Development of passenger car equivalents for freeway merging sections

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DEVELOPMENT OF PASSENGER CAR EQUIVALENTS FOR FREEWAY MERGING SECTIONS

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Development of Passenger Car Equivalents for Freeway Merging Sections

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ABSTRACT

Development of Passenger Car Equivalents for Freeway Merging Sections

by

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The Highway capacity manual (HCM, 2000) uses the Passenger Car Equivalents (PCE) to estimate the impact of trucks on freeways. PCE values for trucks recommended by HCM 2000 for level terrain are same under different truck percentages and different freeway conditions such as merging, diverging, weaving section etc. Several studies have shown that the PCE values vary for different traffic conditions such as for weaving, oversaturated conditions etc. The objective of this thesis is to develop the PCE values for freeway merging sections using simulation software CORSIM. The equal density methodology is used to compute the PCE values. For this study PCE values are calculated for different traffic conditions including truck percentage, volume ratios (VR), and LOS. Analysis is also done with trucks on freeway only and ramp only.

From the results of the study, estimated PCE values vary with level of service, truck percentage and volume ratio for merging section, adjacent upstream and downstream section. The study also shows that HCM overestimates the capacity of the merging sections. Since the results of this study are based on only one case study location, the
PCE values developed may not be transferable to other locations and/or for other traffic conditions. However, the general relationship between the PCE values and traffic conditions such as percentage of trucks, VR, LOS are expected to be the similar. More extensive studies are recommended to validate the findings of this study.
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1.1 Background Information

Highway capacity is typically expressed in terms of passenger cars per hour per lane (pcphpl). Highway Capacity Manual (HCM, 2000) defines the capacity of a multilane highway as the maximum sustained hourly flow rate at which vehicles reasonably can be expected to traverse a uniform segment under prevailing roadway and traffic conditions. The presence of large and/or low performance vehicles in the traffic stream results in a reduction in the capacity. The HCM reasons that the reduction in throughput is due to the fact that heavy vehicles take up more space and have lower performance as compared to passenger cars, especially on grades. The HCM defines the passenger car equivalents (PCE) as the number of passenger cars displaced by a single heavy vehicle of a particular type under specified roadway, traffic, and control conditions. The traffic volumes containing a mix of vehicle types must be converted into an equivalent flow of passenger cars using PCE. The procedure in the HCM allows that freeway traffic volume (containing a mix of vehicle types) is adjusted by the use of a heavy vehicle factor, $f_{HV}$, to obtain an equivalent flow rate of passenger cars. The heavy vehicle adjustment factor is based on the PCE of trucks/buses, and recreation vehicles (RVs). According to the HCM 2000 (1), the heavy vehicle adjustment factor is shown in Equation 1.
\[ f_{HV} = \frac{1}{1 + E_T (1 - P_T) + E_R (1 - P_R)} \]  

where,

- \( P_T \) = proportion of trucks/buses in traffic stream
- \( P_R \) = proportion of Recreational Vehicles (RVs) in the traffic stream
- \( E_T \) = PCE value for trucks/buses
- \( E_R \) = PCE value for RVs respectively.

The corresponding equivalent number of passenger cars for a given mix traffic volume \( (V_m) \) is shown in Equation 2.

\[ V_{PC} = \frac{V_M}{f_{HV}} \]  

The HCM (2000) is used for design and operational analysis of highways. The PCE values used in the HCM (2000) are developed based on the equivalent density approach (Tiwari, 2000). According to Van Aerde and Yagar (1984) “Passenger car equivalents have generally been assumed to be similar for capacity, speed, platooning, and other types of analysis. This notion appears to be incorrect and is perhaps one of the main sources of discrepancies among the various PCE studies”.

Moreover, the usage of PCE values developed for basic freeways segment may not be suitable for the on-ramps and merging section because the lane changing on the basic freeway segment is discretionary, but on the on-ramps (merging scenarios) lane change is mandatory to merge in the mainline. In merging scenarios, the gaps may however be sufficient to merge the passenger cars in the traffic but heavy vehicles requires more time headway and space headway in order to merge and they have to wait for more time to find a suitable gap under flow conditions. The length of acceleration lanes may impact
the PCE values for trucks, because it allows trucks to search for appropriate headways and they can provide enough acceleration that can reduce the turbulence caused by the heavy vehicles. The PCE values may vary from free flow conditions to the congested conditions.

1.2 Problem Statement

The HCM (2000) assumes that PCE values for heavy vehicle traffic are the same for basic freeway sections, merging sections, diverging sections, on-ramps, off-ramps and weaving sections under given scenario of geometric conditions. Also the PCE values in HCM (2000) remain the same for different levels of congestion. During the low traffic flows (under-saturated conditions) passenger cars can easily maneuver even in the presence of heavy vehicles because of the sufficient availability of acceptable gaps, under heavy traffic flow (saturated conditions) maneuvering opportunity are reduced. Therefore, the PCE values may be different under the different levels of congestion (free flow conditions to forced flow conditions). The study conducted by Vermijis (1998) showed that PCE values are different for weaving sections as compared to the basic freeway segment. The study conducted by Webster and Elefteriadou (1999) showed that the PCE values are dependent upon the traffic flow; values of PCE are different for low flow condition and high flow conditions.

1.3 Research Objective

The objective of this study is to evaluate current HCM PCE values and to compute new PCE values for freeway on-ramp merging sections. Analysis is done for different
flow rates and different truck percentages. For this study, development of PCE values for merging sections and basic freeway segments is done only for level terrain. This study will equip transportation officials with a better tool to design the merging section, by accurately taking into account effect of heavy vehicles based on the new developed PCE values. The study is conducted using computer simulation for a selected case study location.

1.4 Organization of Thesis

This thesis is organized in the seven chapters.

Chapter 2 presents the past research related to this subject. Literature related to PCE values for basic freeway segments, capacity analysis, and relevant literature related to model calibration of freeways is also reviewed.

Chapter 3 describes the methodology used for this research. This includes selection of case study location, data collection, simulation software CORSIM, and model calibration. It describes different approaches to calculate PCE values and the parameters collected from the field for the traffic simulation program.

Chapter 4 describes the model calibration procedure. The different types of simulation parameters that are studied in the research are heavy vehicle percentages, acceleration lane length, flow on on-ramps and freeway segment etc.

Chapter 5 presents and discusses the results and the limitations of the study.

Chapter 6 presents the conclusions and recommendations drawn from the results of this research.

Chapter 7 presents the limitations of the study and software.
CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

In this chapter, methodologies to calculate PCE values and literature related to analysis of PCE values for freeway section is discussed.

HCM recommends users to use the same PCE values for freeway, on-ramps and merging sections. The PCE values are developed based on an average weight-to-power ratio of 167 lb/hp, which is typical of trucks on multilane highways in the United States. In basic freeway segments the lane change is discretionary, but in the case of merging sections, vehicles coming for on-ramp have to merge with in limited time and space. The heavy vehicles have difficulties in merging if the volume on freeway is high because of non-availability of acceptable gaps in terms of time and headway, which in turn can affect the PCE values for merging section. The value of PCE may be different for the different levels of congestion.

2.2 Methodologies to calculate PCE values

This section summarizes the various methods that have been used for calculating PCE values, categorizing them by performance measure on which PCE values are based.
2.2.1 PCE values based on headways

Cunagin and Messer (1982) used the headways method used in which the relative amount of space consumed by a vehicle as the basis for calculating PCE values. Under the idea that the size of the headway depends on the following vehicle in a pair of vehicles, PCE values can be estimated as Equation 3.

\[
PCE_{ij} = \frac{H_{ij}}{H_{pcj}}
\]

where

\(PCE_{ij}\) = PCE of vehicle type \(i\) under conditions \(j\),

\(H_{ij}\) = average headway for vehicle type \(i\) for condition \(j\), and

\(H_{pcj}\) = average headway of passenger car for conditions \(j\).

