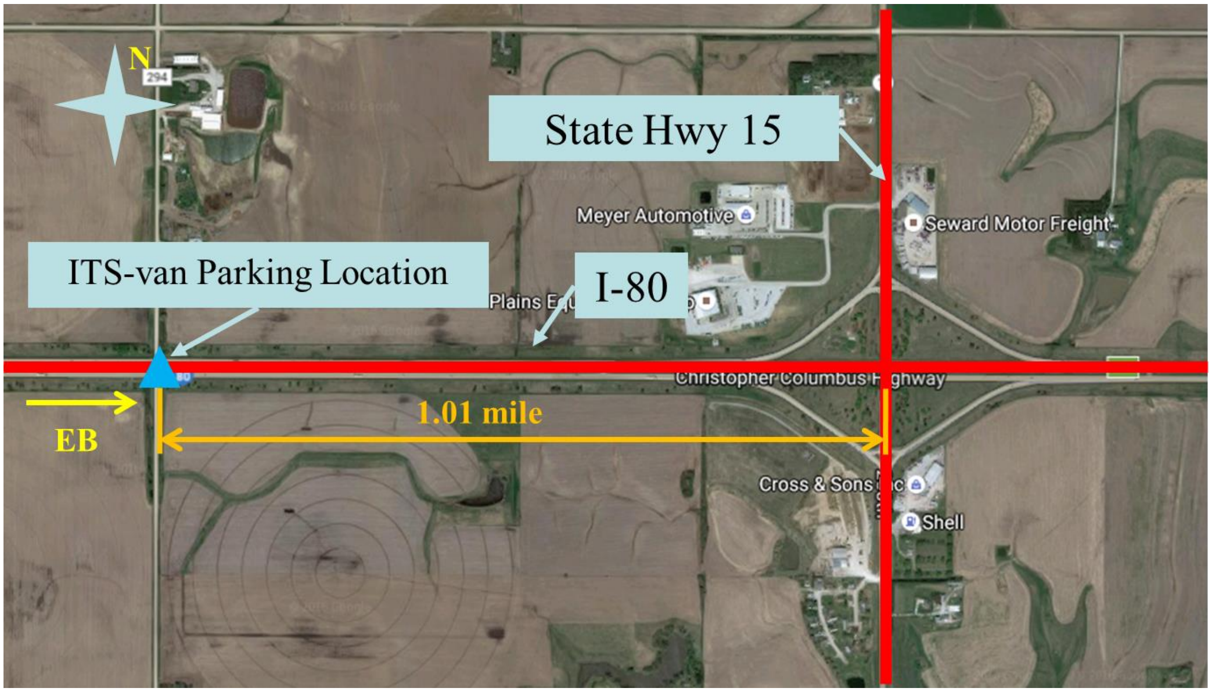
APPENDIX B DATA COLLECTION SITES



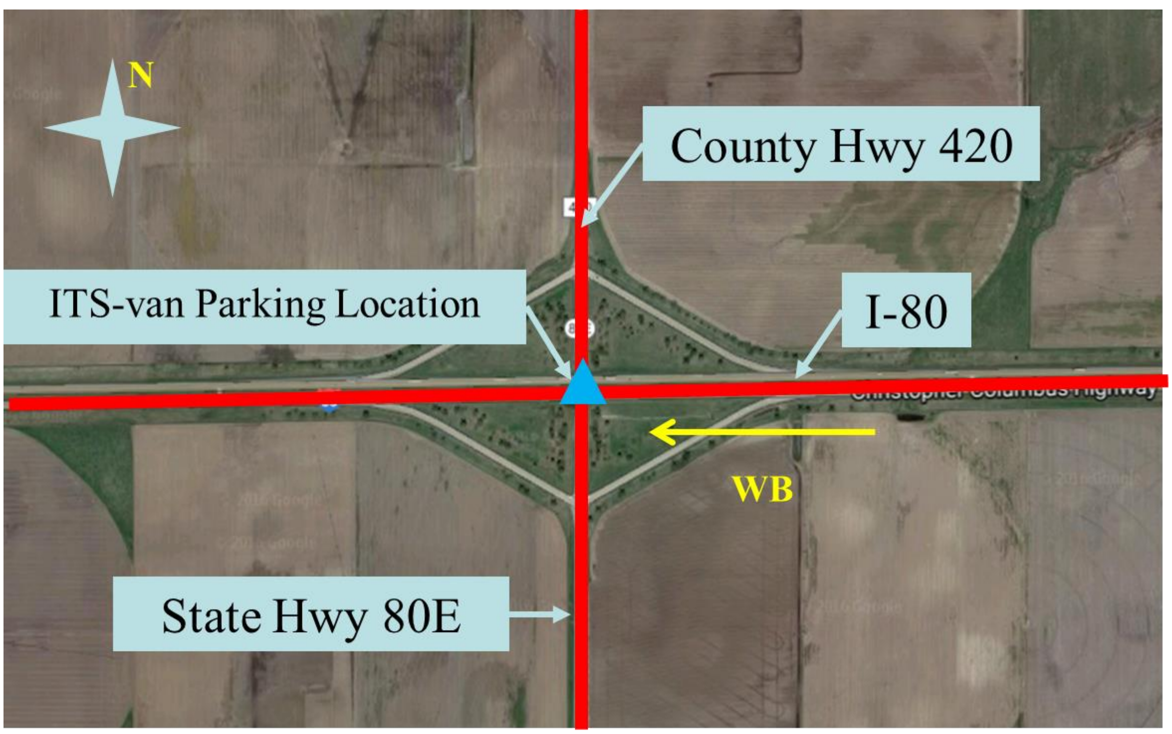
(a) Pleasantdale, NE



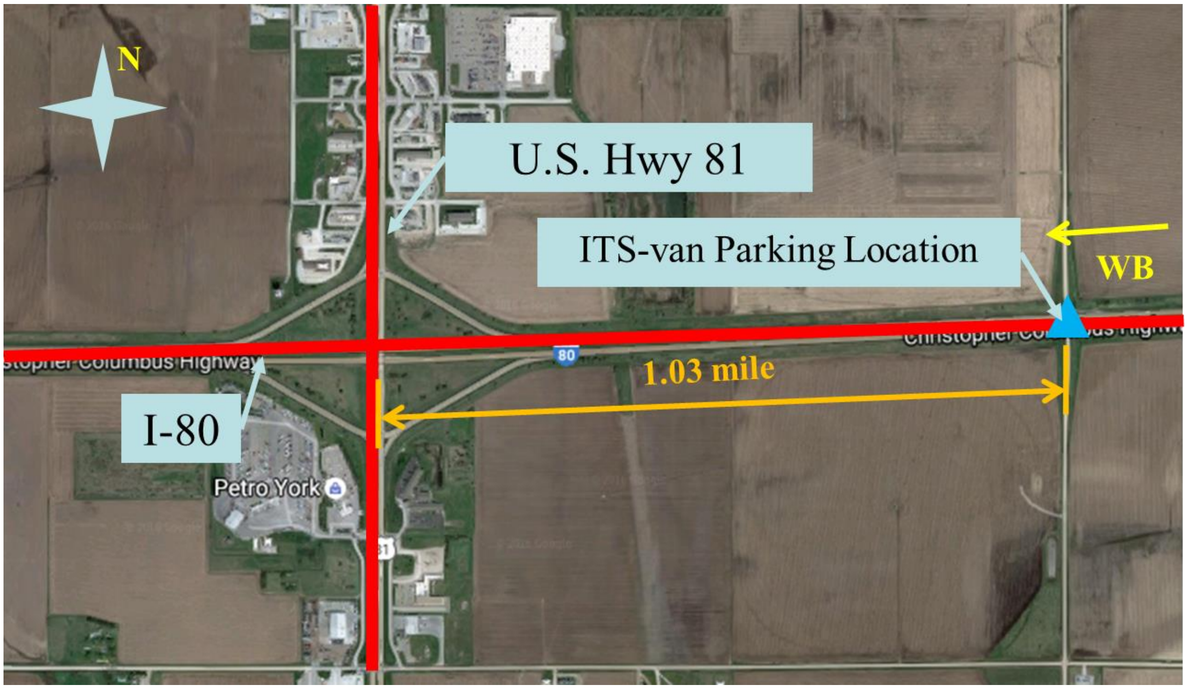
(b) Milford, NE



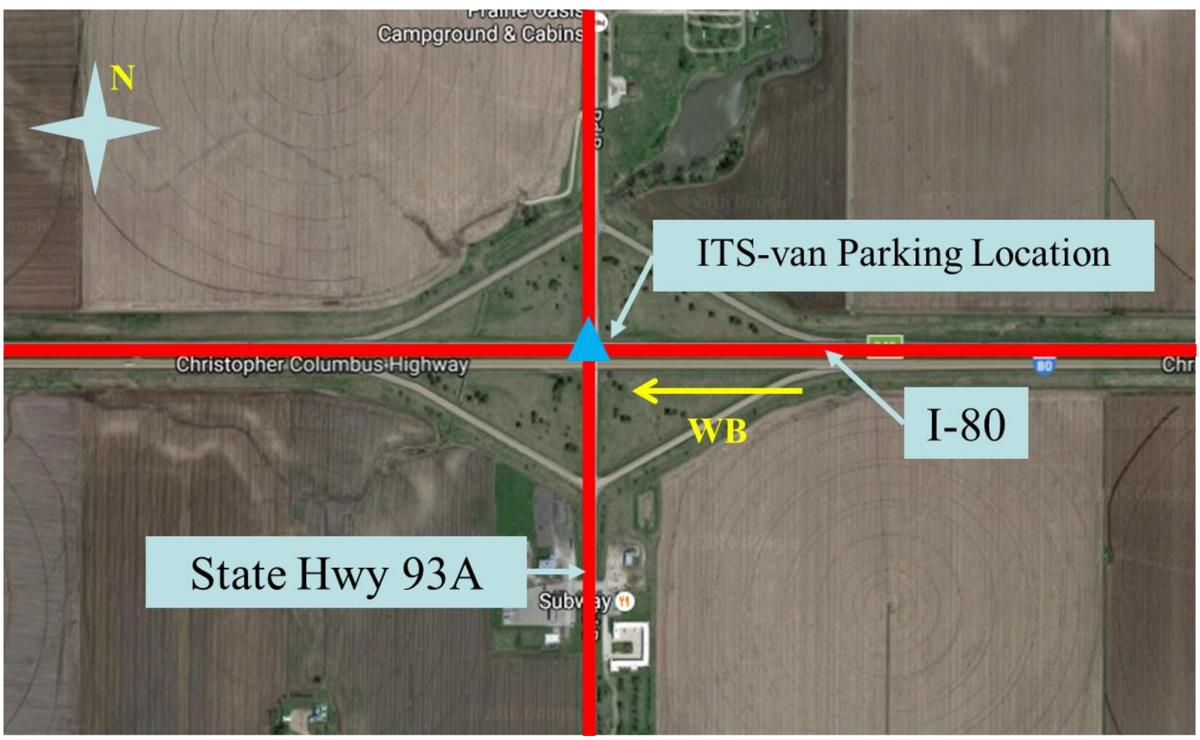
(c) Seward, NE



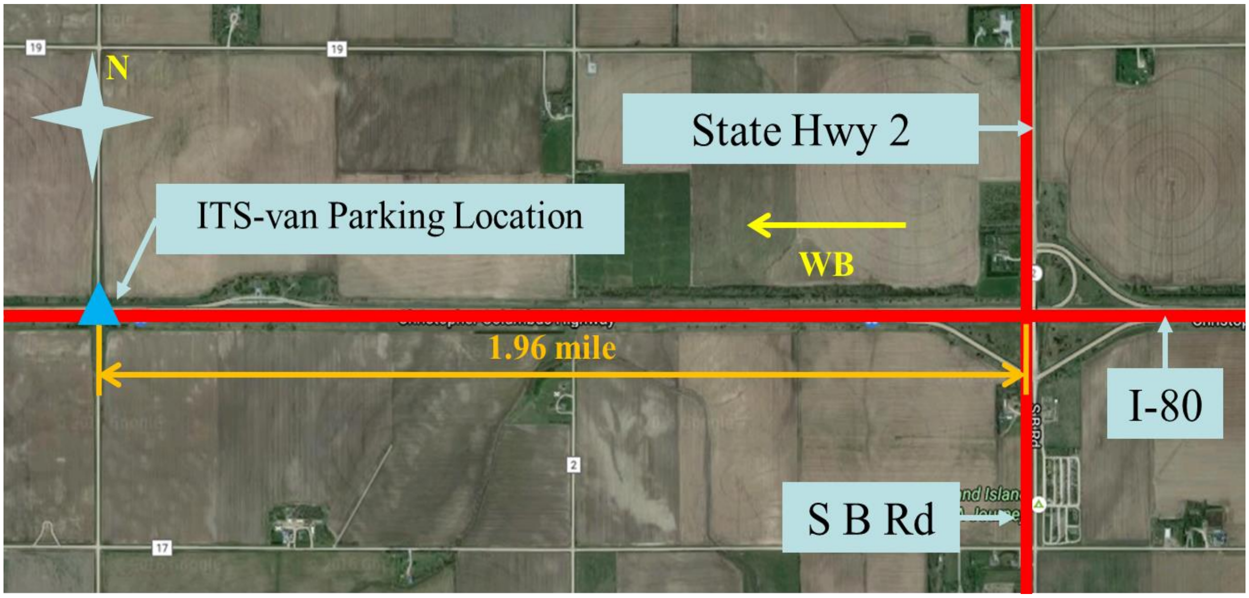
(d) Beaver Crossing, NE



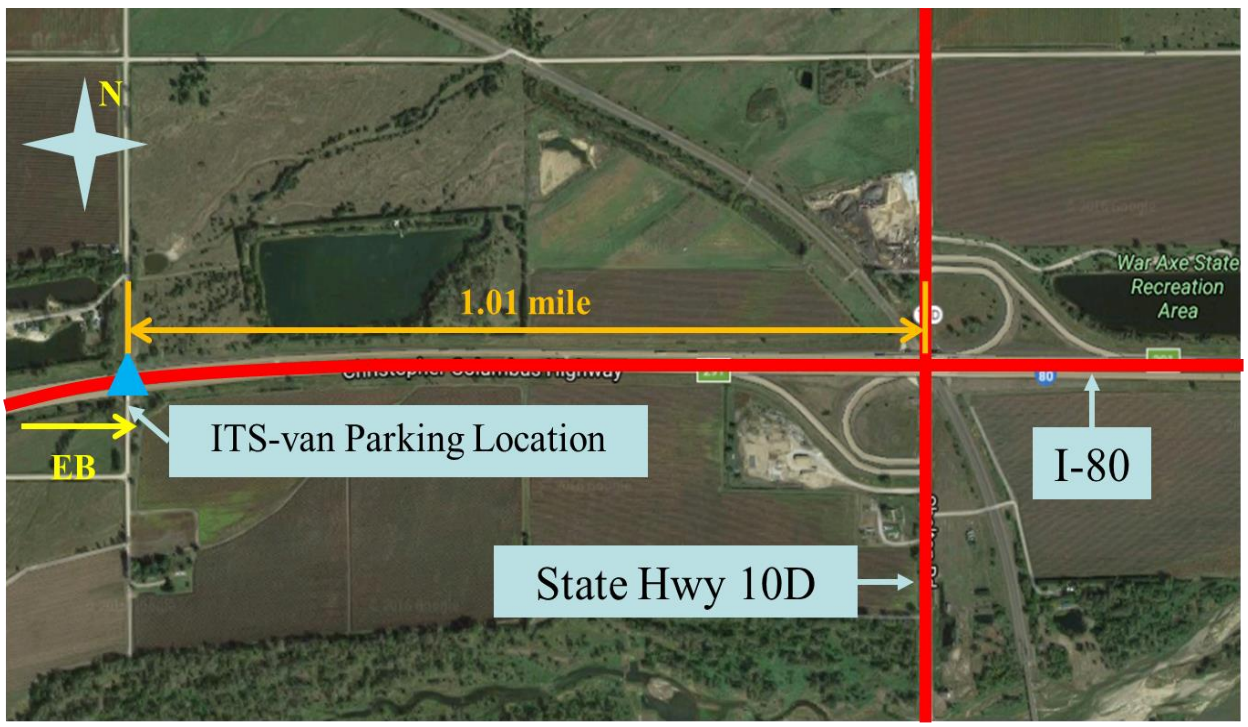
(e) York, NE



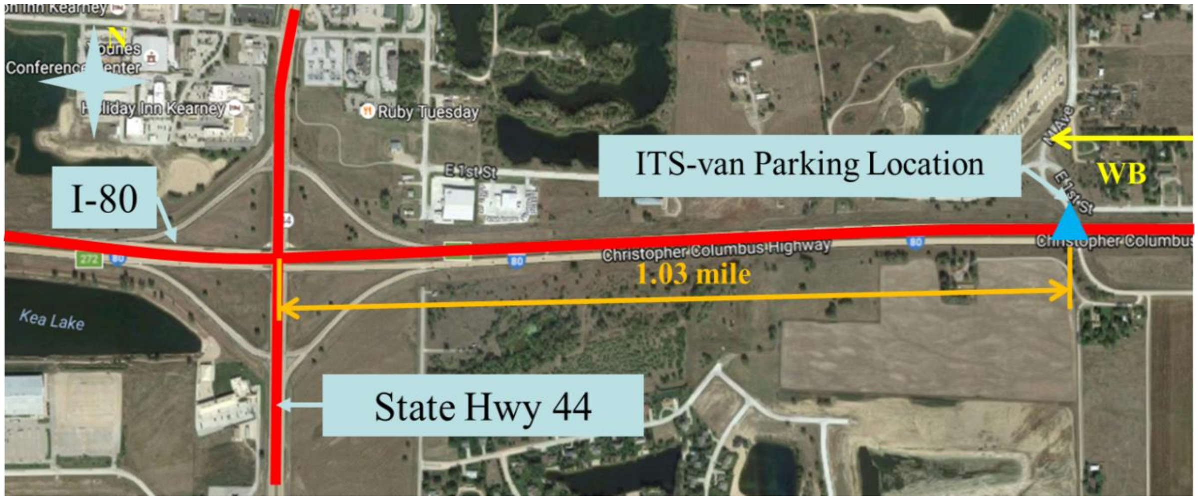
(f) Henderson, NE



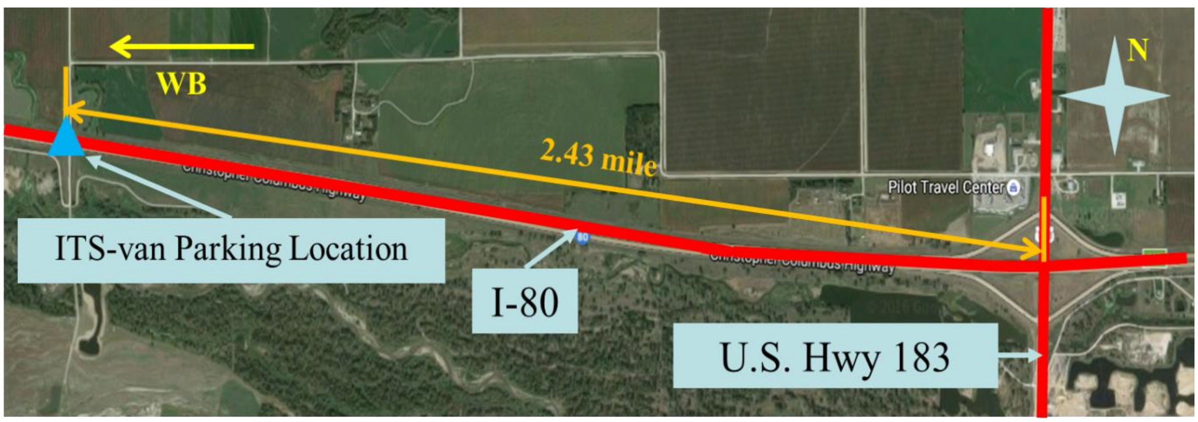