2.2.2 PCE values based on relative delays

Cunagin and Messer (1983) used an extension of the HCM method (1965) to calculate PCE values for multilane highways based on relative delay. In their approach, they used a combination of the Walker method (1975) of relative number of passings and the relative delay method. They recognized that on multilane highways, passing or overtaking vehicles are inhibited only by concurrent flow traffic. The results from the study showed that the PCE for trucks is equal to 1.5 for LOS A, for LOS B to D, PCE is 1.6 and for LOS D and E, PCE is 1.7. The formula used to calculate the PCE values is shown in Equation 4.

\[
E_{r} = \frac{(OT_i / VOL_i) \left(1 / SP_M \right) - \left(1 / SP_h \right)}{(OT_{Ipc} / VOL_{Ipc}) \left(1 / SP_{PC} \right) - \left(1 / SP_{h} \right)}
\]

where,

\(OT_i\) = the number of over takings of vehicle type \(i\) by passenger cars,
\[ \text{VOL}_i = \text{the volume of vehicle type } i, \]
\[ \text{OT}_{\text{LPC}} = \text{the number of overtakings of lower performance passenger cars by passenger cars}, \]
\[ \text{VOL}_{\text{LPC}} = \text{the volume of lower performance passenger cars}, \]
\[ \text{Sp}_\text{M} = \text{the mean speed of the mixed traffic stream}, \]
\[ \text{Sp}_\text{B} = \text{mean speed of the base traffic stream with only high performance passenger cars, and} \]
\[ \text{Sp}_\text{PC} = \text{the mean speed of the traffic stream with passenger cars only.} \]
\[ \text{Sp}_\text{PC}^{\text{P}} = \text{is the mean speed of the traffic stream with only passenger cars} \]

Golias (2003) investigated the influence of taxis on urban traffic conditions and found out the taxi equivalent factors (TEF) are different from each other based on delays and capacities. Using computer simulation and the TEF, a concept similar to the passenger car equivalents for heavy vehicles, the impacts of taxi traffic on the capacity, and delays at urban road sections is quantified. The taxi equivalence factor is based either on capacity or on delay. The result suggested that the TEF is significantly higher for one-lane roads than it is for two-lane roads. The TEF were dependent on green time to cycle length \((g/C)\) ratios, volume/capacity ratios and taxis percentage. From the study, at \(g/C = 0.5\), \(V/C = 1.0\) and percentage of taxis = 8%, the TEF values based on the delays was 2.217 but on the basis of the capacity it was 1.055. This indicated that the presence of taxis did not seem to affect road capacity, but it significantly affected traffic delay.

2.2.3 PCE values based on speed

Van Aerde and Yagar (1984) developed a methodology to calculate PCE based on relative rate of speed reduction. This PCE was intended for use in average speed analysis
of capacity, which is unique to two lane highways. A multiple linear regression model was developed to estimate the free flow speed and coefficients of various percentile speeds for each vehicle type. The multiple linear regression models were given in Equation 5. The results for the study were reported for generalized conditions under different speed percentiles using the Equation 5, for 10th percentile speed PCE value was equal to 11.4, for 50th speed percentile PCE value was equal to 6.1 and for 90th speed percentile the PCE values was reduced to 3.8. The authors attributed the reduction in the PCE values based on the speed percentile to the relative effect of trucks on lower percentile driver was more pronounced.

\[
\text{Percentile speed} = v_f + C_1(n_1) + C_2(n_2) + C_3(n_3) + C_4(n_4) + C_5(n_5) \quad (5)
\]

where,

- \( v_f \) = free flow speed,
- \( n_1 \) = number of passenger cars,
- \( n_2 \) = number of trucks,
- \( n_3 \) = number of RVs,
- \( n_4 \) = number of other vehicles,
- \( n_5 \) = number of opposing vehicles, and
- Coefficients \( C_1 \) to \( C_5 \) are the relative sizes of speed reductions for each vehicle type.

Although this model was formulated for two lane highways with opposing traffic flow, it could be applied to multilane highways by setting the coefficient \( C_5 \) to zero. Using the speed reduction coefficients given in Equation 6,

\[
\text{The PCE for a vehicle type } n = C_n/C_1 \quad (6)
\]
where,

\( C_n \) = the speed reduction coefficient for vehicle type \( n \)

\( C^\prime \) = the speed reduction coefficient for passenger cars.

2.2.4 PCE values based on platoon formation

Van Aerde and Yagar (1984) developed a methodology to calculate PCE values based on platoon formation. On two lane highways, platooning is caused when fast moving vehicles catches up with the slow moving vehicles and not being able to overtake. Heavy vehicles have the higher probability of leading the platoon as compared to the passenger cars. The PCE values are calculated by using the ratios of percentage leads, by vehicle types, to percentage of total freeway traffic count, by vehicle type. When these ratios were normalized with respect to the original ratio of passenger cars, PCE were determined in terms of platoon leadership. From the study, the results of PCE values based on the platooning formation of trucks under low volume (less than 650 vph) was 1.23 and under high volume the PCE value was equal to 1.20.

2.2.5 PCE values based on performance measure

Huber (1982) conducted a study on PCE values and developed in Equation 7, which relates PCE values to the flow of a passenger car only traffic stream and a mixed vehicle traffic stream. The effect of trucks was quantified by relating the traffic flows for an equal level of service (LOS). Any equivalent LOS or impedance could be chosen for the equality. If for example, density was used to define the equal LOS criteria, the flow-density relationship could be used to relate the traffic flows at an equal density value. Huber’s basic Equation is shown in Equation 7:
\[ E_T = \frac{1}{P_T} \left( \frac{q_B}{q_M} - 1 \right) + 1 \]  

(7)

where,

- \( E_T \) = passenger car equivalent
- \( P_T \) = percentage of trucks in the mixed flow
- \( q_B \) = the base flow rate (passenger cars only),
- \( q_M \) = the mixed flow rate,

Huber used the assumption of equal average travel time as the measure of LOS. Equal average travel time on a one-mile segment is equivalent to the inverse of the space mean speed. The consequence of his assumption of equal speed was that PCE values decreased as volumes increases. A slow moving truck will have a smaller impact on the average speed when the total volume is higher. Huber found this result objectionable and suggested that equal total travel time be used as a measure of LOS. Huber formulated equal total travel time as the volume in vehicles per hour multiplied by the average travel time in hours per mile. By this representation, equal total travel time was equivalent to equal density because it described equal vehicle occupancy on the roadway in vehicles per mile. The calculation of PCE by equal density is discussed later.

Sumner et al. (1984) expanded the relationship described by Huber to calculate the PCE of a single truck in a mixed traffic stream, which includes multiple truck types. This calculation requires an observed base flow, mixed flow, and flow with the subject vehicles. The equal LOS or impedance measure would cut across all three-flow curves. The relationship described by Sumner et al. is formulated as Equation 8.

\[ E_T = \frac{1}{\Delta p} \left( \frac{q_B}{q_s} - \frac{q_B}{q_M} \right) + 1 \]  

(8)
where \( q_b \) and \( q_M \) are defined in Equation (7),

\[
\Delta p = \text{the proportion of subject vehicles that is added to the mixed flow and subtracted from the passenger car proportion},
\]

\[
q_s = \text{the flow rate including the added subject vehicles.}
\]

Sumner et al. (1984) used total travel time in terms of vehicle hours as the equal measure of LOS.

Demarchi and Setti (2003) developed a method based on the estimation of an aggregate PCE and discussed the quantitative analysis of errors associated with each type of PCE. In an algebraic derivation, they proved that PCE values developed for a single truck type in a mixed traffic flow containing multiple truck types using Equation (8) did not fully account for the interaction between trucks. Authors proposed a model in which aggregate equivalence factor is calculated for the mix flow was used. Authors modified the Equation proposed by Sumner (1984) and added a coefficient of \( \delta \) which can take into account the interaction between different truck types. The value of \( \delta = 0 \), if there is no interaction existed between the two truck types, if \( \delta > 0 \) then \( E_1 \) and \( E_2 \) are underestimated and \( \delta < 0 \) if the \( E_1 \) and \( E_2 \) are overestimated.

\[
\frac{q_b}{q_M} = 1 - p_1 - p_2 + p_1E_1 + p_2E_2 + \delta
\]

(9)

where \( q_M \) is defined in Equation (7) and \( q_s \) is defined in Equation (8),

\[
p_1 = \text{proportion of the heavy vehicle type 1},
\]

\[
p_2 = \text{proportion of the heavy vehicle type 2},
\]

\[
E_1 = \text{passenger car equivalent for heavy vehicle type 1},
\]

\[
E_2 = \text{passenger car equivalent for heavy vehicle type 2}, \text{ and}
\]
\( \delta = \text{Truck interaction factor.} \)

\[
\begin{array}{c}
\text{Impedance} \\
\text{Subject flow (subject vehicle + mixed flow)} \\
\text{Same impedance} \\
\text{Mixed Flow (Cars + trucks)} \\
\text{Base Flow (Car only)} \\
\end{array}
\]

Figure 1 PCE derivation for more than one type of trucks

2.3 Preview of PCE research

The PCE values recommended by the HCM (1994) were developed based on the study conducted by Linzer et al. (1979). Authors only took into account the basic freeway segments and constant volume/capacity ratios constant for given LOS, whereby PCE values were calibrated such that the mixed traffic flow will produce the same v/c ratio as a passenger car only flow. The studies did not take into account the PCE values based on different levels of congestion. The research by Linzer et al. (1979) made use of design charts resulting from micro-simulation done by the Midwest Research Institute (MRI). The design chart related the percent grade, mixed vehicle flow, and percent reference trucks to percent capacity (equivalent to v/c ratio). The typical truck used in calculation of PCE values for the Linzer et al. was of 182.7 kg/kW (300 lb/hp), slightly less than the 197.9 kg/kW (325 lb/hp) truck used in the HCM (1965), and reflected the
increased performance of trucks since the 1960’s. In addition, a light truck of 91.4 kg/kW (150 lb/hp) and a heavy truck of 213.2 kg/kW (350 lb/hp) were used to calculate PCE values. Truck performance curves were used from research conducted by Pennsylvania State University, with initial truck speed of 88.5 km/h (55 mi/h). Since the research by Linzer et al. (1979) calculated PCE values for truck populations with a single weight to power ratio, the percent reference trucks method proposed by the MRI was used by assuming that only trucks of the given weight to power ratio existed. The PCE was formulated as Equation 10.

\[
E_T = \frac{q_B - q_M (1 - P_T)}{q_M + P_T}
\]  

(10)

where \(E_T\) and \(P_T\) are defined in Equation (7),

\[ q_B = \text{the equivalent passenger car only flow rate for a given v/c ratio, and} \]

\[ q_M = \text{the mixed flow rate for a given v/c ratio.} \]

Webster and Elefteriadou (1999) conducted simulation study for PCE values for basic freeway segments and showed that PCE are dependant on traffic volume, grade/grade length and percentage of trucks. They used the equivalent traffic density as main parameter on which PCE values were developed. The authors developed their model in FRESIM, to estimate the values of PCE for basic freeway segment. The authors investigated the influence of different trucks types based on their weight/power (wt/hp) ratio and length of the trucks on the PCE values. Authors also looked at several other parameters like number of lanes, free flow speed, percentage of trucks, grades and length of grade. From the study, PCE values for basic freeway segment for level segments were equal to 1.0 for flow up to 1500 vphpl for truck percentages up to 15%. When the flow
was increased to the 2000 vphpl, the PCE value increased to 1.5 for 5% trucks and for 15 to 25% trucks the PCE value was 2.0. They summarized that PCE value tends to increase with traffic flow, free flow speed, and grade, and decreases with an increase in the truck percentage and number of lanes. Authors reported decrease in PCE values with increase in percentage of trucks under low flow rate conditions and opposite under high flow rate conditions. The authors recommended that for the future study, the effect on PCE values for ramps and weaving section should be analyzed.

Elefteriadou et al. (1997) developed the passenger car equivalents for freeways, two lane highways and arterials. The study investigated the impact of different variables such as levels of traffic flows, truck percentage, grade and length of grade. The study used speed as a performance measure to develop the PCE values. The results showed that the PCE were a function of truck percentage and weight to horse power ratio. PCE for two lane freeways with 0% grade, length of grade equal to 0.805 km, truck percentage =15% and low traffic flow, for single unit truck (length = 12.2 m and wt/hp = 300) PCE was equal to 1, and under same scenario, double trailer (length = 12.2 m and wt/hp = 300) PCE was equal to 2. The study also showed that the PCE values for arterials under low traffic volumes, level terrain and with 12% trucks, can be as high as 4-5 for two lanes based on weight/horse power ratio and in case of four lanes, for similar traffic volumes and same truck percentage the PCE values were as low as 1-2 based on weight/horse power ratio.

Al-Kaisy et al. (2002) investigated the effect of heavy vehicles on traffic is greater during congestion as compared to the under-saturated conditions using the field observation. Authors computed the PCE using the queue discharge flow (QDF) capacity
and they assumed that the flow from queue discharge flow shows minimal variation if traffic stream was uniform and composed of only passenger cars. Non linear programming was performed optimizations on a number of data sets required for the two test sites. The case studies sites were located in Ontario Canada for this study and one site was entrance ramp merge area. The authors developed the PCE values for oversaturated conditions (LOS F); authors used the queue discharge flow of a bottleneck situation. They selected the two sites and emphasized that HCM (2000) underestimated the impact of heavy vehicle on freeways after the onset of congestion. Authors suggested that ratio of average headways between heavy vehicles and passenger cars increases upon the formation of congestion and acceleration/deceleration cycles experienced during congestion imposes an extra limitation on the performance of heavy vehicles thus increases the effect on traffic flow. The PCE value for level terrain from the site study, from the site was 2.36 as compared to HCM (2000) recommended 1.5 and for second site (1-km long, 3% upgrade is 2.0) the PCE values for one direction of travel was 3.21 and for other was 2.7 as compared to the HCM recommended value of 2.0. Furthermore, the authors suggested that the PCE values are not a function of weather conditions and roadside maintenance works.

Al-Kaisy et al. (2005) conducted the study to develop PCE factors for heavy vehicles on freeways and multilane highways during congestion. The study was in continuation of the study conducted in 2002 and for this study, INTEGRATION software was used to simulate the field conditions of the sites reported in Al-Kaisy (2002). The PCE values ranges from 2.4 (for 2% heavy vehicles) to 2.7 (for 25% heavy vehicles) as compared to HCM results which is 1.5 for (2-25% of heavy vehicles). Results from the study showed
effect of grades, grade length and percentage of trucks on the PCE values. From there observations from two sites reported that PCE is underestimated from 35-75% depending upon the traffic conditions and percentage of heavy traffic. The PCE values obtained from the simulation study in 2005 are within the confidence interval limit of the results obtained from the field results obtained from 2002 study.

Vermijs (1998) studied the weaving section type A using micro-simulation mode FOSIM and showed that PCE values were not the same for the basic freeway segments and weaving sections type A. In the simulations weaving section length, weaving flow rate and truck percentage were varied to study the effects of these variables on PCE values. The study showed that the PCE values changed with the different lane configurations for weaving section type A from as low as 2.5 to as high as 3.6 for level grades which were much higher than the recommended values by HCM (2000). Authors concluded that weaving capacity declines with increasing truck percentage and increasing weaving flow rate and weaving section length had little influence on capacity. In addition to that authors reported that PCE values were not same for all the weaving configurations. The current PCE value overestimates the capacity of the weaving section by using the same values of basic freeway segment.

Rakha and Zhang (2006) used the simulation software INTEGRATION to estimate the capacity of 34 freeway-weaving sections. They studied the impact of the weaving length, weaving type, volume ratio and weaving ratio (type A, type B and type C) on the capacity of the freeway section. For illustration purposes, authors used the weaving configuration BX4 means weaving configuration type B and number of three freeway lanes and one auxiliary lane that connects the on-ramp and off-ramp. From the Equation
derived from the study conducted by Rakha (2006) estimated the capacity of the weaving section was 5734 pcph and HCM (2000) estimated the weaving section capacity equals to 7258 pcph. Authors concluded, “Furthermore, the study demonstrates that the HCM (2000) procedures tend to overestimate weaving section capacities significantly (errors in excess of 100% in some instances)”. Authors estimated the capacity for different weaving section types based on the assumption that the heavy vehicle factor was properly addressed in the HCM (2000). However, Vermijs (1998) study showed the potential impact of PCE on the capacity and stressed on the underestimation of PCE values reported by HCM (2000).