(g) Grand Island, NE



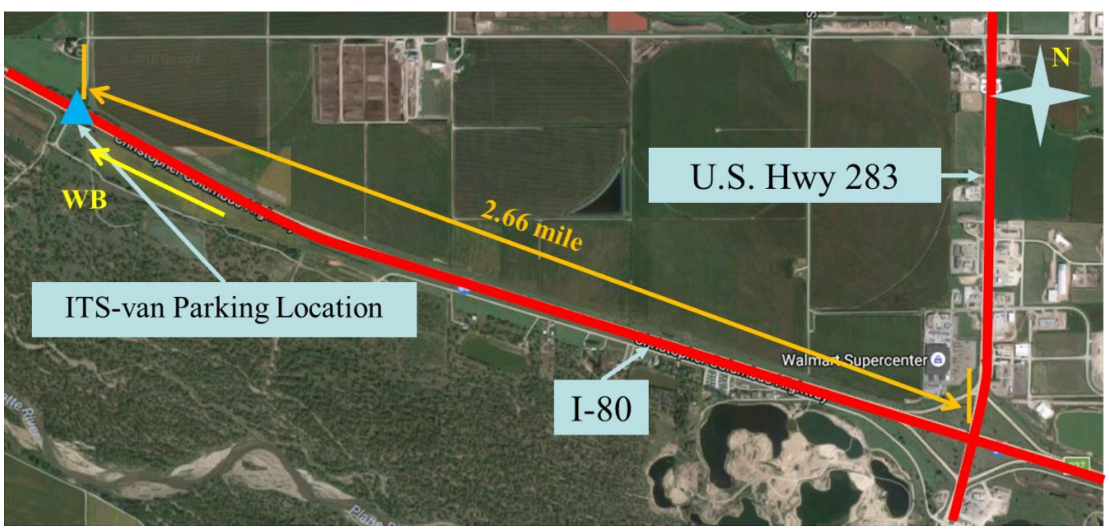
(h) Shelton, NE



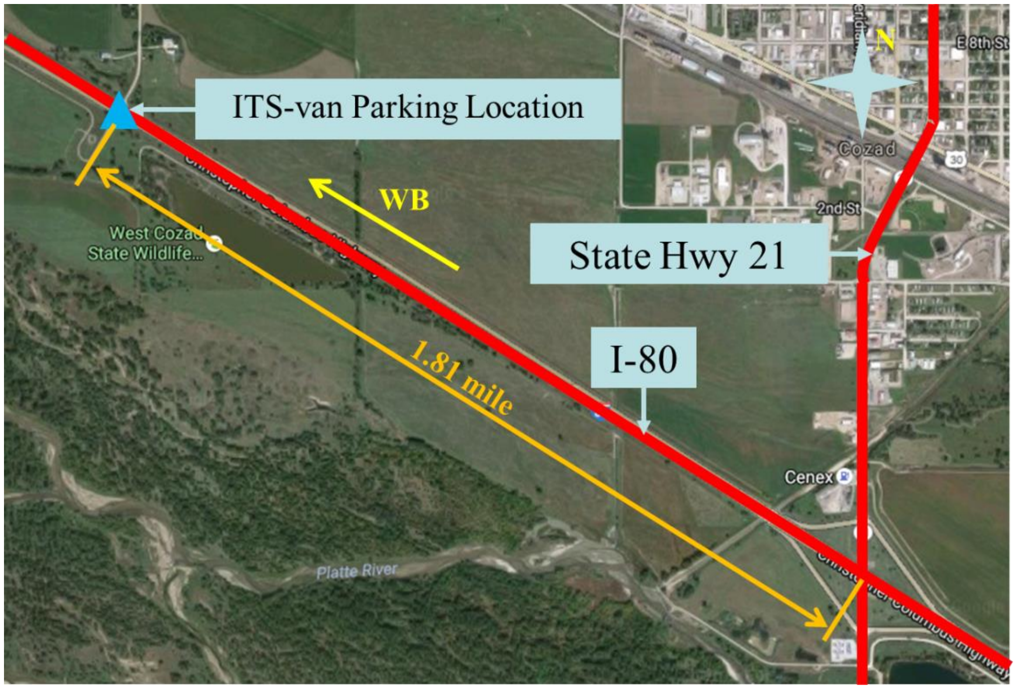
(i) Kearney, NE



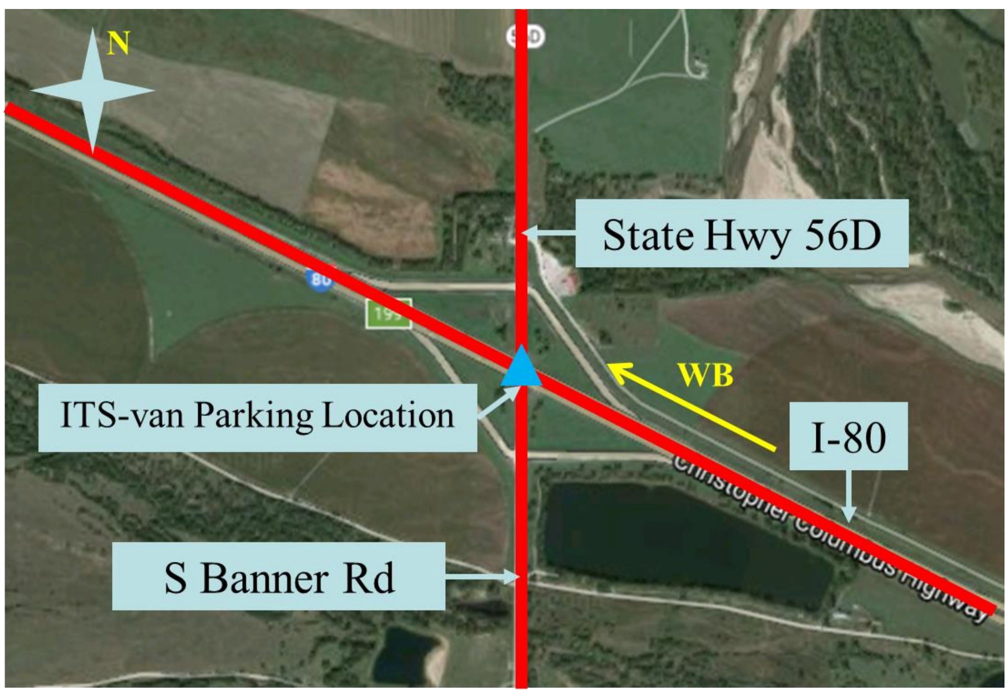
(j) Elm Creek, NE



(k) Lexington, NE

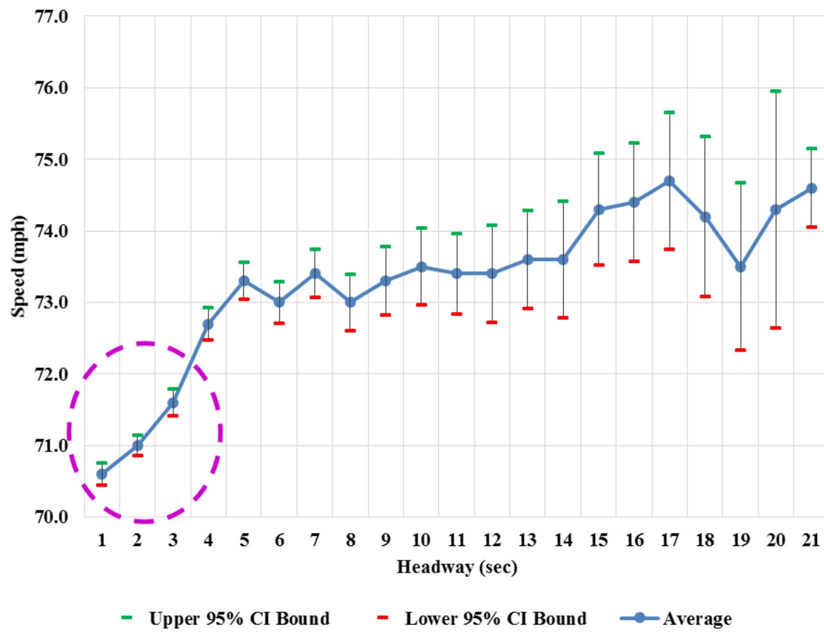


(l) Cozad, NE

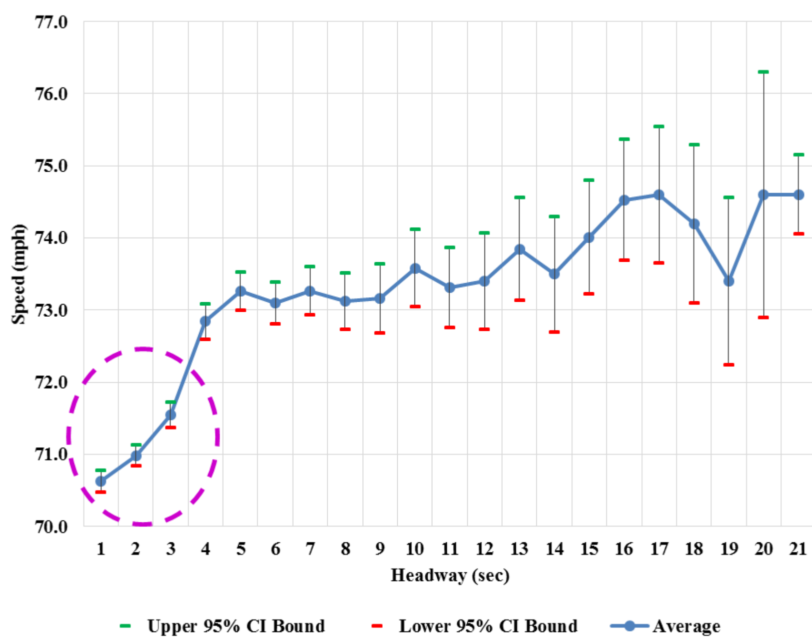


(m) Brady, NE

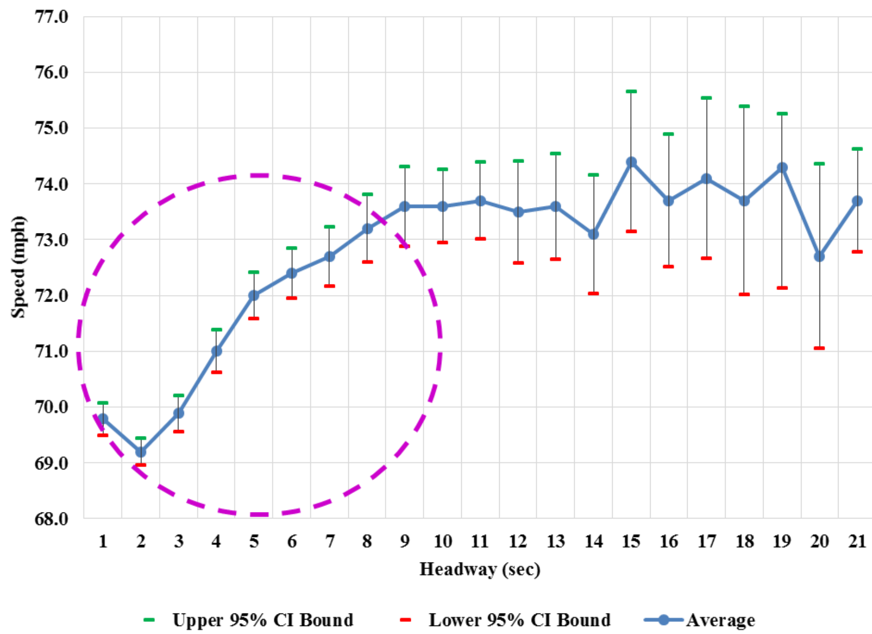
APPENDIX C RELATIONSHIP BETWEEN AVERAGE SPEED AND HEADWAY FOR DIFFERENT HEADWAY TYPE