2.4 Summary of the literature review

A survey of the literature also reveals a need for an improved understanding of the usage of PCE values under the different scenarios like congested conditions, free flow conditions, weaving etc. The different methodologies can be utilized to calculate the PCE values. For this study, the equivalent density approach will be adopted as the current PCE values are based on equivalent density.

2.5 Description of the different calibration approaches

This section summarizes the different calibration approaches to calibrate the models and lane-changing model. The calibration is necessary for any microscopic simulation models, so that the model can reproduce the field conditions in the model.

Hourdakis et al. (2003) proposed an easy to follow procedure for calibration of microscopic traffic simulation models. Authors categorized the simulation parameters in

17
two major categories: global (those parameters that affect the performance of the entire model) and the local parameters (parameters that affect that local section of simulation model). Examples of the global parameters are the length, width of lanes, desired speed, max acceleration, max deceleration and local parameters like number of lanes, speed limits etc. Authors suggested that during the calibration process the global parameters are calibrated first and then followed by the local parameters. The calibration process is performed in two main stages, first volume based calibration, and then speed based calibration (speed calibration procedure is more sensitive to measure to the fluctuations of flows) and at the end, the objective based calibration (ramp queues) should be fine-tuned.

Yang and Koutsopoulos (1996) developed a rule-based lane-changing model that is applicable only for freeways. Their model was implemented in MITSIM. In the model, lane change was classified as either mandatory lane change (MLC) or discretionary lane change (DLC). Authors used a probabilistic method to model drivers' lane change behavior when they face conflicting goals. A driver considers a DLC only when the speed of the leader is below a desired speed, and checks neighboring lanes for opportunities to increase speed. Two parameters, impatience factor and speed indifference factor, were used to determine whether the current speed is low enough and the speeds of the other lanes are high enough to consider a DLC. They also developed a gap acceptance model that captures the fact that the critical gap length (defined as the minimum acceptable gap length) under an MLC situation is lower than that under a DLC situation. Authors also pointed out that in case of merging into a traffic parallel to the current lane, a gap is acceptable only when both the lead and lag gaps are acceptable.
Skabardonis (2002) studied the eight freeway weaving section areas (1-Type A, 3-Type B and 4-Type C) using the simulation software CORSIM in California. The results from the study indicate that CORSIM default model parameter generally under-predicts the traffic performance for freeway lane drops and weaving sections. The study reported that the car-following sensitivity factors, lane change aggressiveness factor, and percentage of freeway through vehicles that yield to merging traffic factors can significantly affect the calibration results. The author recommended that the position of warning sign for vehicles exiting through the off-ramp should be placed as far upstream as possible at least 3500 ft from the node designating off-ramp location. Study also recommended that the lane changing aggressiveness set to 1, percent yield equal to 40, and for the car following sensitivity factors (120, 112, 104, 96, 88, 72, 64, 56 and 48). The calibrated model form the study resulted in the average speeds with ± 5 mph of accuracy of the speeds observed at the weaving locations.

Chien et al. (2001) discussed about the effect of the free flow speed distribution on the capacity of basic freeway segment. Authors investigated how the vehicle speed distribution speed and average speed can affect the capacity of the freeway segment. The results concluded that maximum flow rates decreases with the increasing speed variances for all one lane, two-lane and three-lane freeways. The vehicle speed standard deviation under non-congested conditions decreases when the difference between the link free flow speed and the desired speed of cautious driver decreases.

Rakha and Crowther (2003) conducted a study on the comparison and calibration of FRESIM and INTEGRATION steady-state car-following behavior. The paper developed a procedure for calibrating the FRESIM Driver Sensitivity Factor using macroscopic
loop detector data. The basic model in the FRESIM incorporates the distance headway and speed differential between the lead and follower vehicle as two independent variables. The FRESIM steady-state car-following behavior is characterized by the Pipes model, which requires the calibration of three parameters: the jam density headway, the free-speed, and a Driver Sensitivity Factor ($c_3$). May (1990) suggests typical values of jam densities to range between 110 and 150 vehicles/km. The Pipes model represents a linear increase in the travel speed as the distance headway increases. The Driver Sensitivity Factor ($c_3$) defines the slope of the speed-headway relationship, while the intercept with the x-axis is defined by the jam density headway ($h_j$) of traffic. A third parameter required in characterizing the speed-headway relationship is the free-speed, or the maximum speed of travel when a vehicle is not constrained by the surrounding traffic. Consequently, the Pipes car-following model requires the calibration of three parameters: the free speed, the jam density headway, and a driver sensitivity factor.

The Pipes car-following relationship in the congested regime is characterized by Equation 11. Since traffic stream density is the inverse of the space headway, Equation 12 describes the basic speed-density relationship that evolves from the Pipes car-following model.

$$h = h_j + c_3 u$$  \hspace{1cm} (11)

$$k = \frac{1}{h_j + c_3 u}$$  \hspace{1cm} (12)

where,

$h$ = distance headway between the front bumper of lead vehicle and front bumper of the following vehicle;
\[ h_j = \text{space headway between the vehicles when they are completely stopped;} \]
\[ c_3 = \text{driver sensitivity factor;} \]
\[ k = \text{density (vpmpl),} \]
\[ u = \text{the speed of the vehicle,} \]

Using the basic traffic stream relationship, flow is equal to density multiplied by speed, in combination with Equation 12; the speed-flow relationship can be derived, as shown in Equation 13.

\[ q = \frac{u}{h_j + c_3u} \quad \text{(13)} \]

where \( h_j, c_3, \) and \( u \) are defined in Equation (11) and (12),

\[ q = \text{flow (vphpl),} \]

Equation 14 shows that the slope of the speed-flow relationship is computed as the derivative of flow with respect to speed. Given that the jam density headway of vehicles is non-negative and non-zero, the final form of the slope computed by Equation 15 is a strict monotonic function. Consequently, the maximum flow will occur at the extreme point (i.e. at the maximum speed which is the free-speed).

\[ \frac{\partial q}{\partial u} = \frac{\partial}{\partial u} \left( \frac{u}{h_j + c_3u} \right) = \frac{1}{h_j + c_3u} - \frac{c_3u}{(h_j + c_3u)^2} \quad \text{(14)} \]

\[ \frac{\partial q}{\partial u} = \frac{h_j}{(h_j + c_3u)^2} \quad \text{where: } h_j > 0 \quad \text{(15)} \]

where \( h_j, c_3, \) and \( u \) are defined in Equation (11) and (12),

\[ \frac{dq}{du} = \text{change of flow with change of speed.} \]
The maximum flow can be derived from Equation 13 by substituting the flow for the roadway capacity \((q_c)\) and the speed for the free-flow-speed \((u_f)\). By rearranging the terms in Equation 16, the Driver Sensitivity Factor can be computed, as defined in Equation 17. Equation 17 requires three parameters that can be obtained from standard loop detector data. These parameters include the roadway capacity (maximum flow rate), the spacing of vehicles at jam density, and the roadway free-speed.

\[
q_c = \frac{u_f}{h_j + c_3 u_f}
\]  

\[
c_3 = \frac{1}{q_c} \left( -\frac{h_j}{u_f} \right) \quad \text{where} \ h_j > 0
\]

where \(h_j\) and \(c_3\) are defined in Equation (11) and (12),

- \(q_c\) = roadway capacity, and

- \(u_f\) = free flow speed in mph.

2.6 Summary of the model calibration

The different model calibration procedure highlighted the need of the calibration in the traffic simulation software’s to obtain more accurate and reliable results. For this study, the model calibration will be done based on the measure of effectiveness, speed (on-ramps, merging, upstream and downstream section) and traffic flows (upstream and on-ramp) from the simulation model.
CHAPTER 3

DATA COLLECTION AND METHODOLOGY

3.1 Introduction

In this chapter, data collection and methodology is discussed. This chapter is divided into following section. The following sections are presented which will discuss the steps used for data collection and methodology used.

- Selection criteria for the case study location,
- Data collection methodology,
- Computer Simulation model,
- Model calibration, and
- Methodology behind the development of PCE values.