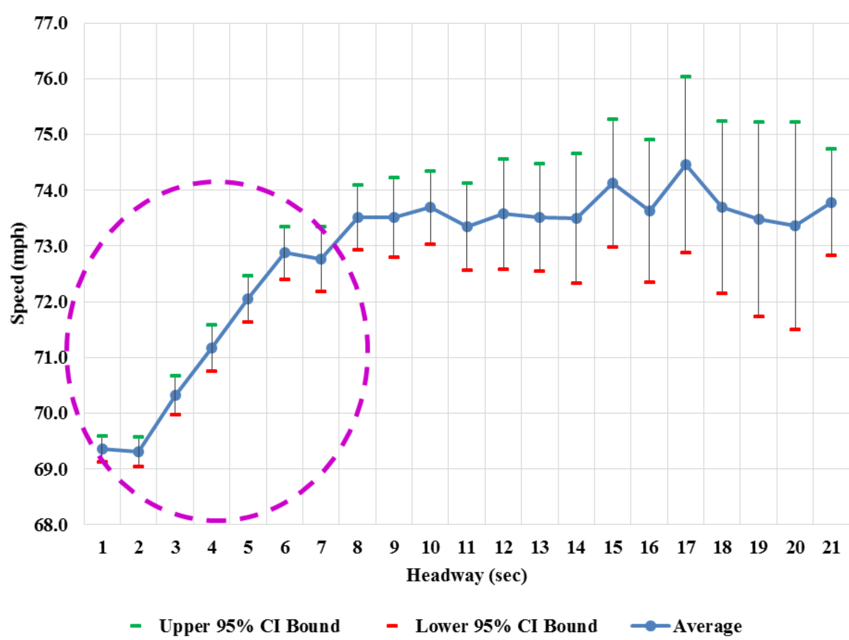
(a) cc-leading headway



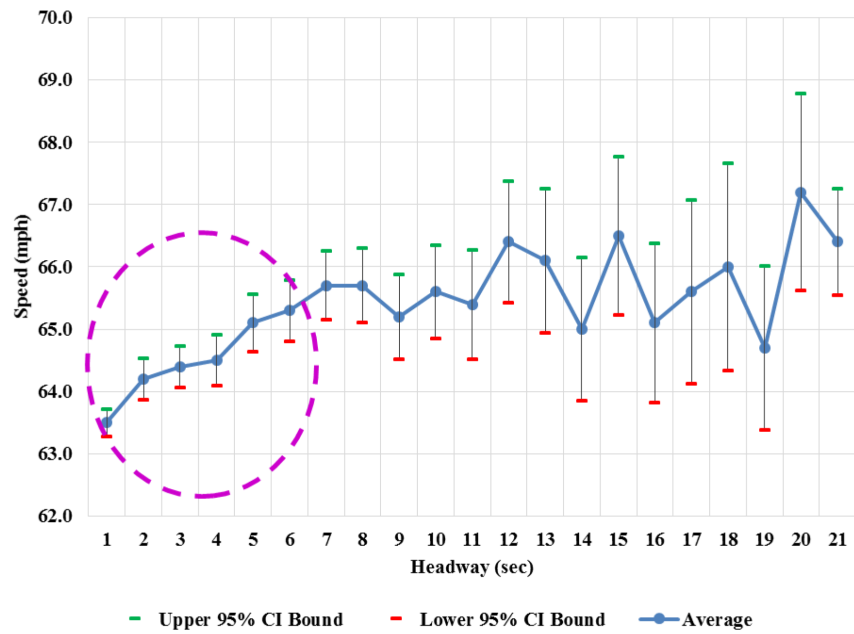
(b) cc-lagging headway



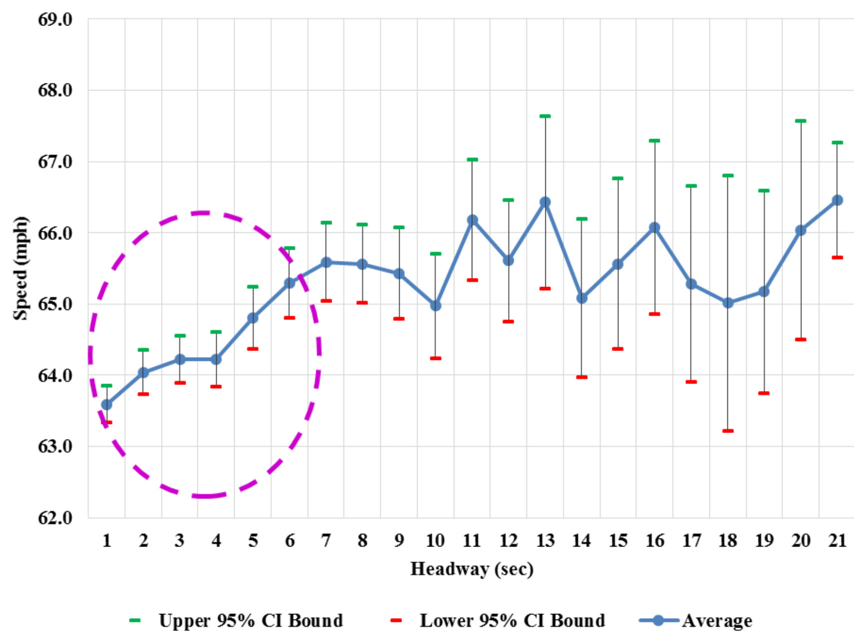
(c) ct-leading headway



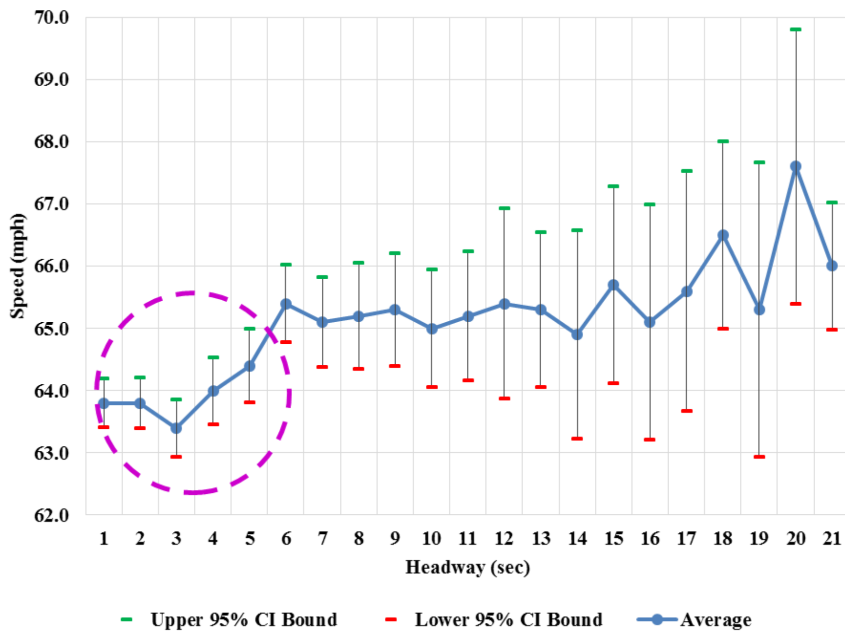
(d) ct-lagging headway



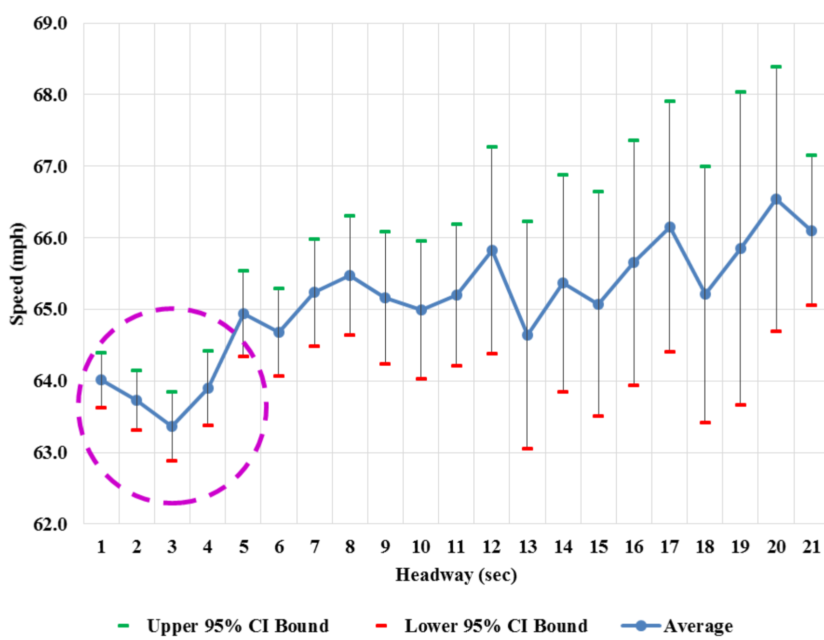
(e) tc-leading headway



(f) tc-lagging headway

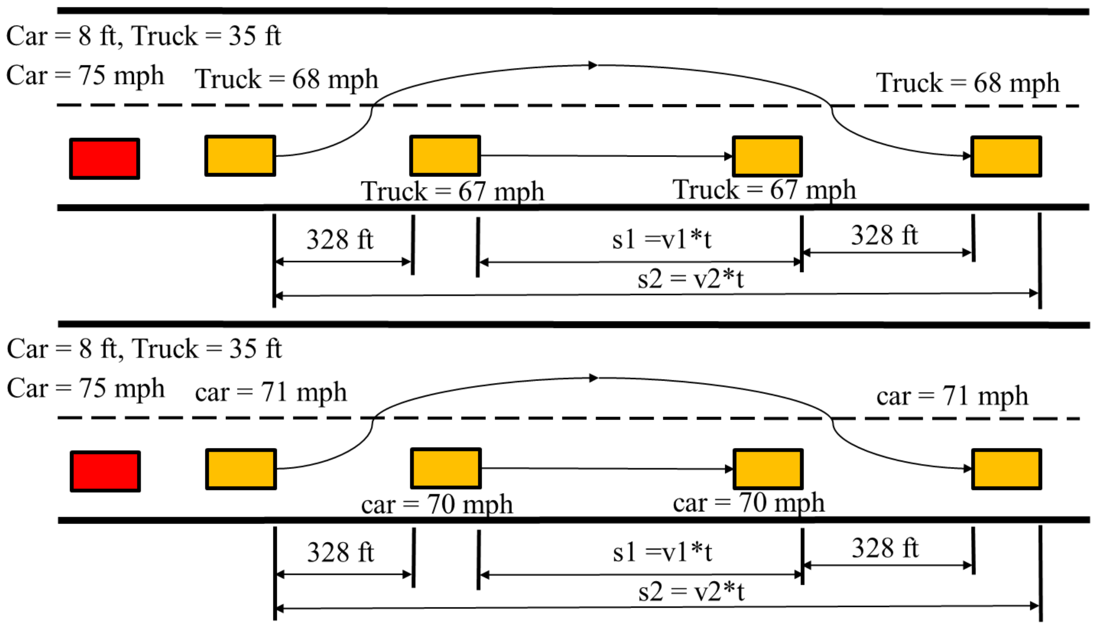


(g) tt-leading headway

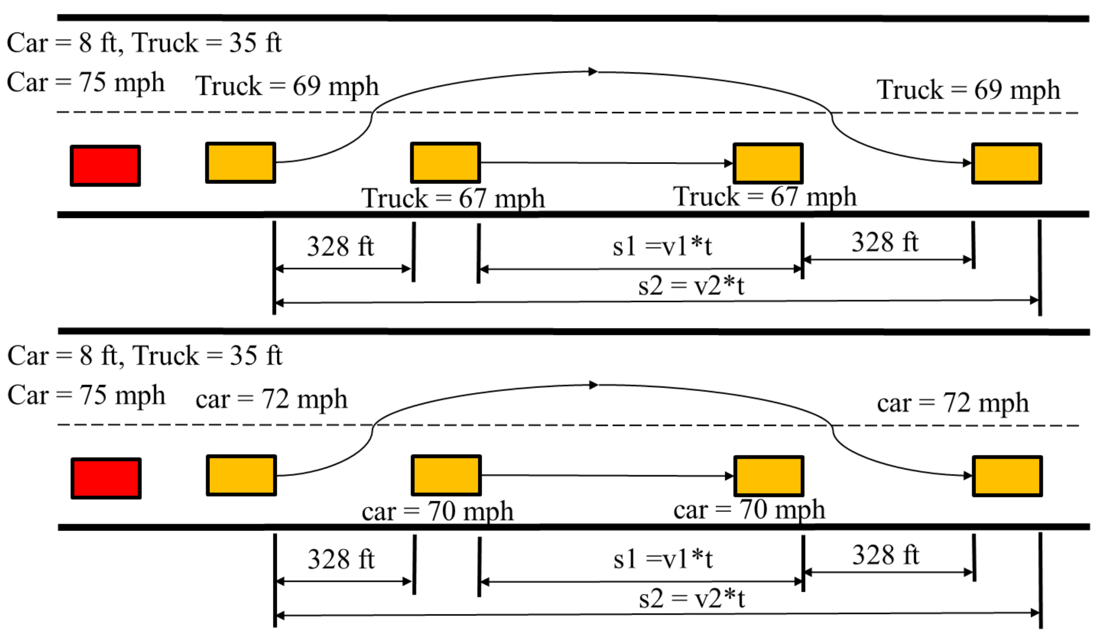


(h) tt-lagging headway

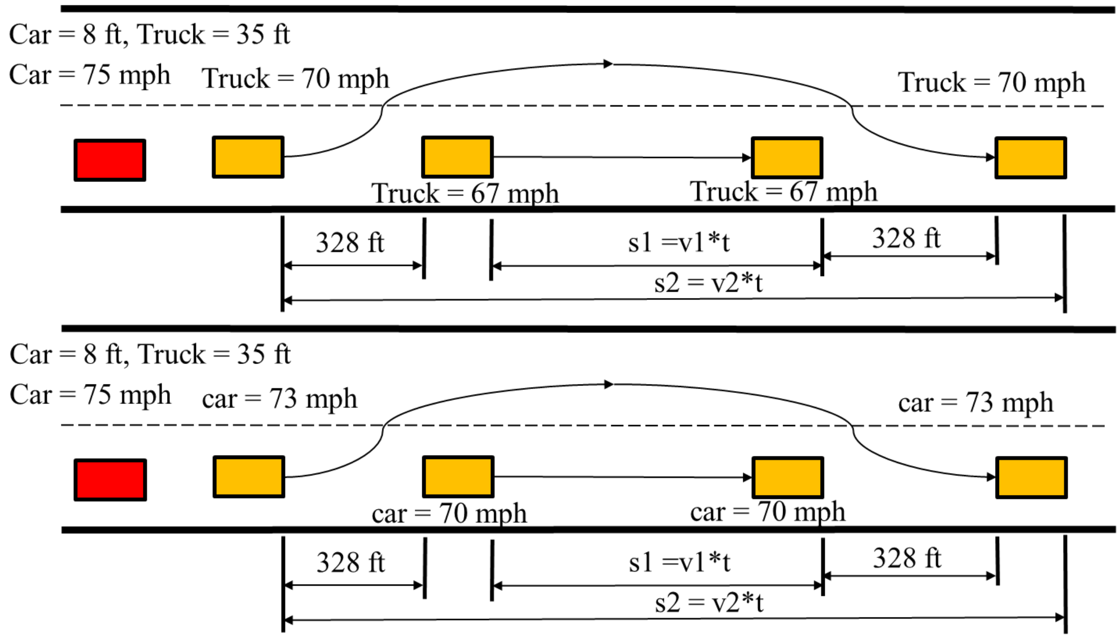
APPENDIX D THEORETICAL ANALYSIS FOR MOVING BOTTLENECK EXISTENCE TIME AND DISTANCE ON TWO LANES WITH DIFFERENT MOVING BOTTLENECK SPEED DIFFERENCE



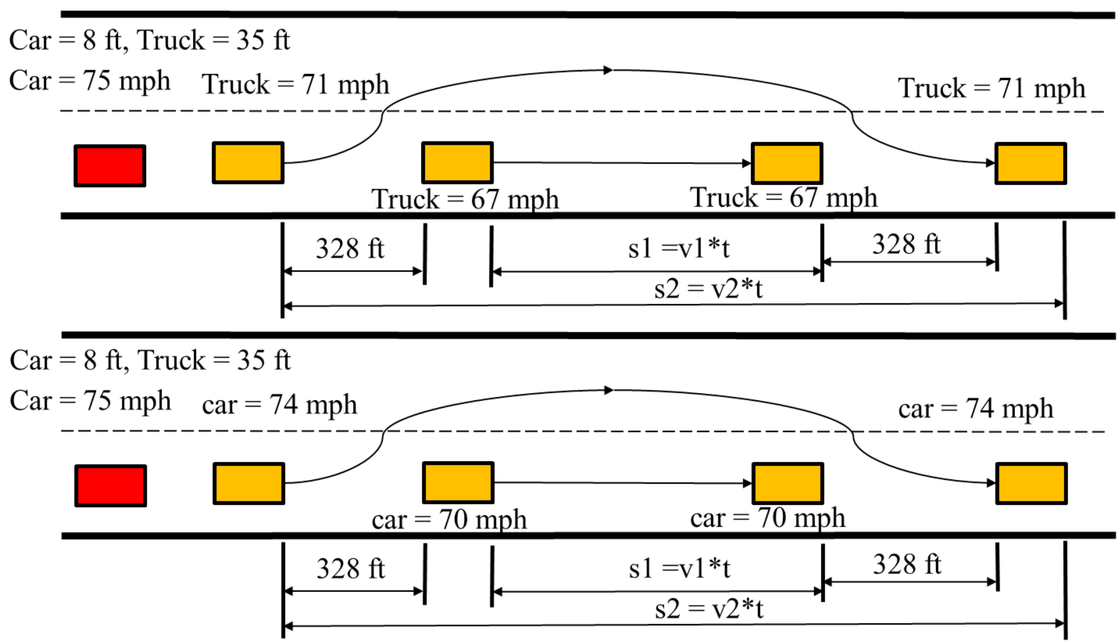
(a) 1 mph



(b) 2 mph

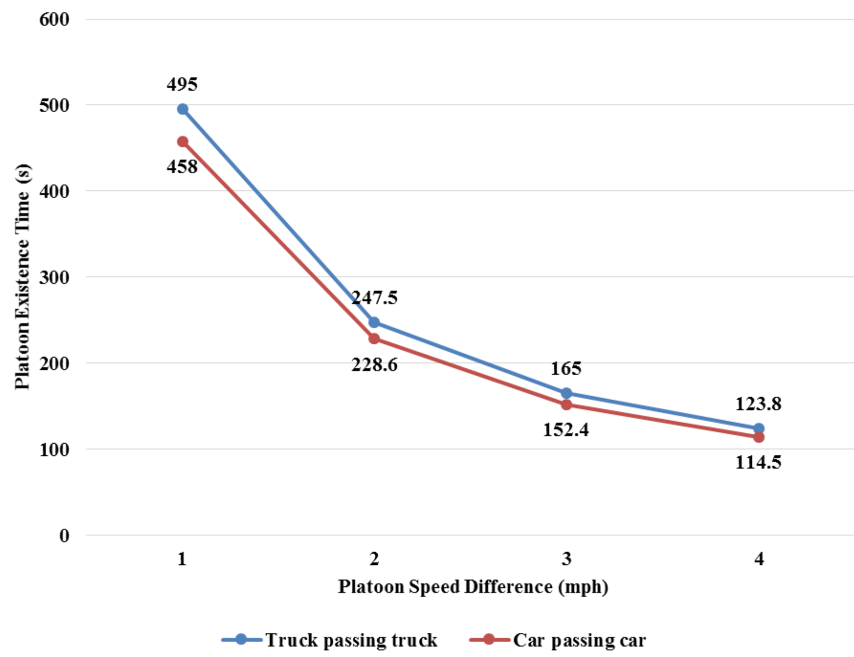

Car = 8 ft, Truck = 35 ft
Car = 75 mph
Truck = 70 mph
Truck = 70 mph
Truck = 67 mph
Truck = 67 mph
328 ft
s1 =v1*t
328 ft
s2 = v2*t
Car = 8 ft, Truck = 35 ft
Car = 75 mph
car = 73 mph
car = 73 mph
car = 70 mph
car = 70 mph
328 ft
s1 =v1*t
328 ft
s2 = v2*t