3.2 Case study location criteria

The section to be studied for this case study was freeway on-ramp section, which includes the merging section, upstream and downstream section on the freeway merging section. The selection of the freeway and on-ramps junction was based on the following criteria:

- No adjacent on-ramps or off-ramps within 2500 ft of the selected on-ramp,
- The gradient of the freeway will be close to 0% (less than 2%),
• The on-ramp will either be level or downgraded,
• No horizontal curves within the vicinity of on-ramp, and
• Truck percentage will be high (greater than 10%)

Figure 2 Flow chart for the study of methodology

3.3 Data requirements

For this study the following input parameters were required for the model development, model calibration and simulation:
• Traffic flow for on-ramps and freeways (for each lanes),
• Average speed on on-ramps and freeways (Mean and standard deviation),
  (at upstream, downstream and merging section),
• Percentage of heavy vehicles on freeway and on-ramps, and
• Geometric characteristics of the selected section (grade, lane width, shoulder width etc.).

The parameters mentioned above will ensure that the model can represent the actual field conditions, fully calibrated and validated. For calibration and validation purposes, the parameters mentioned above were collected from the field.

3.4 Data collection methodology

From the criteria’s mentioned above, Cheyenne on-ramp on southbound I-15 is suitable for case study. The data was collected for the morning peak when the ramp volume was maximizing, so that merging volumes will be high. Based on the historical trends obtained from Nevada Department of Transportation (NDOT) ftp site (ftp://ftp.nevadadot.com), the time period between 10:00 a.m. to 11:15 a.m. was selected. The weekly count conducted by NDOT in 2006 is attached in Appendix 1.

3.4.1 Traffic counts

The traffic counts were collected for 1 hr 15 minutes during the peak period in 5-minute intervals for freeway and on-ramp. The traffic was categorized in the trucks, buses, passenger cars and motorcycles. The traffic count was measured for each lane on the freeway in southbound direction. For ramps, the traffic was not measured by the individual lanes. In order to determine the operating speeds of the freeways (upstream,
merging, downstream) and ramps (ramp speed, merging speed and downstream speed) floating car method is used.

3.4.2 Speed measurement

For the measurement of speeds in the upstream, downstream, on-ramp and merging section floating car method using GPS equipment was used. The GPS unit was set up to capture distance traveled, speed, longitude, latitude, and travel time after every 2 seconds. For the study, four cars were used to determine speed and travel time during the survey, the two cars made several runs from the on-ramp from Cheyenne S/B on-ramp to Lake Mead off ramp for ramp speeds and the other two cars used for freeway made several runs from Craig on-ramp to East Lake Mead on-ramp so that speed for upstream, merging and downstream speed of target ramp (Cheyenne on-ramp) can be measured. In this method, two vehicles collected the speed from on-ramps southbound to the adjacent off-ramp. The time headway between the cars on the ramps was based on the travel time so that the cars can capture the more accurate travel time for this study. For freeway speed distribution the two cars made several runs and each car was assigned each lane so that travel time and lane individual speeds can be estimated. The snapshot of the section of interest for this study is shown in Figure 3.
3.5 Guidelines in selecting the proper simulation model

Elefteriadou et al. (1999) presented a framework for selecting simulation models that are applicable to the problem at hand. In the 8-step guidelines, two important and crucial steps in the selection of a simulation model are:

1. First, the user becomes familiar with the strengths, weaknesses, and limitations of each simulation model, and

2. Second, the user knows how to properly interpret the output statistics.
The study team stated that an understanding of the models capabilities is “probably the most important aspect of selecting a simulation model.” The authors identified some of the key model features that can be used to evaluate a model. These include the size of network, network representation (urban streets, freeways etc.), traffic representation (microscopic or macroscopic), traffic composition, traffic operations, traffic control, and model output.

The authors additionally stated that the user should “know how to interpret the simulation results, draw any inferences from them, and determine whether they constitute a reasonable and valid representation of the traffic environment. For example, in some simulation models, stop time delay may be defined as the time during which a vehicle has a speed less than 5 ft/s, while in others, it may be defined as the time during which the vehicle is completely stopped. An understanding of how each output statistic is defined is important in the interpretation of the simulation results. The authors presented a series of steps to follow when selecting and applying a simulation model. The steps are:

1. **Project Scoping:** The first step is to identify the problem and the purpose of the study.

2. **HCM Assessment:** The next step is to consider the available Highway Capacity Manual procedures, and determine if any of them can be applied to the issues identified in project scoping. Limitations of the HCM procedures, with respect to the problem statement and issues from step 1 should be identified. If the limitations cannot be overcome with HCM procedures, simulation may be a viable alternative.
3. As the authors note, every simulation model has its strengths and weaknesses. It is important for the analyst to understand model limitations and deficiencies, relate the limitations to the needs of the project, and select the model that best satisfies the specified needs. Model capabilities, data requirements and availability, ease of use, staff expertise, technical support, and past model application and experience should all be taken into consideration.

4. Data Assembly.

5. Data Input.

6. Model Calibration and Validation.

7. Output Analysis.

8. Alternatives Analysis.

3.6 Model comparison studies

Skabardonis (1999) performed an evaluation and comparison of five simulation models. The evaluation was based on model capabilities and features, input requirements, output options, relationship with traditional planning and operational analysis tools, and modeling costs. The evaluated models were CORSIM, INTEGRATION, MITSIM, PARAMICS, and VISSIM. Based on the study's findings, the following recommendations were made regarding model selection, application, and technical support.

1. Corridor Improvement Strategies: INTEGRATION appears to be the best model for explicitly handling capacity improvements and HOV treatments.
2. Freeway Operations: CORSIM appears to be the leading model for the testing and evaluation of alternative geometric scenarios (weaving, merging, diverging), incidents and work-zone impacts, and ramp metering schemes. INTEGRATION can be used for network-wide ramp metering impacts, particularly for traffic diversions.

3. Arterial Operations: CORSIM appears to be the leading model for the evaluation of various intersection design alternatives, signal coordination schemes, and transit modeling along exclusive lanes or in mixed traffic.

4. INTEGRATION appears to be the leading model for the evaluation of ITS scenarios along corridors that involve real-time route guidance systems, or changes in traffic patterns due to ramp-metering strategies. CORSIM can be used in the assessment of traffic control strategies in which route-selection is fixed.

3.7 CORSIM Simulation Tool

From the previous research (Skabardonis, 1999), it was shown that CORSIM can handle the merging section scenarios effectively. Due to non availability of INTEGRATION software, for this study the simulation software CORSIM is used.

CORSIM (corridor simulation) is a microscopic simulation process that integrates both NETSIM (Network Simulation) and FRESIM (Freeway Simulation). CORSIM model can be used for a wide range of applications including modeling of weaving areas, merging areas, diverging, HOV lanes and capacity reducing effects such as lane changing and freeway merging. Because CORSIM simulates the traffic and traffic control conditions of a network over a period of time, the input must accommodate specifications
that not only differ from one point on the network to another, but that might also change with time. The network is modeled in such a way that each vehicle is considered as a separate entity. The behavior of each vehicle is represented in the model through interaction with its surrounding environment, which includes the freeway geometry and other vehicles. TSIS, which is the windows version of the integrated traffic software system, supports the execution of CORSIM and supports programs.

CORSIM contains a set of diagnostic tests for input, which are executed in the following sequence:

1. Test the structure of the input stream, and that all records are in proper sequence.
2. Test that each data item is valid and that its values lie within a range.
3. Test that all the data items on the set of records belonging to one record type are consistent and that the set is complete.
4. Test that the data items of all classifications are compatible and completely define the network.

TRAFVU is an interactive graphics processor designed to display and animates the results of CORSIM simulations. TRAFVU provides a window environment to view the input network and all the output generated by CORSIM. It enables us to animate traffic simultaneously in multiple views of the same or different traffic networks under the same or differing traffic conditions. TRAFVU is suitable for traffic operations analysis as well as the presentations of before and after studies to convince the audience of the utility of simulation results.
The various parameters that can potentially impact the results of the CORSIM are discussed below:

- **Car following sensitivity**: This value specifies the minimum headway between vehicles for different driver types. The lower the value of the sensitivity factor, shorter the spacing between the vehicles. The default values used in the FRESIM for car following sensitivity factors are mentioned in Table 1.