(c) 3 mph

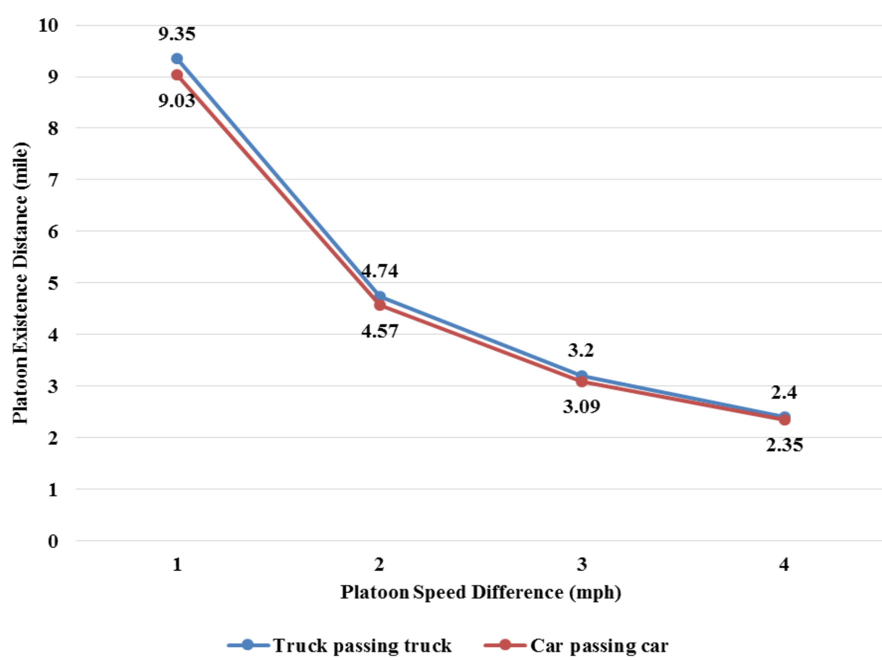

Car = 8 ft, Truck = 35 ft
Car = 75 mph
Truck = 71 mph
Truck = 71 mph
Truck = 67 mph
Truck = 67 mph
328 ft
s1 =v1*t
328 ft
s2 = v2*t
Car = 8 ft, Truck = 35 ft
Car = 75 mph
car = 74 mph
car = 74 mph
car = 70 mph
car = 70 mph
328 ft
s1 =v1*t
328 ft
s2 = v2*t

(d) 4 mph

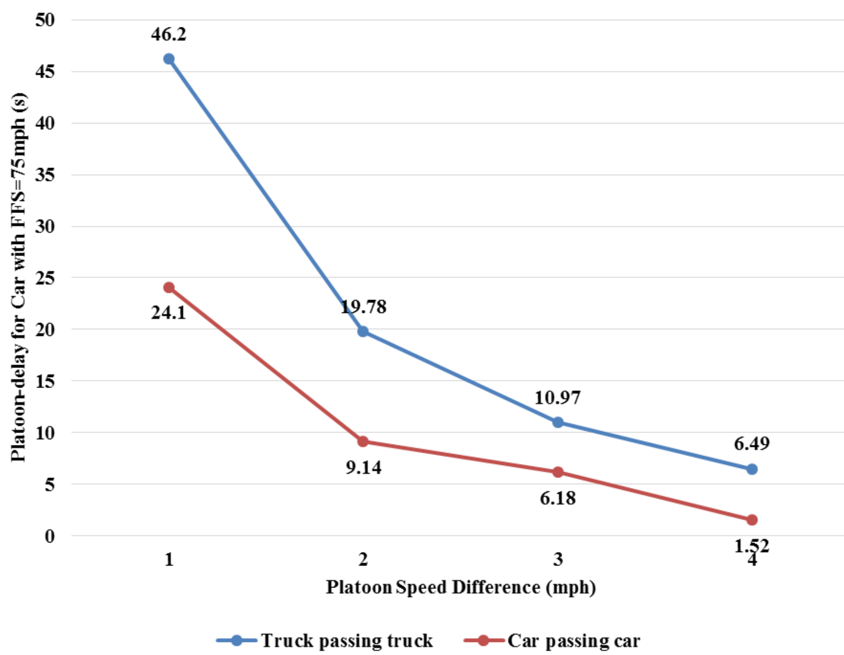
APPENDIX E RESULTS OF THEORETICAL ANALYSIS FOR EFFECTS OF PLATOON SPEED DIFFERENCE



(a) Moving bottleneck existence time



(b) Moving bottleneck existence distance



(c) Moving bottleneck delay for cars with FFS = 75 mph

APPENDIX F VOLUME-DENSITY CURVES IN ED_PCES ESTIMATION

(a) Developed volume-density curves in ED_PCEs estimation using CORSIM 6.3 with a single free-flow speed

Truck Percentage (%)	Term (Volume)	Number of Simulation									
		1	2	3	4	5	6	7	8	9	10
0 (Base flow)	Quadratic	-0.42	-0.430	-0.412	-0.423	-0.415	-0.413	-0.411	-0.415	-0.419	-0.413
	Linear	76.021	74.017	74.304	74.013	74.420	76.854	77.820	74.419	75.221	75.743
5	Quadratic	-0.412	-0.423	-0.404	-0.415	-0.407	-0.405	-0.404	-0.408	-0.411	-0.405
	Linear	74.400	72.539	72.753	72.483	72.753	75.080	76.080	72.645	73.631	73.966
10	Quadratic	-0.406	-0.416	-0.398	-0.408	-0.401	-0.399	-0.397	-0.402	-0.405	-0.399
	Linear	72.363	70.344	70.518	70.578	70.525	73.004	73.882	70.672	71.476	72.084
15	Quadratic	-0.398	-0.408	-0.390	-0.401	-0.393	-0.391	-0.390	-0.395	-0.398	-0.391
	Linear	70.049	67.828	68.010	68.100	68.441	70.753	71.631	68.526	68.938	69.543
20	Quadratic	-0.390	-0.400	-0.382	-0.391	-0.384	-0.382	-0.382	-0.386	-0.389	-0.382
	Linear	68.300	66.017	66.136	66.358	66.619	69.036	69.857	66.755	67.079	67.678
25	Quadratic	-0.382	-0.391	-0.375	-0.384	-0.376	-0.374	-0.374	-0.379	-0.382	-0.374
	Linear	66.393	64.027	64.063	64.627	64.634	67.101	67.946	65.002	65.140	65.725
30	Quadratic	-0.378	-0.387	-0.371	-0.380	-0.372	-0.370	-0.370	-0.374	-0.377	-0.370
	Linear	64.714	62.261	62.401	62.840	63.040	65.508	66.344	63.447	63.544	63.970
35	Quadratic	-0.375	-0.384	-0.367	-0.376	-0.369	-0.366	-0.366	-0.371	-0.374	-0.367
	Linear	63.266	60.904	60.874	61.507	61.718	64.153	64.820	61.860	62.065	62.443
40	Quadratic	-0.368	-0.377	-0.361	-0.369	-0.362	-0.359	-0.360	-0.365	-0.368	-0.361
	Linear	61.409	58.886	59.185	59.769	59.964	62.217	63.115	59.917	60.368	60.750
45	Quadratic	-0.363	-0.372	-0.356	-0.363	-0.357	-0.353	-0.355	-0.360	-0.363	-0.355
	Linear	60.060	57.428	57.780	58.383	58.593	60.822	61.718	58.617	59.021	59.276
50	Quadratic	-0.356	-0.365	-0.350	-0.356	-0.351	-0.347	-0.349	-0.352	-0.356	-0.348
	Linear	58.706	56.037	56.525	56.941	57.368	59.416	60.443	57.244	57.575	58.011
55	Quadratic	-0.350	-0.359	-0.344	-0.351	-0.345	-0.341	-0.344	-0.347	-0.350	-0.343
	Linear	57.557	54.927	55.451	55.701	56.301	58.362	59.194	56.043	56.392	56.843
60	Quadratic	-0.345	-0.354	-0.340	-0.345	-0.339	-0.336	-0.338	-0.342	-0.345	-0.338
	Linear	56.113	53.406	53.906	54.235	54.860	56.779	57.686	54.572	55.087	55.503
65	Quadratic	-0.340	-0.349	-0.334	-0.340	-0.333	-0.330	-0.333	-0.336	-0.339	-0.333
	Linear	54.906	52.165	52.635	53.100	53.744	55.664	56.574	53.248	53.775	54.300
70	Quadratic	-0.333	-0.342	-0.327	-0.333	-0.327	-0.324	-0.326	-0.329	-0.333	-0.326

	Linear	53.688	50.864	51.340	51.966	52.644	54.476	55.437	52.029	52.460	53.108
75	Quadratic	-0.329	-0.338	-0.323	-0.329	-0.323	-0.320	-0.322	-0.325	-0.329	-0.322
	Linear	52.695	49.852	50.299	50.874	51.619	53.528	54.442	51.036	51.522	52.194
80	Quadratic	-0.326	-0.335	-0.320	-0.326	-0.320	-0.317	-0.320	-0.323	-0.326	-0.320
	Linear	51.771	48.909	49.465	49.900	50.684	52.601	53.490	50.070	50.680	51.358
85	Quadratic	-0.323	-0.332	-0.318	-0.324	-0.317	-0.314	-0.317	-0.320	-0.323	-0.317
	Linear	51.004	48.152	48.771	49.174	49.899	51.759	52.767	49.298	49.979	50.539
90	Quadratic	-0.320	-0.329	-0.314	-0.320	-0.313	-0.311	-0.313	-0.316	-0.320	-0.313
	Linear	50.087	47.275	47.835	48.247	49.019	50.794	51.904	48.309	49.032	49.559

(b) Developed volume-density curves in ED_PCEs estimation using CORSIM 6.3 with a single free-flow speed (continue)

Truck Percentage (%)	**Term (Volume)**	**Number of Simulation**									
		11	12	13	14	15	16	17	18	19	20
0 (Base flow)	Quadratic	-0.410	-0.427	-0.413	-0.422	-0.419	-0.430	-0.427	-0.423	-0.418	-0.423
	Linear	75.222	74.173	77.592	75.215	74.629	77.777	75.437	77.302	76.684	77.070
5	Quadratic	-0.402	-0.419	-0.405	-0.413	-0.412	-0.422	-0.420	-0.414	-0.410	-0.414
	Linear	73.503	72.496	75.937	73.753	73.086	76.156	73.931	75.682	74.941	75.579
10	Quadratic	-0.395	-0.413	-0.399	-0.407	-0.405	-0.416	-0.415	-0.408	-0.404	-0.408
	Linear	71.446	70.560	73.737	71.914	70.855	74.250	71.767	73.762	72.889	73.703
15	Quadratic	-0.388	-0.405	-0.391	-0.400	-0.398	-0.408	-0.407	-0.401	-0.397	-0.400
	Linear	69.078	68.293	71.286	69.499	68.731	71.885	69.462	71.656	70.413	71.476
20	Quadratic	-0.380	-0.397	-0.382	-0.390	-0.390	-0.400	-0.398	-0.393	-0.389	-0.392
	Linear	67.273	66.543	69.488	67.651	66.973	70.032	67.552	69.881	68.566	69.740
25	Quadratic	-0.372	-0.390	-0.374	-0.382	-0.382	-0.392	-0.391	-0.385	-0.380	-0.384
	Linear	65.263	64.556	67.501	65.616	64.933	68.000	65.554	68.067	66.814	67.835
30	Quadratic	-0.368	-0.386	-0.370	-0.378	-0.378	-0.388	-0.386	-0.380	-0.377	-0.380
	Linear	63.594	62.969	65.984	63.823	63.364	66.448	63.744	66.245	65.053	66.284
35	Quadratic	-0.364	-0.383	-0.367	-0.375	-0.375	-0.385	-0.383	-0.377	-0.373	-0.377
	Linear	62.005	61.609	64.511	62.482	61.913	65.020	62.218	64.656	63.611	64.808
40	Quadratic	-0.357	-0.377	-0.360	-0.369	-0.368	-0.379	-0.376	-0.370	-0.367	-0.370
	Linear	60.259	59.729	62.736	60.531	59.928	63.231	60.440	62.653	61.912	63.000
45	Quadratic	-0.353	-0.372	-0.356	-0.363	-0.363	-0.373	-0.372	-0.365	-0.361	-0.365
	Linear	58.960	58.462	61.384	59.069	58.581	61.951	59.116	61.249	60.479	61.541

50	Quadratic	-0.345	-0.364	-0.349	-0.357	-0.356	-0.366	-0.365	-0.358	-0.354	-0.358
	Linear	57.589	57.155	60.046	57.669	57.248	60.478	57.815	59.981	58.993	60.133
55	Quadratic	-0.339	-0.359	-0.344	-0.351	-0.350	-0.360	-0.359	-0.352	-0.348	-0.352
	Linear	56.451	56.104	58.937	56.566	56.134	59.384	56.780	58.756	57.919	59.049
60	Quadratic	-0.334	-0.353	-0.339	-0.346	-0.345	-0.355	-0.353	-0.347	-0.343	-0.347
	Linear	54.996	54.599	57.459	55.118	54.593	57.853	55.262	57.181	56.391	57.567
65	Quadratic	-0.328	-0.348	-0.332	-0.341	-0.340	-0.349	-0.347	-0.341	-0.337	-0.342
	Linear	53.733	53.395	56.218	53.875	53.370	56.660	54.167	55.909	55.069	56.297
70	Quadratic	-0.322	-0.341	-0.326	-0.334	-0.334	-0.343	-0.341	-0.334	-0.330	-0.336
	Linear	52.467	52.181	55.042	52.597	52.169	55.383	52.911	54.810	53.899	55.197
75	Quadratic	-0.317	-0.337	-0.322	-0.330	-0.329	-0.339	-0.337	-0.330	-0.326	-0.332
	Linear	51.502	51.236	53.952	51.543	51.094	54.368	51.843	53.850	52.871	54.219
80	Quadratic	-0.314	-0.334	-0.319	-0.328	-0.326	-0.336	-0.334	-0.327	-0.323	-0.329
	Linear	50.504	50.310	52.983	50.532	50.138	53.461	50.981	52.837	51.878	53.207
85	Quadratic	-0.312	-0.331	-0.316	-0.325	-0.323	-0.333	-0.331	-0.324	-0.320	-0.326
	Linear	49.778	49.469	52.233	49.793	49.406	52.760	50.211	52.021	51.056	52.424
90	Quadratic	-0.308	-0.328	-0.312	-0.321	-0.320	-0.330	-0.328	-0.321	-0.317	-0.323
	Linear	48.864	48.569	51.339	48.964	48.445	51.885	49.365	51.167	50.213	51.533

(c) Developed volume-density curves in ED_PCEs estimation using VISSIM 9.0 with a single free-flow speed distribution

Truck Percentage (%)	**Term (Volume)**	**Number of Simulation**									
		1	2	3	4	5	6	7	8	9	10
0 (Base flow)	Quadratic	-0.405	-0.415	-0.397	-0.408	-0.413	-0.412	-0.404	-0.396	-0.403	-0.404
	Linear	78.439	76.548	77.086	76.965	78.748	80.466	80.449	77.650	77.021	80.556
5	Quadratic	-0.397	-0.407	-0.388	-0.399	-0.403	-0.404	-0.396	-0.388	-0.394	-0.396
	Linear	76.533	74.831	75.342	75.248	76.992	78.479	78.369	75.894	75.189	78.675
10	Quadratic	-0.390	-0.400	-0.382	-0.391	-0.396	-0.397	-0.388	-0.381	-0.387	-0.389
	Linear	74.259	72.336	73.254	73.135	74.760	76.336	75.898	73.773	72.965	76.559
15	Quadratic	-0.381	-0.392	-0.372	-0.383	-0.388	-0.388	-0.379	-0.372	-0.379	-0.380
	Linear	71.787	69.828	70.885	70.840	72.184	73.765	73.208	71.441	70.714	74.307
20	Quadratic	-0.371	-0.383	-0.363	-0.373	-0.378	-0.379	-0.370	-0.363	-0.370	-0.370
	Linear	69.742	67.755	68.987	68.728	70.055	71.815	71.170	69.309	68.832	72.415
25	Quadratic	-0.363	-0.374	-0.354	-0.365	-0.370	-0.371	-0.361	-0.355	-0.362	-0.362

	Linear	67.618	65.713	66.971	66.485	68.010	69.812	68.932	67.147	66.722	70.096
30	Quadratic	-0.358	-0.369	-0.349	-0.361	-0.365	-0.367	-0.357	-0.351	-0.357	-0.357
	Linear	65.85	63.994	65.308	64.662	66.209	67.942	67.328	65.324	64.922	68.422
35	Quadratic	-0.355	-0.366	-0.346	-0.357	-0.361	-0.363	-0.353	-0.347	-0.353	-0.354
	Linear	64.158	62.256	63.533	62.984	64.384	66.348	65.523	63.648	63.399	66.875
40	Quadratic	-0.347	-0.358	-0.338	-0.350	-0.354	-0.356	-0.346	-0.340	-0.346	-0.346
	Linear	62.143	60.299	61.347	61.028	62.532	64.143	63.338	61.755	61.216	64.723
45	Quadratic	-0.342	-0.352	-0.333	-0.343	-0.348	-0.351	-0.340	-0.334	-0.339	-0.340
	Linear	60.558	58.799	59.641	59.540	60.970	62.439	61.700	60.118	59.558	63.184
50	Quadratic	-0.334	-0.343	-0.324	-0.335	-0.340	-0.343	-0.332	-0.326	-0.331	-0.332
	Linear	59.024	57.181	57.966	58.109	59.433	60.949	60.202	58.606	57.884	61.576
55	Quadratic	-0.328	-0.337	-0.319	-0.329	-0.334	-0.336	-0.326	-0.320	-0.325	-0.326
	Linear	57.758	55.871	56.596	56.836	58.074	59.741	59.005	57.456	56.626	60.364
60	Quadratic	-0.322	-0.331	-0.313	-0.324	-0.327	-0.330	-0.320	-0.314	-0.319	-0.320
	Linear	56.236	54.253	55.178	55.413	56.435	58.292	57.388	56.050	55.000	58.794
65	Quadratic	-0.316	-0.324	-0.307	-0.318	-0.322	-0.324	-0.314	-0.309	-0.313	-0.313
	Linear	54.878	52.772	53.896	53.976	55.065	56.894	56.030	54.791	53.763	57.551
70	Quadratic	-0.308	-0.316	-0.300	-0.311	-0.314	-0.317	-0.307	-0.302	-0.305	-0.305
	Linear	53.582	51.404	52.624	52.593	53.829	55.641	54.606	53.606	52.525	56.371
75	Quadratic	-0.304	-0.311	-0.295	-0.307	-0.310	-0.312	-0.303	-0.298	-0.300	-0.300
	Linear	52.478	50.344	51.625	51.409	52.659	54.500	53.534	52.398	51.320	55.304
80	Quadratic	-0.301	-0.308	-0.292	-0.304	-0.307	-0.309	-0.300	-0.295	-0.297	-0.297
	Linear	51.478	49.301	50.611	50.425	51.686	53.542	52.621	51.482	50.317	54.246
85	Quadratic	-0.298	-0.306	-0.289	-0.300	-0.303	-0.306	-0.296	-0.291	-0.295	-0.294
	Linear	50.578	48.335	49.679	49.450	50.822	52.721	51.651	50.669	49.437	53.277
90	Quadratic	-0.294	-0.302	-0.285	-0.297	-0.299	-0.302	-0.293	-0.288	-0.291	-0.290
	Linear	49.52	47.289	48.570	48.416	49.812	51.706	50.647	49.566	48.301	52.179

(d) Developed volume-density curves in ED_PCEs estimation using VISSIM 9.0 with a single free-flow speed distribution (continue)

Truck Percentage (%)	**Term (Volume)**	**Number of Simulation**									
		11	12	13	14	15	16	17	18	19	20
0 (Base flow)	Quadratic	-0.402	-0.397	-0.397	-0.404	-0.409	-0.410	-0.411	-0.408	-0.405	-0.406
	Linear	80.974	76.907	80.490	77.529	79.270	80.326	79.201	78.587	79.259	77.715

5	Quadratic	-0.393	-0.388	-0.387	-0.395	-0.401	-0.401	-0.401	-0.400	-0.395	-0.397
	Linear	79.142	75.176	78.432	75.698	77.494	78.250	77.349	76.557	77.288	75.708
10	Quadratic	-0.386	-0.381	-0.380	-0.388	-0.394	-0.394	-0.394	-0.392	-0.387	-0.390
	Linear	76.893	72.834	76.012	73.483	75.093	76.152	74.962	74.312	74.968	73.247
15	Quadratic	-0.378	-0.372	-0.370	-0.379	-0.386	-0.385	-0.385	-0.384	-0.379	-0.381
	Linear	74.408	70.531	73.696	70.943	72.727	73.909	72.719	71.711	72.514	70.985
20	Quadratic	-0.368	-0.362	-0.362	-0.369	-0.376	-0.376	-0.376	-0.375	-0.370	-0.371
	Linear	72.265	68.472	71.576	68.771	70.620	72.028	70.713	69.716	70.343	68.758
25	Quadratic	-0.359	-0.353	-0.352	-0.360	-0.367	-0.367	-0.367	-0.366	-0.360	-0.362
	Linear	70.296	66.532	69.603	66.826	68.309	69.972	68.476	67.648	68.379	66.815
30	Quadratic	-0.355	-0.348	-0.348	-0.356	-0.363	-0.363	-0.363	-0.361	-0.356	-0.358
	Linear	68.365	64.827	67.815	65.028	66.438	68.260	66.684	65.830	66.613	65.165
35	Quadratic	-0.351	-0.345	-0.345	-0.353	-0.359	-0.360	-0.359	-0.358	-0.352	-0.354
	Linear	66.684	63.077	66.032	63.445	64.638	66.448	64.942	64.065	65.007	63.378
40	Quadratic	-0.344	-0.338	-0.337	-0.346	-0.351	-0.353	-0.352	-0.350	-0.345	-0.348
	Linear	64.813	61.031	64.207	61.416	62.596	64.234	63.060	61.981	63.045	61.455
45	Quadratic	-0.338	-0.332	-0.331	-0.340	-0.346	-0.347	-0.346	-0.344	-0.340	-0.342
	Linear	63.239	59.513	62.588	59.972	60.942	62.663	61.548	60.362	61.569	59.821
50	Quadratic	-0.330	-0.324	-0.323	-0.332	-0.339	-0.339	-0.339	-0.337	-0.331	-0.333
	Linear	61.629	57.907	60.904	58.484	59.256	61.245	60.058	58.685	60.104	58.385
55	Quadratic	-0.325	-0.318	-0.318	-0.327	-0.333	-0.333	-0.333	-0.331	-0.325	-0.327
	Linear	60.247	56.556	59.734	57.272	57.874	60.046	58.760	57.302	58.874	57.080
60	Quadratic	-0.318	-0.313	-0.312	-0.321	-0.328	-0.327	-0.327	-0.326	-0.319	-0.321
	Linear	58.580	55.168	58.305	55.723	56.309	58.583	57.166	55.926	57.378	55.673
65	Quadratic	-0.312	-0.307	-0.306	-0.315	-0.321	-0.320	-0.321	-0.320	-0.313	-0.316
	Linear	57.232	53.929	57.049	54.374	55.068	57.276	55.905	54.620	56.069	54.221
70	Quadratic	-0.305	-0.299	-0.298	-0.307	-0.314	-0.313	-0.314	-0.312	-0.306	-0.309
	Linear	55.839	52.633	55.819	52.958	53.897	55.975	54.579	53.374	54.788	52.963
75	Quadratic	-0.301	-0.295	-0.293	-0.303	-0.309	-0.308	-0.309	-0.307	-0.301	-0.304
	Linear	54.817	51.469	54.627	51.879	52.750	54.793	53.494	52.333	53.776	51.957
80	Quadratic	-0.298	-0.292	-0.290	-0.299	-0.306	-0.306	-0.306	-0.304	-0.298	-0.301
	Linear	53.776	50.430	53.697	50.951	51.719	53.798	52.395	51.297	52.694	50.877
85	Quadratic	-0.295	-0.289	-0.286	-0.297	-0.303	-0.302	-0.303	-0.302	-0.295	-0.298
	Linear	52.866	49.515	52.841	49.997	50.818	52.844	51.434	50.458	51.745	49.892
90	Quadratic	-0.291	-0.285	-0.282	-0.292	-0.300	-0.299	-0.299	-0.297	-0.291	-0.294
	Linear	51.721	48.553	51.848	48.881	49.787	51.721	50.366	49.409	50.793	48.920

(e) Developed volume-density curves in ED_PCEs estimation using VISSIM 9.0 with empirical free-flow speed distributions

Truck Percentage (%)	**Term (Volume)**	**Number of Simulation**									
		1	2	3	4	5	6	7	8	9	10
0 (Base flow)	Quadratic	-0.379	-0.389	-0.376	-0.376	-0.381	-0.387	-0.376	-0.380	-0.388	-0.379
	Linear	90.821	87.053	90.896	87.590	92.652	92.115	88.045	90.005	88.250	87.504
5	Quadratic	-0.368	-0.377	-0.365	-0.366	-0.369	-0.376	-0.364	-0.369	-0.377	-0.369
	Linear	85.519	82.273	85.534	82.729	87.024	86.568	83.115	84.778	83.289	82.656
10	Quadratic	-0.359	-0.368	-0.357	-0.357	-0.360	-0.366	-0.355	-0.361	-0.368	-0.360
	Linear	79.871	77.064	80.130	76.876	81.439	80.689	77.433	78.592	78.037	76.726
15	Quadratic	-0.346	-0.355	-0.343	-0.346	-0.349	-0.354	-0.341	-0.349	-0.354	-0.347
	Linear	74.925	72.154	74.756	72.171	76.197	76.227	72.748	73.799	73.538	72.092
20	Quadratic	-0.340	-0.349	-0.336	-0.340	-0.342	-0.347	-0.335	-0.343	-0.348	-0.341
	Linear	70.489	67.888	70.714	67.364	71.994	71.871	68.380	69.362	69.295	67.829
25	Quadratic	-0.323	-0.332	-0.320	-0.322	-0.325	-0.329	-0.318	-0.326	-0.330	-0.326
	Linear	66.464	63.942	66.911	62.938	67.668	67.949	64.046	65.184	65.129	63.613
30	Quadratic	-0.315	-0.324	-0.312	-0.314	-0.318	-0.321	-0.310	-0.318	-0.322	-0.318
	Linear	62.734	60.245	62.991	59.334	63.836	64.030	60.402	61.214	61.646	59.758
35	Quadratic	-0.304	-0.313	-0.302	-0.303	-0.307	-0.310	-0.300	-0.307	-0.312	-0.306
	Linear	59.451	57.156	59.415	56.224	60.506	60.938	57.383	58.027	58.446	56.288
40	Quadratic	-0.298	-0.308	-0.296	-0.298	-0.301	-0.304	-0.294	-0.301	-0.307	-0.301
	Linear	56.589	54.430	56.382	53.490	57.794	57.969	54.656	55.203	55.492	53.147
45	Quadratic	-0.291	-0.300	-0.289	-0.290	-0.294	-0.296	-0.287	-0.294	-0.299	-0.294
	Linear	54.012	51.693	53.786	50.746	54.975	55.240	52.233	52.451	52.975	50.418
50	Quadratic	-0.286	-0.296	-0.284	-0.285	-0.289	-0.292	-0.283	-0.289	-0.294	-0.290
	Linear	51.613	49.359	51.288	48.328	52.554	52.963	49.703	50.050	50.681	47.889
55	Quadratic	-0.280	-0.290	-0.278	-0.279	-0.283	-0.286	-0.277	-0.284	-0.288	-0.283
	Linear	49.529	47.324	49.166	46.389	50.643	50.897	47.713	48.110	48.551	45.941
60	Quadratic	-0.273	-0.283	-0.271	-0.272	-0.276	-0.278	-0.270	-0.277	-0.281	-0.276
	Linear	47.502	45.278	47.096	44.347	48.660	48.893	45.779	45.927	46.677	43.892
65	Quadratic	-0.271	-0.280	-0.268	-0.270	-0.273	-0.276	-0.268	-0.274	-0.279	-0.273
	Linear	45.914	43.532	45.437	42.659	46.919	47.186	44.241	44.183	44.958	42.451
70	Quadratic	-0.266	-0.276	-0.264	-0.266	-0.269	-0.271	-0.263	-0.270	-0.274	-0.269

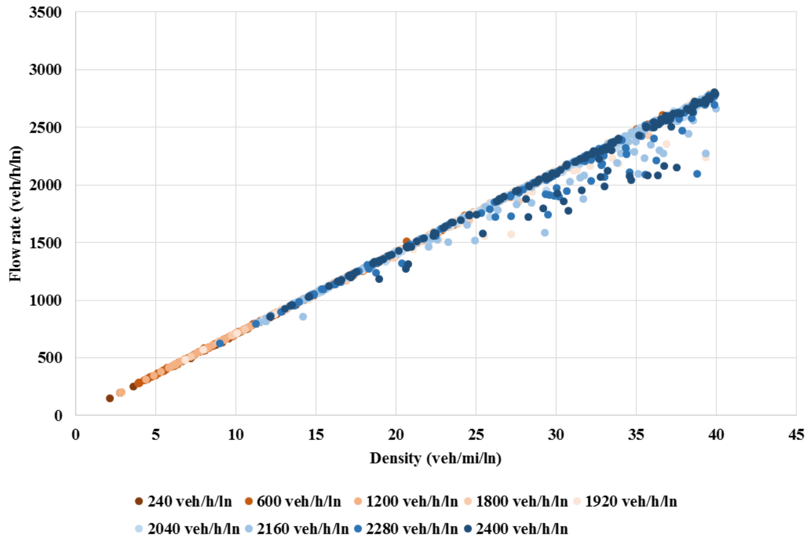
	Linear	44.250	41.996	43.794	40.987	45.183	45.531	42.564	42.407	43.288	40.928
75	Quadratic	-0.261	-0.270	-0.258	-0.260	-0.263	-0.266	-0.258	-0.264	-0.269	-0.264
	Linear	42.874	40.651	42.390	39.582	43.782	44.208	41.134	40.965	41.781	39.510
80	Quadratic	-0.258	-0.268	-0.255	-0.258	-0.261	-0.264	-0.256	-0.262	-0.267	-0.262
	Linear	41.744	39.610	41.192	38.389	42.574	43.047	39.928	39.891	40.730	38.281
85	Quadratic	-0.257	-0.266	-0.254	-0.256	-0.259	-0.262	-0.254	-0.260	-0.265	-0.260
	Linear	40.560	38.440	39.898	37.306	41.465	41.841	38.741	38.656	39.531	37.197
90	Quadratic	-0.254	-0.264	-0.251	-0.254	-0.257	-0.260	-0.251	-0.258	-0.262	-0.258
	Linear	39.385	37.280	38.683	36.029	40.357	40.612	37.469	37.521	38.413	36.083

(f) Developed volume-density curves in ED_PCEs estimation using VISSIM 9.0 with empirical free-flow speed distributions (continue)

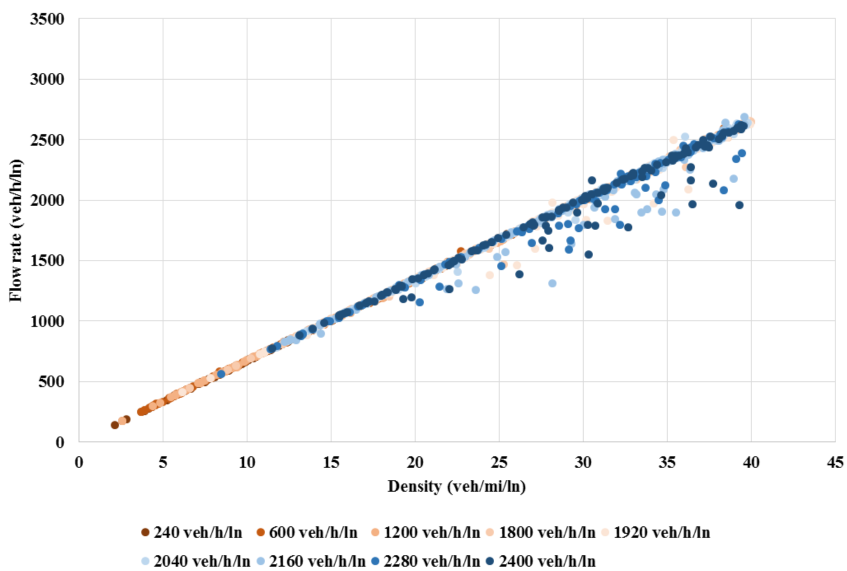
Truck Percentage (%)	**Term (Volume)**	**Number of Simulation**									
		11	12	13	14	15	16	17	18	19	20
0 (Base flow)	Quadratic	-0.375	-0.369	-0.371	-0.374	-0.384	-0.380	-0.374	-0.383	-0.378	-0.378
	Linear	92.171	93.197	93.615	93.653	92.814	91.537	93.149	90.292	89.458	92.061
5	Quadratic	-0.363	-0.357	-0.360	-0.361	-0.373	-0.369	-0.361	-0.371	-0.367	-0.367
	Linear	86.615	87.486	87.841	87.874	87.161	86.077	87.446	85.021	84.314	86.522
10	Quadratic	-0.354	-0.349	-0.351	-0.353	-0.365	-0.360	-0.352	-0.362	-0.358	-0.357
	Linear	81.398	81.927	82.727	81.723	81.334	80.334	81.588	79.915	78.330	80.848
15	Quadratic	-0.341	-0.336	-0.339	-0.341	-0.352	-0.348	-0.338	-0.348	-0.346	-0.344
	Linear	76.420	77.236	77.987	76.504	76.480	75.084	76.543	75.351	73.131	76.174
20	Quadratic	-0.336	-0.330	-0.333	-0.334	-0.346	-0.342	-0.333	-0.342	-0.340	-0.338
	Linear	72.073	72.949	73.746	72.029	71.681	70.724	72.219	70.716	68.660	71.905
25	Quadratic	-0.318	-0.312	-0.317	-0.318	-0.331	-0.325	-0.317	-0.325	-0.324	-0.322
	Linear	68.200	69.025	70.032	67.820	67.254	66.695	68.143	66.642	64.350	68.093
30	Quadratic	-0.309	-0.304	-0.308	-0.310	-0.322	-0.317	-0.309	-0.316	-0.316	-0.314
	Linear	64.448	65.333	65.988	64.412	63.387	62.648	64.295	62.942	60.661	64.266
35	Quadratic	-0.299	-0.292	-0.298	-0.299	-0.311	-0.307	-0.299	-0.305	-0.306	-0.303
	Linear	61.427	62.309	62.405	61.386	60.345	59.456	61.303	59.681	57.492	61.158
40	Quadratic	-0.293	-0.286	-0.291	-0.293	-0.305	-0.302	-0.293	-0.299	-0.300	-0.297
	Linear	58.347	59.516	59.573	58.344	57.358	56.719	58.512	56.865	54.487	58.269
45	Quadratic	-0.286	-0.279	-0.285	-0.285	-0.298	-0.295	-0.286	-0.292	-0.292	-0.289
	Linear	55.690	56.881	57.186	55.588	55.020	54.137	56.014	54.332	51.686	55.777