<table>
<thead>
<tr>
<th>Driver Type</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sensitivity Factor</td>
<td>1.25</td>
<td>1.15</td>
<td>1.05</td>
<td>.95</td>
<td>.85</td>
<td>.75</td>
<td>.65</td>
<td>.55</td>
<td>.45</td>
<td>.35</td>
</tr>
</tbody>
</table>

- **Lane–Change Maneuvering Time**: The default value in the CORSIM is 2 seconds (20 tenths of a second). The model assumes that during this time interval, a vehicle occupies both lanes (original and target lane). Lower the values of this parameters result in quick lane changing.

- **Lag acceleration time**: When the leading vehicle accelerates and the space headway between the leading and lagging vehicles increases. The time delays that lagging vehicle experiences when starting to accelerate is the lag acceleration time. This is the time elapsed between the when the leading vehicle accelerates and when lagging vehicle accelerates. It can influence the merging speeds on the freeway and capacity of the merging section.
• Gap acceptance parameter: This entry specifies the parameter for determining the acceptable gap for mandatory lane changes. The lowest number represents the most aggressive lane-changing behavior (small headway) and the highest represents the least aggressive lane-changing behavior (large headway) for all drivers.

• Percent yield values: The percentage of drivers desiring to yield the right-of-way to lane-changing vehicles attempting to merge ahead of them. The FRESIM model assumes that a certain fraction of putative followers in the target lane of a vehicle desiring to make a lane change will cooperate with the lane-changer to increase the probability of the lane change being successful.

• Desired Speed Distribution: The speed distribution of the vehicles on the freeway is known from the data collected. In order to reproduce the same field conditions in terms of distribution of speed (mean and standard deviation). It can be done by changing the speed distribution percentages in the CORSIM. The sum of percentage multipliers for all the 10 driver types must be equal to 1000. The default values used in the CORSIM model are mentioned in Table 2.

For example, when a link on CORSIM is assigned a free flow speed of 65 mph. Then it means 10% of the drivers (driver type 1) will be having a free flow speed $= 0.88 \times 65 = 57.2$ mph
### Table 2 Default values of free flow speed percentages

<table>
<thead>
<tr>
<th>Driver Type</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percentage multiplier of Free-flow speed</td>
<td>88</td>
<td>91</td>
<td>94</td>
<td>97</td>
<td>99</td>
<td>101</td>
<td>103</td>
<td>106</td>
<td>109</td>
<td>112</td>
</tr>
</tbody>
</table>

3.8 Car following model used in CORSIM

The FRESIM model utilizes the Pitt car-following behavior that was developed by the University of Pittsburgh (Halati et al., 1996). Car-following model consists of a car-following Equation together with a set of constraints which is appropriate for micro-simulation models. The Equations are established for the leader-follower relationship as it relates to the internal dynamics in simulation models. The CORSIM uses the Pitt car-following Equation. The basic car following model incorporates the distance headway and speed differential between the lead and the following model are two independent variables as shown in the Equation 18 below. Given the steady-state conditions are characterized by both lead and following vehicle, the third term of the car-following model tends to zero under steady-state driving ($\Delta u = 0$). The steady state model incorporated by the FRESIM can be described by Equation 19 in the congested conditions and a constraint of maximum speed on the roadway as described in the Equation 20 (uncongested conditions).

**General Equation**

$$h = h_f + c_3 u + b c_3 \Delta u^2$$  \hspace{1cm} (18)

**Steady state conditions**

$$h = h_f + c_3 u$$  \hspace{1cm} (19)

$$u = \min \left( \frac{h - h_f}{c_3}, u_f \right)$$  \hspace{1cm} (20)
where,

\[ h = \text{distance headway between the front bumper of lead vehicle and front bumper of the following vehicle}; \]

\[ h_f = \text{space headway between the vehicles when completely stopped}; \]

\[ c_3 = \text{driver sensitivity factor}; \]

\[ b = \text{calibration constant, which is equal to 0.1, if the speed of the following vehicle exceeds the speed of the lead vehicles otherwise it is set to zero}; \]

\[ \Delta u = \text{speed differential between the lead and following vehicle}; \text{ and} \]

\[ u_f = \text{roadway free speed}. \]

The Pitt car-following model is a separation constant \((c_3)\). The smaller the constant the closer together two vehicles are allowed. The FRESIM model utilizes 10 driver types, which are characterized by driver sensitivity factors and default values of CORSIM for each driver type which is mentioned in 3.7, to define the space headway in feet based on speed measurements in ft/s.

3.9 Evaluation of existing PCE values and development of new PCE values

The PCE values will be calculated for the capacity of merging section. In order to determine the capacity of the merging sections, flow will be increased on both freeway and ramps in a stepwise manner. The capacity can be determined by plotting the results of either speed vs. flow or flow vs. density. The diagram shown in Figure 4 is representing flow vs. density curves from which the capacity can be determined. Figure 4 is based on the incremental flow for the merging section based on which the capacity of the merging section will be determined.
PCE values will be determined at the capacity as mentioned above and then the results will be compared with the PCE values recommended by HCM (2000). The PCE values will be calculated based on the equivalent density concept approach.

Figure 4 Flow vs. density curve

Huber (1982) proposed the methodology of equivalent density, and it is used in this study to estimate PCE values. Huber assumed that a flow rate $q_B$ of a base stream (containing only cars) and a flow rate $q_M$ of a mixed stream, containing a proportion $p$ of trucks and a proportion $(1-p)$ of cars that shows the same density, can be equated as in Equation 21.

$$q_B = (1-p) \times q_M + e \times p \times q_M$$

(21)

where $q_M$, $q_B$, and $p$ are defined in Equation (7),

e = PCE of trucks.
The procedure for the determination of \( q_M \), \( q_B \) and \( e \) is described in the following four steps:

1. Establish the relationship between the (density) and flow rate for the base stream, containing passenger cars only,

2. Establish the relationship between the density and flow rate for the mixed stream, containing \((1-p)\) fraction of passenger cars and \(p\) fraction of trucks

3. Find equivalent flow rates \( q_M \) and \( q_B \) for the same density value as shown in Figure 5 and,

4. Calculate the passenger car equivalence factor \( e \).

![Figure 5 Equivalent density concept](image)
3.10 Factors potentially affecting the PCE values

In this study, different scenarios will be tested to evaluate their potential impact on the PCE values. Since the study is conducted only for level terrain, the following factors are considered to evaluate the impact on PCE values (under normal operating conditions of freeway):

- Traffic Flow on freeway and on-ramps;
- Percentage of trucks on freeway and on-ramps.

3.11 HCM methodology for freeway merging sections

The input data requirements for computation of LOS for on-ramp and mainlines junctions are geometric conditions, free flow speed at on-ramps and mainline, and demand. Then the flow is adjusted for peak hour factor, and driver population factor. After that the total flow on the freeway and ramps are converted into the passenger cars using heavy vehicle adjustment factors using the PCE factors developed for basic freeway segment. Determination of demand flow rate (in terms of pcppl only) in the two lanes adjacent to on-ramps are used in the model are discussed to calculate the capacity of the freeway and on-ramp junction. The determination of the flow in the lane 1 and 2 adjacent to the junctions is calculated using Equation 22. The capacities are computed for the total flow leaving merging area and maximum flow entering merging area. If the adjusted demand flow is less than the capacity, densities, LOS and speeds are computed.
The merge influence area is 1500 ft from the point of intersection of on-ramp and mainline to the downstream of mainline. If the there is a on-ramp or off-ramp exists with in 2500 ft, then that section is designed as weaving section.

\[ v_{12} = P_{FM} \cdot v_F \]  \hspace{1cm} (22)

where,

\( v_{12} \)  = flow rate in Lanes 1 and 2 of freeway immediately upstream of merge (pc/h),

\( P_{FM} \)  = proportion of approaching vehicles in lanes 1 and 2 immediately upstream of the merge junction, in decimal form,

\( v_F \)  = freeway demand flow rate immediately upstream of merge (pc/h).
Input data: geometric data, ramp free flow speed, freeway free flow speed, demand

Demand flow rate adjustment: peak hour factor, driver population factor, heavy vehicle factor

Compute flow rate in pcp/hpl

Compute demand flow rate upstream and downstream of merge influence area
Lane 1 and 2 of the mainline

Compute capacity
-Total flow leaving area
-Maximum flow merging area

Adjust demand < capacity

Yes
Compute density, LOS and speed

No
LOS F

Figure 6 HCM Methodology for merging section
(HCM, 2000)
CHAPTER 4

DESCRIPTION OF THE SIMULATION MODEL

4.1 Introduction

In this chapter, the description of the simulation model is discussed. The following sections are presented which will discuss the steps used to develop freeway merging section model.