50	Quadratic	-0.281	-0.274	-0.280	-0.281	-0.293	-0.291	-0.282	-0.287	-0.288	-0.285
	Linear	53.467	54.638	54.889	53.026	52.753	51.798	53.750	51.863	49.308	53.450
55	Quadratic	-0.275	-0.268	-0.273	-0.275	-0.287	-0.284	-0.276	-0.281	-0.281	-0.280
	Linear	51.373	52.602	52.818	50.920	50.523	49.880	51.463	49.701	47.345	51.481
60	Quadratic	-0.268	-0.261	-0.266	-0.267	-0.280	-0.277	-0.269	-0.274	-0.274	-0.272
	Linear	49.461	50.520	50.660	48.912	48.352	47.782	49.258	47.843	45.504	49.295
65	Quadratic	-0.266	-0.259	-0.264	-0.265	-0.277	-0.275	-0.267	-0.271	-0.272	-0.270
	Linear	48.028	48.935	48.927	47.480	46.743	46.336	47.597	46.116	44.045	47.769
70	Quadratic	-0.261	-0.254	-0.260	-0.261	-0.273	-0.270	-0.262	-0.266	-0.268	-0.266
	Linear	46.509	47.382	47.413	45.933	45.208	44.705	45.893	44.478	42.536	46.138
75	Quadratic	-0.255	-0.248	-0.254	-0.256	-0.268	-0.265	-0.257	-0.261	-0.262	-0.260
	Linear	45.121	46.014	46.091	44.654	43.829	43.328	44.514	43.160	41.098	44.860
80	Quadratic	-0.252	-0.246	-0.252	-0.253	-0.265	-0.263	-0.255	-0.259	-0.259	-0.258
	Linear	43.896	44.978	44.985	43.578	42.784	42.309	43.315	41.925	39.937	43.804
85	Quadratic	-0.250	-0.244	-0.250	-0.252	-0.264	-0.261	-0.253	-0.257	-0.258	-0.256
	Linear	42.705	43.878	43.684	42.488	41.682	41.177	42.182	40.859	38.735	42.640
90	Quadratic	-0.248	-0.241	-0.247	-0.249	-0.261	-0.259	-0.251	-0.255	-0.255	-0.253
	Linear	41.518	42.663	42.593	41.275	40.499	40.000	40.970	39.587	37.487	41.478

APPENDIX G EXAMPLES OF SCATTER PLOTS IN EC_PCE ESTIMATION

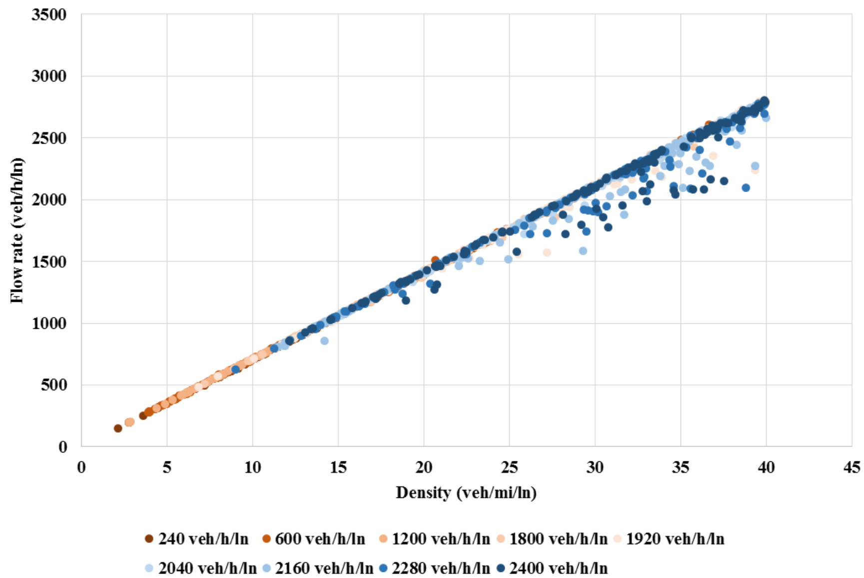


(a1) Auto-only flow at 1% grade, 1.5 miles distance

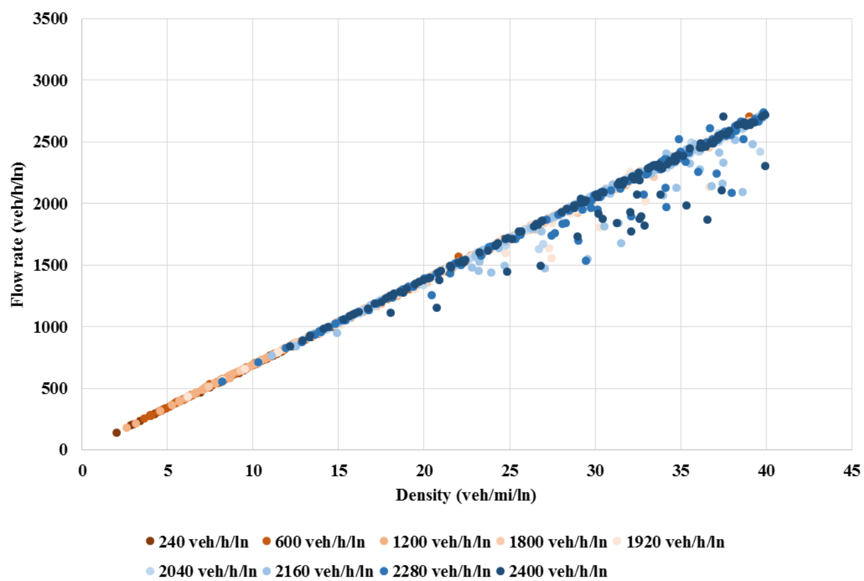


(a2) Auto-only flow at 2% grade, 1.5 miles distance

(a) Example of flow-density scatter plots at different grade for the auto-only flow under HCM 2016 research conditions

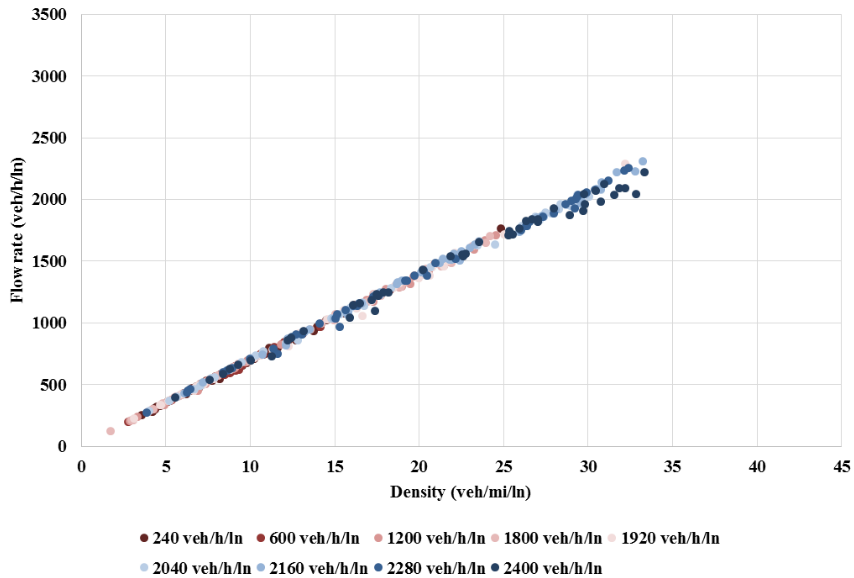


(b1) Auto-only flow at 1% grade, 1.5 miles distance

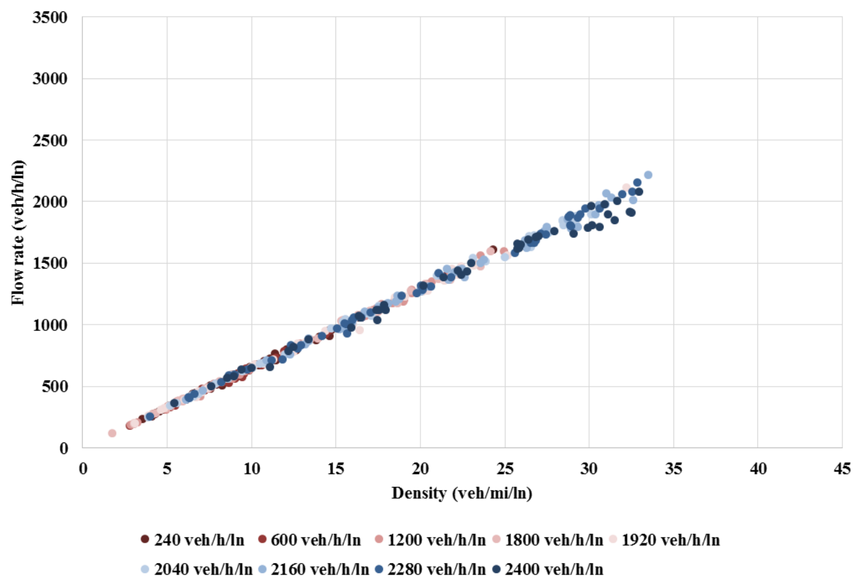


(b2) Auto-only flow at 1% grade, 2.5 miles distance

(b) Example of flow-density scatter plots at different distance for the auto-only flow under HCM 2016 research conditions

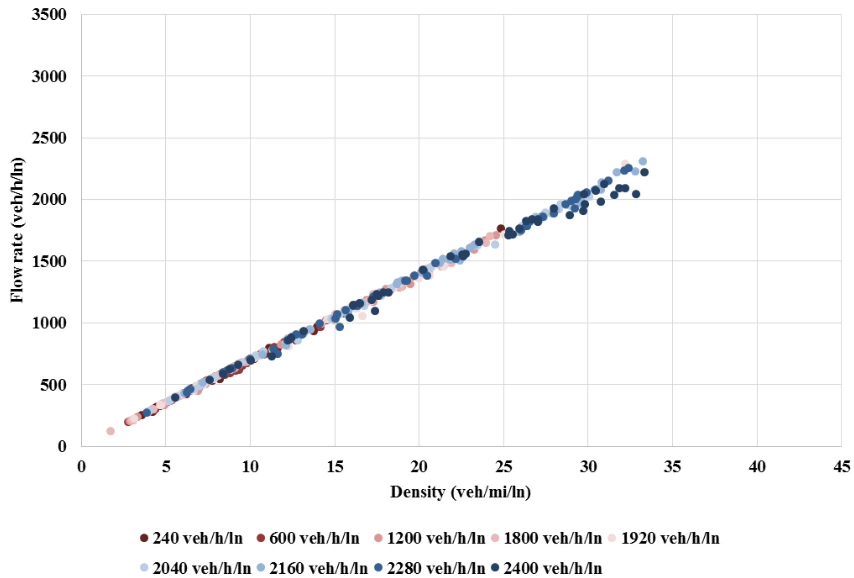


(c1) Mixed flow at 10% truck, 30% SUT/70% TT, 1% grade, 1.5 miles distance

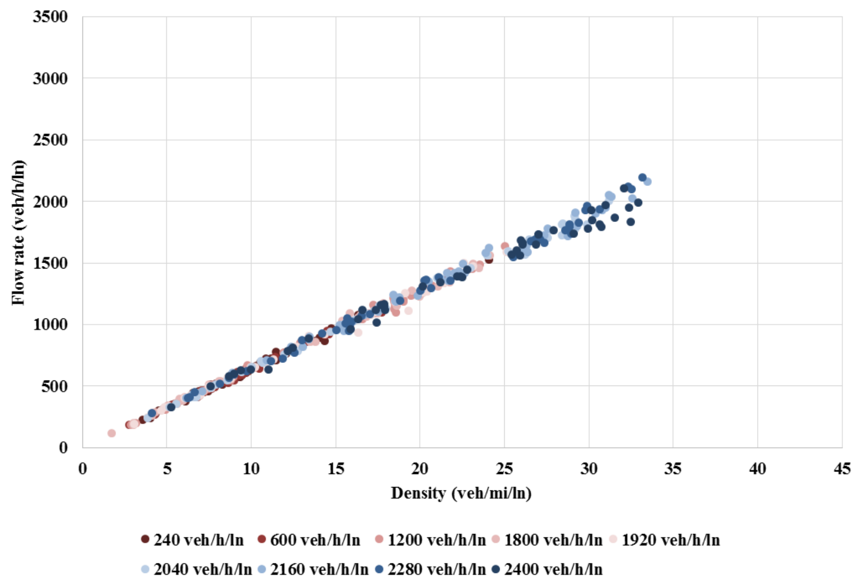


(c2) Mixed flow at 15% truck, 30% SUT/70% TT, 1% grade, 1.5 miles distance

(c) Example of flow-density scatter plots at different truck percentage for the mixed flow under HCM 2016 research conditions

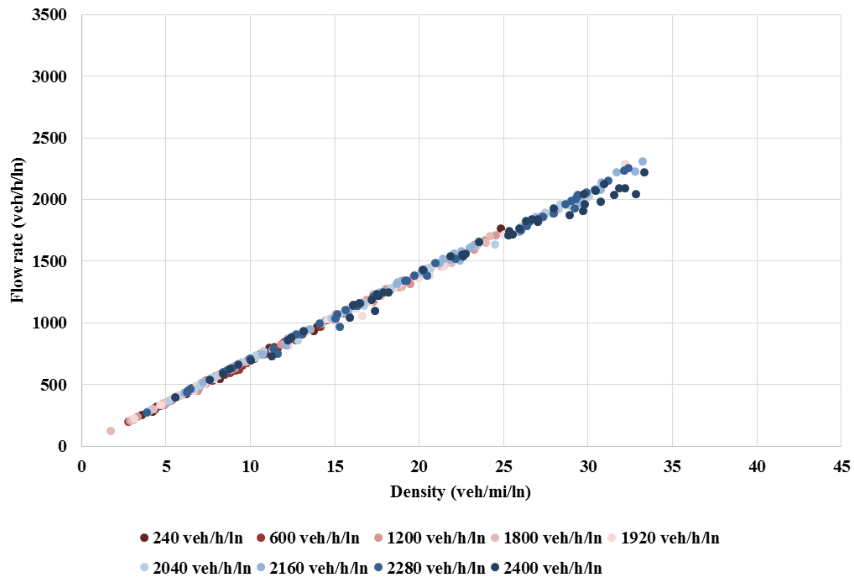


(d1) Mixed flow at 10% truck, 30% SUT/70% TT, 1% grade, 1.5 miles distance

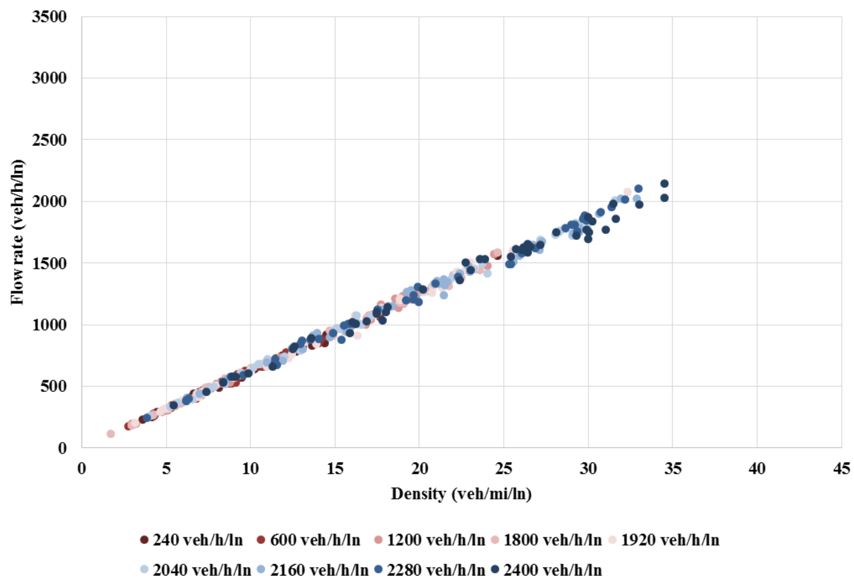


(d2) Mixed flow at 10% truck, 50% SUT/50% TT, 1% grade, 1.5 miles distance

(d) Example of flow-density scatter plots at different truck composition for the mixed flow under HCM 2016 research conditions

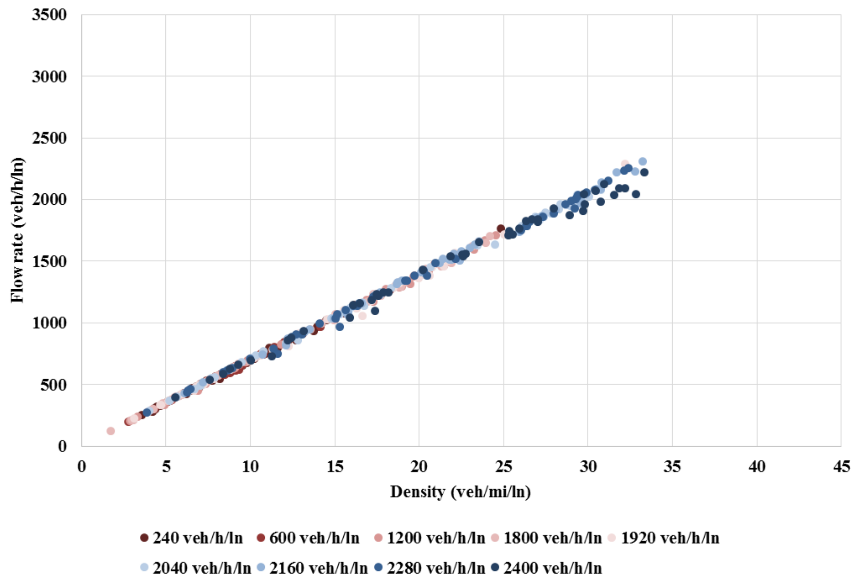


(e1) Mixed flow at 10% truck, 30% SUT/70% TT, 1% grade, 1.5 miles distance

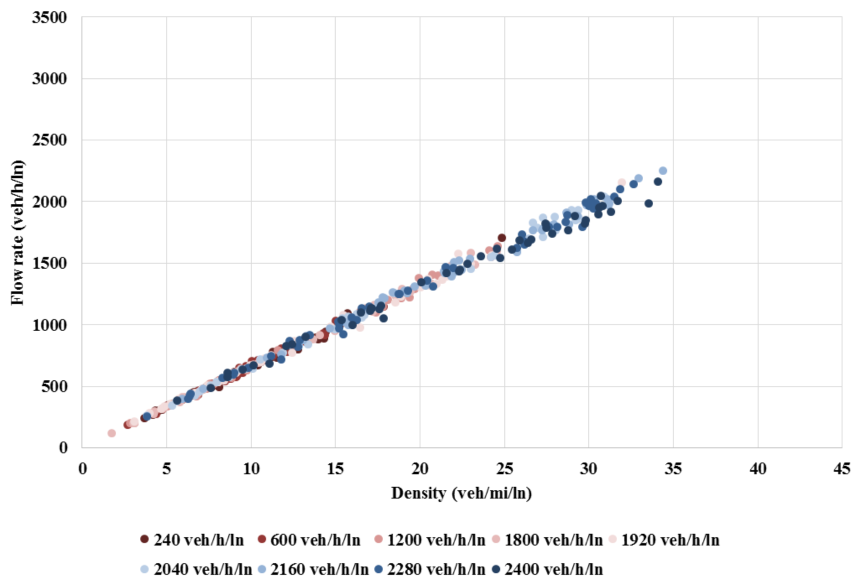


(e2) Mixed flow at 10% truck, 30% SUT/70% TT, 2% grade, 1.5 miles distance

(e) Example of flow-density scatter plots at different grade for the mixed flow under HCM 2016 research conditions



(f1) Mixed flow at 10% truck, 30% SUT/70% TT, 1% grade, 1.5 miles distance



(f2) Mixed flow at 10% truck, 30% SUT/70% TT, 1% grade, 2.5 miles distance

(f) Example of flow-density scatter plots at different distance for the mixed flow under HCM 2016 research conditions

APPENDIX H EC_PCE AS FUNCTION OF SPEED LIMIT LEVEL FOR ALL LENGTH

(a) EC_PCE as a function of speed limit level for 0.125 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.31	2.77	2.68	2.68	2.74	2.85	3.00	3.20	3.49
	50% SUT, 50% TT	2.72	2.30	2.25	2.27	2.33	2.42	2.54	2.71	2.93
	70% SUT, 30% TT	2.50	2.16	2.12	2.15	2.20	2.29	2.40	2.54	2.73
70 mph, 0%, 1 min	30% SUT, 70% TT	2.64	2.39	2.36	2.38	2.44	2.53	2.65	2.81	3.03
	50% SUT, 50% TT	2.34	2.17	2.15	2.18	2.23	2.30	2.39	2.52	2.68
	70% SUT, 30% TT	2.21	2.05	2.02	2.03	2.06	2.11	2.17	2.25	2.35
80 mph, 0%, 1 min	30% SUT, 70% TT	4.14	3.10	2.91	2.88	2.92	3.03	3.19	3.43	3.75
	50% SUT, 50% TT	3.82	2.89	2.73	2.72	2.79	2.90	3.08	3.33	3.68
	70% SUT, 30% TT	3.72	2.79	2.63	2.61	2.65	2.74	2.87	3.05	3.31
85 mph, 0%, 1 min	30% SUT, 70% TT	4.73	3.40	3.15	3.11	3.16	3.29	3.49	3.78	4.21
	50% SUT, 50% TT	4.69	3.30	3.06	3.03	3.10	3.24	3.47	3.80	4.30
	70% SUT, 30% TT	4.63	3.21	2.96	2.91	2.95	3.06	3.24	3.49	3.86

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(b) EC_PCE as a function of speed limit level for 0.375 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.52	2.86	2.74	2.74	2.80	2.90	3.06	3.27	3.57
	50% SUT, 50% TT	2.86	2.35	2.28	2.30	2.36	2.45	2.57	2.74	2.96
	70% SUT, 30% TT	2.62	2.21	2.16	2.18	2.23	2.31	2.42	2.57	2.77

70 mph, 0%, 1 min	30% SUT, 70% TT	2.83	2.47	2.41	2.43	2.49	2.58	2.70	2.87	3.10
	50% SUT, 50% TT	2.44	2.21	2.18	2.20	2.25	2.32	2.41	2.54	2.70
	70% SUT, 30% TT	2.34	2.10	2.06	2.06	2.09	2.13	2.20	2.28	2.38
80 mph, 0%, 1 min	30% SUT, 70% TT	4.31	3.17	2.96	2.92	2.97	3.08	3.25	3.49	3.83
	50% SUT, 50% TT	3.94	2.93	2.77	2.75	2.81	2.93	3.11	3.36	3.73
	70% SUT, 30% TT	3.88	2.86	2.68	2.65	2.69	2.77	2.91	3.10	3.36
85 mph, 0%, 1 min	30% SUT, 70% TT	4.89	3.46	3.20	3.15	3.21	3.34	3.54	3.85	4.29
	50% SUT, 50% TT	4.80	3.35	3.09	3.06	3.13	3.27	3.50	3.84	4.35
	70% SUT, 30% TT	4.81	3.28	3.01	2.95	2.99	3.10	3.28	3.54	3.93

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(c) EC_PCE as a function of speed limit level for 0.625 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.70	2.94	2.80	2.79	2.84	2.95	3.10	3.32	3.63
	50% SUT, 50% TT	2.92	2.38	2.30	2.31	2.37	2.46	2.58	2.75	2.98
	70% SUT, 30% TT	2.69	2.23	2.18	2.19	2.24	2.33	2.44	2.59	2.79
70 mph, 0%, 1 min	30% SUT, 70% TT	2.96	2.52	2.45	2.46	2.52	2.61	2.74	2.91	3.14
	50% SUT, 50% TT	2.51	2.24	2.20	2.21	2.26	2.33	2.43	2.55	2.71
	70% SUT, 30% TT	2.41	2.13	2.07	2.07	2.10	2.15	2.21	2.29	2.39
80 mph, 0%, 1 min	30% SUT, 70% TT	4.42	3.22	2.99	2.95	3.00	3.11	3.28	3.53	3.88
	50% SUT, 50% TT	4.01	2.96	2.79	2.77	2.83	2.95	3.13	3.38	3.75
	70% SUT, 30% TT	3.97	2.90	2.71	2.67	2.71	2.80	2.93	3.12	3.39
85 mph, 0%, 1 min	30% SUT, 70% TT	4.98	3.51	3.23	3.18	3.24	3.37	3.58	3.89	4.34
	50% SUT, 50% TT	4.87	3.38	3.12	3.08	3.14	3.29	3.52	3.86	4.38
	70% SUT, 30% TT	4.90	3.32	3.04	2.98	3.02	3.13	3.31	3.57	3.96

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(d) EC_PCE as a function of speed limit level for 0.875 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.79	2.97	2.82	2.81	2.86	2.97	3.13	3.35	3.66
	50% SUT, 50% TT	2.95	2.39	2.31	2.32	2.37	2.46	2.59	2.76	2.99
	70% SUT, 30% TT	2.72	2.25	2.18	2.20	2.25	2.33	2.45	2.60	2.80
70 mph, 0%, 1 min	30% SUT, 70% TT	3.03	2.55	2.47	2.48	2.53	2.63	2.75	2.93	3.16
	50% SUT, 50% TT	2.55	2.25	2.21	2.22	2.27	2.34	2.43	2.56	2.72
	70% SUT, 30% TT	2.45	2.14	2.08	2.08	2.11	2.15	2.21	2.29	2.40
80 mph, 0%, 1 min	30% SUT, 70% TT	4.47	3.24	3.01	2.97	3.02	3.13	3.30	3.55	3.90
	50% SUT, 50% TT	4.04	2.98	2.80	2.78	2.84	2.96	3.14	3.40	3.76
	70% SUT, 30% TT	4.02	2.92	2.72	2.68	2.72	2.81	2.94	3.14	3.41
85 mph, 0%, 1 min	30% SUT, 70% TT	5.03	3.53	3.25	3.20	3.25	3.38	3.59	3.91	4.37
	50% SUT, 50% TT	4.91	3.39	3.13	3.09	3.15	3.30	3.53	3.87	4.39
	70% SUT, 30% TT	4.95	3.34	3.05	2.99	3.03	3.14	3.32	3.59	3.98

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(e) EC_PCE as a function of speed limit level for 1.25 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.85	3.00	2.84	2.82	2.88	2.99	3.15	3.37	3.68
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.75	2.26	2.19	2.20	2.26	2.34	2.45	2.60	2.80
70 mph, 0%, 1 min	30% SUT, 70% TT	3.07	2.57	2.48	2.49	2.55	2.64	2.77	2.94	3.18
	50% SUT, 50% TT	2.57	2.26	2.21	2.23	2.27	2.34	2.44	2.56	2.73
	70% SUT, 30% TT	2.48	2.15	2.09	2.09	2.11	2.16	2.22	2.30	2.40
	30% SUT, 70% TT	4.51	3.26	3.02	2.98	3.03	3.14	3.31	3.56	3.92

80 mph, 0%, 1 min	50% SUT, 50% TT	4.07	2.99	2.81	2.79	2.85	2.97	3.15	3.40	3.77
	70% SUT, 30% TT	4.06	2.93	2.73	2.69	2.73	2.82	2.95	3.15	3.42
85 mph, 0%, 1 min	30% SUT, 70% TT	5.07	3.54	3.26	3.21	3.26	3.39	3.61	3.92	4.39
	50% SUT, 50% TT	4.93	3.40	3.13	3.09	3.16	3.31	3.54	3.88	4.40
	70% SUT, 30% TT	4.99	3.36	3.06	3.00	3.04	3.15	3.33	3.60	3.99

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(f) EC_PCE as a function of speed limit level for 1.5 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.87	3.00	2.85	2.83	2.88	2.99	3.15	3.37	3.69
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.76	2.26	2.19	2.21	2.26	2.34	2.45	2.60	2.80
70 mph, 0%, 1 min	30% SUT, 70% TT	3.08	2.57	2.49	2.50	2.55	2.64	2.77	2.95	3.18
	50% SUT, 50% TT	2.58	2.26	2.22	2.23	2.27	2.35	2.44	2.57	2.73
	70% SUT, 30% TT	2.48	2.15	2.09	2.09	2.11	2.16	2.22	2.30	2.40
80 mph, 0%, 1 min	30% SUT, 70% TT	4.52	3.26	3.03	2.98	3.03	3.14	3.31	3.56	3.92
	50% SUT, 50% TT	4.08	2.99	2.81	2.79	2.85	2.97	3.15	3.41	3.78
	70% SUT, 30% TT	4.07	2.94	2.73	2.70	2.73	2.82	2.96	3.15	3.42
85 mph, 0%, 1 min	30% SUT, 70% TT	5.08	3.55	3.26	3.21	3.26	3.40	3.61	3.93	4.39
	50% SUT, 50% TT	4.94	3.40	3.14	3.10	3.16	3.31	3.54	3.89	4.41
	70% SUT, 30% TT	5.00	3.36	3.07	3.00	3.04	3.15	3.33	3.60	4.00

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

APPENDIX I EC_PCE AS FUNCTION OF TRUCK RESTRITION LEVEL FOR ALL LENGTH

(a) EC_PCE as a function of truck passing restriction level for 0.125 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.31	2.77	2.68	2.68	2.74	2.85	3.00	3.20	3.49
	50% SUT, 50% TT	2.72	2.30	2.25	2.27	2.33	2.42	2.54	2.71	2.93
	70% SUT, 30% TT	2.50	2.16	2.12	2.15	2.20	2.29	2.40	2.54	2.73
75 mph, 25%, 1 min	30% SUT, 70% TT	2.95	2.45	2.36	2.34	2.37	2.43	2.51	2.62	2.76
	50% SUT, 50% TT	2.57	2.17	2.10	2.10	2.13	2.18	2.26	2.35	2.48
	70% SUT, 30% TT	2.52	2.04	1.97	1.97	2.00	2.05	2.12	2.21	2.32
75 mph, 50%, 1 min	30% SUT, 70% TT	2.52	2.12	2.06	2.06	2.09	2.15	2.23	2.33	2.45
	50% SUT, 50% TT	2.44	2.00	1.94	1.94	1.97	2.02	2.09	2.17	2.29
	70% SUT, 30% TT	2.30	1.83	1.76	1.75	1.76	1.79	1.83	1.88	1.94
75 mph, 75%, 1 min	30% SUT, 70% TT	2.42	2.03	1.96	1.96	1.98	2.02	2.07	2.15	2.23
	50% SUT, 50% TT	2.27	1.86	1.79	1.79	1.80	1.83	1.88	1.93	2.00
	70% SUT, 30% TT	2.19	1.76	1.69	1.66	1.66	1.67	1.69	1.72	1.75
75 mph, 100%, 1 min	30% SUT, 70% TT	2.35	1.96	1.89	1.88	1.89	1.92	1.96	2.01	2.08
	50% SUT, 50% TT	2.30	1.85	1.76	1.74	1.73	1.74	1.76	1.79	1.82
	70% SUT, 30% TT	2.15	1.68	1.60	1.57	1.57	1.57	1.59	1.61	1.63

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(b) EC_PCE as a function of truck passing restriction level for 0.375 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83

75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.52	2.86	2.74	2.74	2.80	2.90	3.06	3.27	3.57
	50% SUT, 50% TT	2.86	2.35	2.28	2.30	2.36	2.45	2.57	2.74	2.96
	70% SUT, 30% TT	2.62	2.21	2.16	2.18	2.23	2.31	2.42	2.57	2.77
75 mph, 25%, 1 min	30% SUT, 70% TT	3.13	2.53	2.41	2.39	2.41	2.47	2.55	2.66	2.81
	50% SUT, 50% TT	2.70	2.21	2.13	2.13	2.16	2.21	2.28	2.38	2.50
	70% SUT, 30% TT	2.67	2.09	2.00	2.00	2.02	2.07	2.14	2.23	2.35
75 mph, 50%, 1 min	30% SUT, 70% TT	2.69	2.18	2.10	2.10	2.13	2.19	2.26	2.36	2.50
	50% SUT, 50% TT	2.47	2.01	1.95	1.95	1.97	2.02	2.09	2.18	2.29
	70% SUT, 30% TT	2.38	1.86	1.78	1.77	1.78	1.80	1.84	1.89	1.95
75 mph, 75%, 1 min	30% SUT, 70% TT	2.64	2.11	2.02	2.01	2.02	2.06	2.12	2.19	2.28
	50% SUT, 50% TT	2.31	1.87	1.80	1.79	1.81	1.84	1.88	1.93	2.00
	70% SUT, 30% TT	2.33	1.81	1.72	1.69	1.68	1.69	1.71	1.73	1.77
75 mph, 100%, 1 min	30% SUT, 70% TT	2.54	2.03	1.94	1.92	1.92	1.95	1.99	2.04	2.11
	50% SUT, 50% TT	2.30	1.85	1.76	1.74	1.73	1.74	1.76	1.79	1.82
	70% SUT, 30% TT	2.28	1.73	1.63	1.60	1.59	1.59	1.60	1.62	1.64

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(c) EC_PCE as a function of truck passing restriction level for 0.625 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.70	2.94	2.80	2.79	2.84	2.95	3.10	3.32	3.63
	50% SUT, 50% TT	2.92	2.38	2.30	2.31	2.37	2.46	2.58	2.75	2.98
	70% SUT, 30% TT	2.69	2.23	2.18	2.19	2.24	2.33	2.44	2.59	2.79
75 mph, 25%, 1 min	30% SUT, 70% TT	3.24	2.57	2.44	2.42	2.44	2.49	2.58	2.69	2.84
	50% SUT, 50% TT	2.77	2.24	2.15	2.14	2.17	2.22	2.29	2.39	2.51
	70% SUT, 30% TT	2.75	2.12	2.02	2.01	2.04	2.09	2.16	2.25	2.36
75 mph, 50%, 1 min	30% SUT, 70% TT	2.80	2.23	2.13	2.12	2.15	2.21	2.29	2.39	2.52
	50% SUT, 50% TT	2.49	2.02	1.95	1.95	1.98	2.03	2.09	2.18	2.29
	70% SUT, 30% TT	2.42	1.88	1.79	1.77	1.78	1.81	1.85	1.90	1.96
	30% SUT, 70% TT	2.78	2.17	2.06	2.04	2.05	2.09	2.14	2.21	2.31

75 mph, 75%, 1 min	50% SUT, 50% TT	2.33	1.88	1.81	1.80	1.81	1.84	1.88	1.94	2.00
	70% SUT, 30% TT	2.41	1.84	1.73	1.70	1.69	1.70	1.72	1.74	1.77
75 mph, 100%, 1 min	30% SUT, 70% TT	2.66	2.07	1.97	1.94	1.95	1.97	2.01	2.06	2.13
	50% SUT, 50% TT	2.31	1.85	1.76	1.74	1.73	1.74	1.76	1.79	1.82
	70% SUT, 30% TT	2.35	1.75	1.64	1.61	1.60	1.60	1.61	1.63	1.65