- Description of the model,
- Field data,
- Model calibration procedure,
- Determination of sample size,
- Simulation output MOE's,
- Different simulation scenarios, and
- Statistical analysis.

4.2 Description of the Model

I-15 freeway Southbound at Cheyenne on-ramp is the selected case study location. The site has two lanes on-ramp that merge in to one just before the joining with mainline freeway merging area. The mainline freeway has two through lanes upstream and downstream of the merging section. Cheyenne on-ramp has a gradient of -2% and the gradient of the freeway are equal to zero. The parallel acceleration lane is about 2500 ft
from the junction of the merging section. The flow is divided in the ratios of 65:35 (shoulder lane to median lane) based on the field data collection results. The trucks are biased to the rightmost lane (shoulder lanes) which means high proportion of the trucks is traveling on shoulder lane based on the field observations. The default values of the passenger cars on CORSIM is equal to 16 ft, for single unit truck 35 ft, semi trailer with full load or with medium load is 53 ft, and for double bottom trailer is equal to 63ft. However, some of these values do not match with the values recommended by AASHTO 2001. Based on the recommendation of AASHTO 2001, the length of passenger car is increased to 19 ft, length of single unit truck is reduced to 30 ft, semi-trailer is increased to 55 ft and for double bottom truck the length is increased to 73 ft. the schematic diagram of the modeled on-ramp section is shown in Figure 7.

4.3 Field data

From the data collection at the case study, the volumes on freeway and ramp are presented in Table 3.
Table 3 Traffic Flow on freeway and ramps as measured in field

<table>
<thead>
<tr>
<th>Hourly flow (vph)</th>
<th>Peak hour factor</th>
<th>Percentage of trucks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway</td>
<td>2134</td>
<td>0.951</td>
</tr>
<tr>
<td>Ramps</td>
<td>1048</td>
<td>0.984</td>
</tr>
</tbody>
</table>

From the car floating method the following speeds, their mean and standard deviation are reported for the different section as described in Table 4.

Table 4 Speeds from the floating car method

<table>
<thead>
<tr>
<th>Section</th>
<th>Average Speed (mph)</th>
<th>Std. deviation (mph)</th>
<th>Number of samples</th>
<th>Confidence Interval 95% speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream</td>
<td>66.10</td>
<td>9.1</td>
<td>11</td>
<td>60.73 71.47</td>
</tr>
<tr>
<td>Merging</td>
<td>56.72</td>
<td>8.4</td>
<td>23</td>
<td>53.28 60.17</td>
</tr>
<tr>
<td>Downstream</td>
<td>54.70</td>
<td>9.4</td>
<td>23</td>
<td>50.84 58.56</td>
</tr>
<tr>
<td>Ramp</td>
<td>46.01</td>
<td>4.7</td>
<td>12</td>
<td>43.35 48.66</td>
</tr>
</tbody>
</table>

4.4 Model Calibration

The objective of the model calibration is to determine the combination of parameters values that can produce measure of effectiveness (MOE's) that are similar to those observed in the field conditions. The calibration process involves a series of multiple simulation runs by changing the model default values and then comparing the results with the field conditions. The following parameters are adjusted for the model calibration:

- Speed on Freeway (upstream area, merging area, and downstream area),

- Average speeds on-ramp, and
4.4.1 Model calibration procedure

Different parameters that can affect the measure of effectiveness (traffic flows and speed) in simulation are acceleration lag, deceleration lag, percentage yield, standard deviation of free flow speed distribution, and standard deviation of driver sensitivity factor. These parameters are varied systematically over a range to evaluate their impact.
on the speeds and flow. The calibration procedure is conducted by varying one parameter at a time over its entire range and keeping other parameters fixed to their default values. The measure of effectiveness is recorded for each of the parameter over their entire range. The range variation for each calibration parameters and measure of effectiveness recorded for each case is shown in Table 3 Appendix I. The regression analysis is done with 95% confidence interval to evaluate the parameters that can potentially affect the speed and flows. From the regression analysis results, the merging flow is not significantly affected by any of the parameters. From the regression analysis results, acceleration lag and free flow speed distribution shown to have impact on significant impact on the speeds. The relation that is used for the calibration procedure is shown below.

\[ \text{Speeds} = f(\text{acceleration lag, free flow speed distribution}) \]

Optimization: The optimization function is used to determine the values of the free flow speed distribution parameter and acceleration lag. The four Equations are formulated, one each for speeds at upstream, downstream, merging and ramp. The minimization is done using the observed values from the sites and the results obtained from the simulation using the key parameters identified. The constraints are defined for the range of key parameters based on the simulation results. The coefficient for acceleration lag and free flow speed distribution are determined for each Equation using the regression analysis. For the optimization, non negative values and non-linear model is assumed and with the help of Microsoft Excel optimization is performed. The optimization Equation and constraints Equation are shown in Equation 23 to 29.

\[
\begin{align*}
\text{Min.} & \left( (V_{\text{obs}} - V_{\text{sim}})_{\text{ramp}}^2 + (V_{\text{obs}} - V_{\text{sim}})_{\text{upstream}}^2 + (V_{\text{obs}} - V_{\text{sim}})_{\text{downstream}}^2 + (V_{\text{obs}} - V_{\text{sim}})_{\text{merging}}^2 \right) \\
\end{align*}
\]
Constraint Equations:

- For Acceleration Lag
  \[ 0.2 \leq \text{Acceleration lag} \leq 1.0 \]  
  \[ (24) \]

- For Free flow speed Distribution (FFSD)
  \[ 1.68 \leq \text{FFSD} \leq 9.29 \]  
  \[ (25) \]

- For Upstream
  \[ V_{\text{U/S}} + 2.01 \text{ (acc. Lag)} + 0.79 \text{ (FFSD)} = 66.10 \]  
  \[ (26) \]

- For Merging Section
  \[ V_{\text{merging}} + 0 \text{ (acc. Lag)} + 1.05 \text{ (FFSD)} = 56.72 \]  
  \[ (27) \]

- For Downstream
  \[ V_{\text{D/S}} + 0 \text{ (acc. Lag)} + 1.13 \text{ (FFSD)} = 54.70 \]  
  \[ (28) \]

- For On-ramps
  \[ V_{\text{on-ramp}} + 0 \text{ (acc. Lag)} + 0.55 \text{ (FFSD)} = 46.01 \]  
  \[ (29) \]

Table 5 Results from the optimization function

<table>
<thead>
<tr>
<th>Sections</th>
<th>U/S</th>
<th>Merging</th>
<th>D/S</th>
<th>Ramp</th>
<th>Acc. Lag</th>
<th>FFSD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed from GPS</td>
<td>66.10</td>
<td>56.72</td>
<td>54.70</td>
<td>46.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Speeds after Minimization function</td>
<td>61.79</td>
<td>55.81</td>
<td>56.85</td>
<td>46.92</td>
<td>0.20</td>
<td>7.29</td>
</tr>
</tbody>
</table>

1 Acceleration lag (Acc. Lag)

2 Free flow speed distribution

4.6 Determination of minimum sample size for simulation runs

The sample size required for tolerable error calculates the minimum number of simulation runs (i.e., the sample size) required to produce results with a "sampling error"
less than or equal to the tolerable error entered by the user. The formula for the sample size calculation is shown in Equation 30:

\[ n = \frac{Z^{2} \sigma^{2}}{E^{2}} \]  

where,

\( n \) = the minimum sample size,

\( Z \) score for 95% confidence interval = 1.96

\( \sigma^{2} \) = the sample variance computed from the simulation runs, and

\( E \) = the tolerable error for the mean.

From the initial simulation runs the standard deviation of 3.2 mph and tolerable error of 1 mph was observed. Based on the 95% confidence interval, the number of simulation runs was calculated using Equation 30.