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(d) EC_PCE as a function of truck passing restriction level for 0.875 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.79	2.97	2.82	2.81	2.86	2.97	3.13	3.35	3.66
	50% SUT, 50% TT	2.95	2.39	2.31	2.32	2.37	2.46	2.59	2.76	2.99
	70% SUT, 30% TT	2.72	2.25	2.18	2.20	2.25	2.33	2.45	2.60	2.80
75 mph, 25%, 1 min	30% SUT, 70% TT	3.30	2.60	2.46	2.43	2.45	2.51	2.59	2.70	2.86
	50% SUT, 50% TT	2.81	2.26	2.16	2.15	2.18	2.23	2.30	2.40	2.52
	70% SUT, 30% TT	2.79	2.14	2.03	2.02	2.05	2.09	2.16	2.25	2.37
75 mph, 50%, 1 min	30% SUT, 70% TT	2.86	2.25	2.15	2.14	2.17	2.22	2.30	2.40	2.54
	50% SUT, 50% TT	2.49	2.02	1.95	1.95	1.98	2.03	2.09	2.18	2.29
	70% SUT, 30% TT	2.44	1.89	1.80	1.78	1.79	1.81	1.85	1.90	1.96
75 mph, 75%, 1 min	30% SUT, 70% TT	2.85	2.19	2.08	2.05	2.06	2.10	2.15	2.23	2.32
	50% SUT, 50% TT	2.34	1.88	1.81	1.80	1.81	1.84	1.89	1.94	2.00
	70% SUT, 30% TT	2.45	1.85	1.74	1.71	1.70	1.71	1.72	1.75	1.78
75 mph, 100%, 1 min	30% SUT, 70% TT	2.72	2.10	1.98	1.95	1.96	1.98	2.02	2.07	2.14
	50% SUT, 50% TT	2.31	1.85	1.76	1.74	1.73	1.74	1.76	1.79	1.82
	70% SUT, 30% TT	2.39	1.77	1.65	1.61	1.60	1.60	1.61	1.63	1.65

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(e) EC_PCE as a function of truck passing restriction level for 1.25 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.85	3.00	2.84	2.82	2.88	2.99	3.15	3.37	3.68
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.75	2.26	2.19	2.20	2.26	2.34	2.45	2.60	2.80
75 mph, 25%, 1 min	30% SUT, 70% TT	3.34	2.61	2.47	2.44	2.46	2.51	2.60	2.71	2.87
	50% SUT, 50% TT	2.83	2.27	2.17	2.16	2.18	2.23	2.30	2.40	2.53
	70% SUT, 30% TT	2.82	2.15	2.04	2.03	2.05	2.10	2.17	2.26	2.38
75 mph, 50%, 1 min	30% SUT, 70% TT	2.90	2.26	2.16	2.15	2.17	2.23	2.31	2.41	2.55
	50% SUT, 50% TT	2.50	2.02	1.95	1.95	1.98	2.03	2.10	2.18	2.29
	70% SUT, 30% TT	2.46	1.89	1.80	1.78	1.79	1.82	1.85	1.90	1.97
75 mph, 75%, 1 min	30% SUT, 70% TT	2.90	2.21	2.09	2.06	2.07	2.11	2.16	2.24	2.33
	50% SUT, 50% TT	2.35	1.89	1.81	1.80	1.82	1.84	1.89	1.94	2.01
	70% SUT, 30% TT	2.48	1.86	1.75	1.71	1.70	1.71	1.73	1.75	1.78
75 mph, 100%, 1 min	30% SUT, 70% TT	2.76	2.11	1.99	1.96	1.97	1.99	2.03	2.08	2.15
	50% SUT, 50% TT	2.31	1.85	1.76	1.74	1.73	1.74	1.76	1.79	1.82
	70% SUT, 30% TT	2.42	1.78	1.66	1.62	1.60	1.61	1.62	1.63	1.65

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(f) EC_PCE as a function of truck passing restriction level for 1.5 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.87	3.00	2.85	2.83	2.88	2.99	3.15	3.37	3.69
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.76	2.26	2.19	2.21	2.26	2.34	2.45	2.60	2.80
	30% SUT, 70% TT	3.35	2.62	2.47	2.44	2.46	2.52	2.60	2.72	2.87

75 mph, 25%, 1 min	50% SUT, 50% TT	2.84	2.27	2.17	2.16	2.18	2.23	2.30	2.40	2.53
	70% SUT, 30% TT	2.83	2.15	2.04	2.03	2.05	2.10	2.17	2.26	2.38
75 mph, 50%, 1 min	30% SUT, 70% TT	2.91	2.27	2.16	2.15	2.18	2.23	2.31	2.41	2.55
	50% SUT, 50% TT	2.50	2.02	1.95	1.95	1.98	2.03	2.10	2.18	2.29
	70% SUT, 30% TT	2.47	1.89	1.80	1.78	1.79	1.82	1.85	1.90	1.97
75 mph, 75%, 1 min	30% SUT, 70% TT	2.92	2.22	2.09	2.07	2.08	2.11	2.17	2.24	2.34
	50% SUT, 50% TT	2.35	1.89	1.81	1.8	1.82	1.85	1.89	1.94	2.01
	70% SUT, 30% TT	2.48	1.87	1.75	1.71	1.71	1.71	1.73	1.75	1.78
75 mph, 100%, 1 min	30% SUT, 70% TT	2.77	2.12	2.00	1.97	1.97	1.99	2.03	2.08	2.15
	50% SUT, 50% TT	2.31	1.85	1.76	1.74	1.73	1.74	1.76	1.79	1.82
	70% SUT, 30% TT	2.43	1.78	1.66	1.62	1.61	1.61	1.62	1.63	1.65

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

APPENDIX J EC_PCE AS FUNCTION OF DATA AGGREGATION INTERVAL LEVEL FOR ALL LENGTH

(a) EC_PCE as a function of data aggregation interval level for 0.125 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.31	2.77	2.68	2.68	2.74	2.85	3.00	3.20	3.49
	50% SUT, 50% TT	2.72	2.30	2.25	2.27	2.33	2.42	2.54	2.71	2.93
	70% SUT, 30% TT	2.50	2.16	2.12	2.15	2.20	2.29	2.40	2.54	2.73
75 mph, 0%, 5 min	30% SUT, 70% TT	2.83	2.46	2.41	2.44	2.50	2.61	2.75	2.94	3.21
	50% SUT, 50% TT	2.53	2.19	2.15	2.18	2.24	2.33	2.44	2.59	2.79
	70% SUT, 30% TT	2.42	2.11	2.08	2.10	2.15	2.22	2.31	2.44	2.60
75 mph, 0%, 10 min	30% SUT, 70% TT	2.81	2.44	2.39	2.42	2.48	2.59	2.72	2.91	3.16
	50% SUT, 50% TT	2.49	2.17	2.13	2.16	2.22	2.30	2.41	2.56	2.75
	70% SUT, 30% TT	2.38	2.09	2.06	2.08	2.13	2.20	2.30	2.42	2.57
75 mph, 0%, 15 min	30% SUT, 70% TT	2.81	2.44	2.39	2.42	2.48	2.58	2.72	2.91	3.16
	50% SUT, 50% TT	2.47	2.16	2.13	2.15	2.21	2.29	2.40	2.54	2.73
	70% SUT, 30% TT	2.35	2.07	2.05	2.07	2.12	2.19	2.28	2.40	2.55
75 mph, 0%, 20 min	30% SUT, 70% TT	2.74	2.39	2.34	2.37	2.43	2.52	2.65	2.82	3.05
	50% SUT, 50% TT	2.42	2.11	2.08	2.11	2.16	2.24	2.34	2.48	2.65
	70% SUT, 30% TT	2.32	2.06	2.03	2.05	2.09	2.16	2.24	2.35	2.48

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(b) EC_PCE as a function of data aggregation interval level for 0.375 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83

75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.52	2.86	2.74	2.74	2.80	2.90	3.06	3.27	3.57
	50% SUT, 50% TT	2.86	2.35	2.28	2.30	2.36	2.45	2.57	2.74	2.96
	70% SUT, 30% TT	2.62	2.21	2.16	2.18	2.23	2.31	2.42	2.57	2.77
75 mph, 0%, 5 min	30% SUT, 70% TT	3.01	2.53	2.46	2.48	2.55	2.65	2.80	2.99	3.27
	50% SUT, 50% TT	2.66	2.24	2.19	2.21	2.27	2.35	2.47	2.62	2.83
	70% SUT, 30% TT	2.55	2.16	2.11	2.13	2.17	2.24	2.34	2.47	2.63
75 mph, 0%, 10 min	30% SUT, 70% TT	2.93	2.49	2.43	2.45	2.51	2.61	2.75	2.94	3.20
	50% SUT, 50% TT	2.63	2.22	2.17	2.19	2.25	2.33	2.44	2.59	2.78
	70% SUT, 30% TT	2.51	2.14	2.10	2.11	2.16	2.23	2.32	2.45	2.61
75 mph, 0%, 15 min	30% SUT, 70% TT	2.90	2.48	2.42	2.44	2.50	2.60	2.75	2.94	3.19
	50% SUT, 50% TT	2.60	2.21	2.16	2.18	2.23	2.32	2.43	2.57	2.76
	70% SUT, 30% TT	2.47	2.12	2.08	2.10	2.14	2.21	2.31	2.43	2.58
75 mph, 0%, 20 min	30% SUT, 70% TT	2.85	2.44	2.38	2.39	2.45	2.55	2.68	2.85	3.09
	50% SUT, 50% TT	2.54	2.16	2.11	2.13	2.19	2.26	2.37	2.50	2.68
	70% SUT, 30% TT	2.44	2.10	2.06	2.07	2.11	2.18	2.26	2.37	2.51

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(c) EC_PCE as a function of data aggregation interval level for 0.625 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.70	2.94	2.80	2.79	2.84	2.95	3.10	3.32	3.63
	50% SUT, 50% TT	2.92	2.38	2.30	2.31	2.37	2.46	2.58	2.75	2.98
	70% SUT, 30% TT	2.69	2.23	2.18	2.19	2.24	2.33	2.44	2.59	2.79
75 mph, 0%, 5 min	30% SUT, 70% TT	3.16	2.59	2.51	2.52	2.58	2.69	2.84	3.04	3.32
	50% SUT, 50% TT	2.74	2.27	2.21	2.23	2.28	2.37	2.48	2.64	2.85
	70% SUT, 30% TT	2.62	2.19	2.13	2.14	2.19	2.26	2.36	2.48	2.65
75 mph, 0%, 10 min	30% SUT, 70% TT	3.02	2.53	2.46	2.47	2.53	2.64	2.78	2.97	3.23
	50% SUT, 50% TT	2.72	2.26	2.20	2.21	2.26	2.35	2.46	2.61	2.81
	70% SUT, 30% TT	2.58	2.17	2.12	2.13	2.17	2.24	2.34	2.46	2.62
	30% SUT, 70% TT	2.98	2.51	2.44	2.46	2.52	2.62	2.77	2.96	3.22

75 mph, 0%, 15 min	50% SUT, 50% TT	2.68	2.24	2.18	2.20	2.25	2.33	2.44	2.59	2.78
	70% SUT, 30% TT	2.54	2.15	2.10	2.11	2.16	2.23	2.32	2.44	2.60
75 mph, 0%, 20 min	30% SUT, 70% TT	2.95	2.47	2.40	2.42	2.48	2.57	2.70	2.88	3.12
	50% SUT, 50% TT	2.60	2.19	2.13	2.15	2.20	2.28	2.38	2.52	2.70
	70% SUT, 30% TT	2.50	2.13	2.08	2.09	2.13	2.19	2.27	2.38	2.52

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(d) EC_PCE as a function of data aggregation interval level for 0.875 mile

Group	**Truck Composition**	**Truck Percentage (%)**								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.79	2.97	2.82	2.81	2.86	2.97	3.13	3.35	3.66
	50% SUT, 50% TT	2.95	2.39	2.31	2.32	2.37	2.46	2.59	2.76	2.99
	70% SUT, 30% TT	2.72	2.25	2.18	2.20	2.25	2.33	2.45	2.60	2.80
75 mph, 0%, 5 min	30% SUT, 70% TT	3.24	2.63	2.53	2.54	2.60	2.71	2.85	3.06	3.34
	50% SUT, 50% TT	2.77	2.28	2.22	2.24	2.29	2.37	2.49	2.65	2.86
	70% SUT, 30% TT	2.67	2.21	2.14	2.15	2.20	2.27	2.37	2.49	2.66
75 mph, 0%, 10 min	30% SUT, 70% TT	3.07	2.55	2.47	2.48	2.55	2.65	2.79	2.98	3.25
	50% SUT, 50% TT	2.75	2.27	2.21	2.22	2.27	2.35	2.47	2.62	2.82
	70% SUT, 30% TT	2.63	2.19	2.13	2.14	2.18	2.25	2.35	2.47	2.63
75 mph, 0%, 15 min	30% SUT, 70% TT	3.02	2.52	2.45	2.47	2.53	2.63	2.77	2.97	3.23
	50% SUT, 50% TT	2.71	2.25	2.19	2.21	2.26	2.34	2.45	2.59	2.79
	70% SUT, 30% TT	2.58	2.17	2.11	2.12	2.17	2.24	2.33	2.45	2.61
75 mph, 0%, 20 min	30% SUT, 70% TT	2.99	2.49	2.42	2.43	2.49	2.58	2.71	2.89	3.13
	50% SUT, 50% TT	2.63	2.20	2.14	2.16	2.21	2.28	2.39	2.52	2.70
	70% SUT, 30% TT	2.53	2.14	2.09	2.09	2.13	2.19	2.28	2.39	2.53

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(e) EC_PCE as a function of data aggregation interval level for 1.25 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.85	3.00	2.84	2.82	2.88	2.99	3.15	3.37	3.68
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.75	2.26	2.19	2.20	2.26	2.34	2.45	2.60	2.80
75 mph, 0%, 5 min	30% SUT, 70% TT	3.29	2.64	2.54	2.55	2.61	2.72	2.87	3.07	3.36
	50% SUT, 50% TT	2.79	2.29	2.23	2.24	2.29	2.38	2.50	2.65	2.86
	70% SUT, 30% TT	2.70	2.22	2.15	2.16	2.21	2.28	2.37	2.50	2.67
75 mph, 0%, 10 min	30% SUT, 70% TT	3.10	2.56	2.48	2.49	2.55	2.65	2.80	2.99	3.26
	50% SUT, 50% TT	2.77	2.28	2.21	2.22	2.28	2.36	2.47	2.62	2.82
	70% SUT, 30% TT	2.66	2.20	2.14	2.15	2.19	2.26	2.35	2.48	2.64
75 mph, 0%, 15 min	30% SUT, 70% TT	3.04	2.53	2.46	2.47	2.54	2.64	2.78	2.97	3.24
	50% SUT, 50% TT	2.73	2.26	2.20	2.21	2.26	2.34	2.45	2.60	2.79
	70% SUT, 30% TT	2.62	2.18	2.12	2.13	2.17	2.24	2.34	2.46	2.62
75 mph, 0%, 20 min	30% SUT, 70% TT	3.02	2.50	2.42	2.44	2.49	2.59	2.72	2.90	3.14
	50% SUT, 50% TT	2.65	2.20	2.14	2.16	2.21	2.29	2.39	2.53	2.71
	70% SUT, 30% TT	2.55	2.15	2.09	2.10	2.14	2.20	2.28	2.39	2.53

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.

(f) EC_PCE as a function of data aggregation interval level for 1.5 mile

Group	Truck Composition	Truck Percentage (%)								
		5	15	25	35	45	55	65	75	85
HCM 2016	30% SUT, 70% TT	2.30	2.04	1.97	1.97	1.97	1.97	1.97	1.97	1.97
	50% SUT, 50% TT	2.31	2.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93
	70% SUT, 30% TT	2.12	1.89	1.83	1.83	1.83	1.83	1.83	1.83	1.83
75 mph, 0%, 1 min* (Empirical)	30% SUT, 70% TT	3.87	3.00	2.85	2.83	2.88	2.99	3.15	3.37	3.69
	50% SUT, 50% TT	2.97	2.40	2.31	2.32	2.38	2.47	2.59	2.76	2.99
	70% SUT, 30% TT	2.76	2.26	2.19	2.21	2.26	2.34	2.45	2.60	2.80
	30% SUT, 70% TT	3.30	2.65	2.55	2.55	2.62	2.72	2.87	3.08	3.36

75 mph, 0%, 5 min	50% SUT, 50% TT	2.80	2.29	2.23	2.24	2.29	2.38	2.50	2.65	2.86
	70% SUT, 30% TT	2.72	2.23	2.16	2.16	2.21	2.28	2.38	2.50	2.67
75 mph, 0%, 10 min	30% SUT, 70% TT	3.11	2.56	2.48	2.49	2.55	2.66	2.80	2.99	3.26
	50% SUT, 50% TT	2.78	2.28	2.21	2.23	2.28	2.36	2.47	2.62	2.82
	70% SUT, 30% TT	2.67	2.21	2.14	2.15	2.19	2.26	2.36	2.48	2.64
75 mph, 0%, 15 min	30% SUT, 70% TT	3.05	2.54	2.46	2.48	2.54	2.64	2.78	2.98	3.24
	50% SUT, 50% TT	2.73	2.26	2.20	2.21	2.26	2.34	2.45	2.60	2.79
	70% SUT, 30% TT	2.63	2.18	2.12	2.13	2.17	2.24	2.34	2.46	2.62
75 mph, 0%, 20 min	30% SUT, 70% TT	3.03	2.51	2.43	2.44	2.50	2.59	2.72	2.90	3.14
	50% SUT, 50% TT	2.65	2.20	2.15	2.16	2.21	2.29	2.39	2.53	2.71
	70% SUT, 30% TT	2.56	2.15	2.09	2.10	2.14	2.20	2.28	2.39	2.53

*Note: The marks mean for this group the EC-PCEs are estimated under the conditions with 75 mph speed limit, 0% truck passing restriction, and 1-minute data aggregation level. The marks in the following cells have the same explanations.