Standard deviation, \( \sigma = 3.2 \) mph, \( E \) tolerable error = 1 mph

Number of simulation runs = 39.2 \( \approx 40 \).

4.7 Simulation output MOE's

MOE’s including speed and flow for on-ramp, merging, upstream and downstream section is collected from output processor. The output consists of the mean value, standard deviation and confidence interval for speed and flow for on-ramp, merging, downstream and upstream section. Since each simulation is run for one hour time period, the time intervals is set equal to 60 seconds and output processing is done after every 60 intervals means one hour. As CORSIM do not give the results for density, when the
multi-run feature is used. Therefore density is computed using the basic speed flow and
density relationship shown in Equation 31.

\[
k = \frac{q}{n \cdot v}
\]  \hspace{1cm} (31)

where,

- \(q\) = flow in vehicles/hour,
- \(n\) = number of lanes,
- \(v\) = velocity in miles/hour, and
- \(k\) = density in vehicles per mile per lane.

4.8 Different simulation scenarios

For the development of the PCE values, the impact of the different parameters is
reviewed. The different scenarios modeled for this study are mentioned below:

Case 1

- Truck percentage (0%, 5%, 10% 20% and 25%) on ramps and freeway both,
- Volume ratio (0.1, 0.2, and 0.3),
- Traffic flows

Number of simulation runs = 5 x 11 x 3 = 165

Case 2

- Trucks (0%, 5%, 10% 20% and 25% trucks) on ramps only,
- Volume ratio (0.1, 0.2, and 0.3),
- Traffic flows

Number of simulation runs = 5 x 11 x 3 = 165
Case 3

- Trucks (0%, 5%, 10%, 20% and 25% trucks) on freeway only.
- Volume ratio (0.1, 0.2, and 0.3),
- Traffic flows

Number of simulation runs = 5 x 11 x 3 = 165

Total number of simulation runs = 165 x 3 = 495

4.9 Statistical Analysis

The statistical test is conducted on the simulation results to see whether the results obtained from the simulation are normally distributed or not.

4.9.1 Kolmogorov-Smirnov test

In statistics, the Kolmogorov–Smirnov test (often called the K-S test) is used to determine whether two underlying one-dimensional probability distributions differ, or whether an underlying probability distribution differs from a hypothesized distribution, in either case based on finite samples. The measure of the difference between the empirical distributions functions and proposes model is based on the maximum observed distance between the two functions. In this study, this test is conducted to check whether the results obtained from the simulation software in normally distributed. The one-sample KS test compares the empirical distribution function with the cumulative distribution function specified by the null hypothesis. The main applications are testing goodness of fit with the normal and uniform distributions. The test will be used to test whether speed, density and flows are normally distributed or not. The Kolmogorov-Smirnov statistic is given by Equation 31.
\[ D_n = \max [F_n(x(i)) - F_{H0}(x(i))] \] (32)

where,

- \( F_n \) is the sample distribution function
- \( F_{H0} \) is the theoretical distribution function (normal distribution)

The test statistic is used to test the hypothesis

\[ H_0: \quad F_n(x) = F_{H0}(x) \]
\[ H_1: \quad F_n(x) \neq F_{H0}(x) \]

The sample calculation of Kolmogorov-Smirnov test for one data set (VR = 0.1, trucks = 0, and total flow = 5100 vph) to show that from the simulation results the data obtained is normally distributed. The value listed as asymptotic significance values of \( \leq 0.05 \) is considered good evidence that the data set is not normally distributed. In this study, the value of asymptotic significance is more that 0.675, so it can said that there is insufficient evidence to suggest that the data set is not normally distributed. The test was conducted using the SPSS software for a confidence interval of 95%. The results from the test are shown in Table 6.

**Table 6 One-Sample Kolmogorov-Smirnov Test**

<table>
<thead>
<tr>
<th></th>
<th>Speed</th>
<th>Flow</th>
<th>Density</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>N</strong></td>
<td>40</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td><strong>Normal Parameters</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>33.462</td>
<td>4449.22</td>
<td>44.38</td>
</tr>
<tr>
<td>Std. Deviation</td>
<td>1.449</td>
<td>40.618</td>
<td>1.535</td>
</tr>
<tr>
<td><strong>Most Extreme Differences</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Absolute</td>
<td>0.113</td>
<td>0.093</td>
<td>0.098</td>
</tr>
<tr>
<td>Positive</td>
<td>0.113</td>
<td>0.093</td>
<td>0.088</td>
</tr>
<tr>
<td>Negative</td>
<td>-0.091</td>
<td>-0.067</td>
<td>-0.098</td>
</tr>
<tr>
<td><strong>Kolmogorov-Smirnov Z</strong></td>
<td>0.722</td>
<td>0.595</td>
<td>0.628</td>
</tr>
<tr>
<td><strong>Asymptotic significance (2-tailed)</strong></td>
<td>0.675</td>
<td>0.871</td>
<td>0.825</td>
</tr>
</tbody>
</table>
4.9.2 Different Scenarios

For this study PCE values is computed for different scenarios:

- For different truck percentage (same volume ratio)
- For different volume ratios
- For different LOS
- For trucks on freeway only
- For trucks on ramp only
CHAPTER 5

RESULTS

5.1 Introduction

All the analyses conducted are for under-saturated conditions. The PCE values under the congestion scenario (LOS F) are not in the scope of this study. So the values reported in the study might not hold good for the congested conditions.

5.2 Review of the current HCM PCE values

The analysis done using HCS for merging section is done for different volume ratios and under different truck percentages shown in Table 7. The capacity results from the simulation are show in Table 7 for different volume ratios. The sample calculation from the Highway capacity software for case VR= 0.1 and truck percentage = 25% is attached in the Appendix I. The capacity of the merging section using simulation is calculated from the help of Figures 10 to 12.

- The capacity of the merging section is overestimated by the HCM when compared with the simulation results shown in Figure 9, Table 7 and Table 8. The overestimation of capacity can be attributed to the underestimation of the PCE values.
- HCM recommends the value of 1.5 for level terrain for different volume ratios, LOS and truck percentage.
### Table 7: Capacity of merging section

<table>
<thead>
<tr>
<th>% of trucks</th>
<th>Simulation capacity flow (vph)</th>
<th>HCM capacity (vph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VR = 0.1</td>
<td>VR = 0.2</td>
</tr>
<tr>
<td>0%</td>
<td>4430</td>
<td>4420</td>
</tr>
<tr>
<td>5%</td>
<td>4210</td>
<td>4190</td>
</tr>
<tr>
<td>10%</td>
<td>4040</td>
<td>4020</td>
</tr>
<tr>
<td>20%</td>
<td>3750</td>
<td>3640</td>
</tr>
<tr>
<td>25%</td>
<td>3580</td>
<td>3510</td>
</tr>
</tbody>
</table>

### Figure 9: Capacity of the merging section vs. percentage of trucks

Figure 9: Capacity of the merging section vs. percentage of trucks

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Figure 10 Flow vs. density curve for merging section with volume ratio 0.1

Figure 11 Flow vs. density curve for merging section with volume ratio 0.2
5.3 Effect of the volume ratios on the capacity of the merging sections

The capacity of the merging section for different volume ratio is calculated from the Figure 10 to 12. The volume ratios can decrease the capacity of the merging sections as shown in Table 8. The reduction in capacity is attributed to the slow moving vehicles merging into the mainline reduces the overall the capacity of the merging section.

5.4 Effect on PCE values merging section for different volume ratios

This section describes the effect of volume ratios on the PCE values for merging section. The sample calculation is shown below for the computation of PCE values from the figure of flow vs. density. The values of PCE calculated for merging section is shown in Table 9.
Sample calculation for PCE value for VR = 0.1 and Truck Percentage = 10%, at capacity using Figure 10.

\[ q_B = 4438 \text{ pcph}, \quad q_M = 4054 \text{ vph} \]

\[ q_B = 90\% (q_M) + 10\% (q_M) \text{ PCE} \quad \text{(from Equation 21)} \]

\[ PCE = \frac{4438 - 0.9 \times 4054}{0.1 \times 4054} = 1.95 \quad \text{(compared to HCM recommended = 1.5)} \]

- In merging section, for volume ratio of 0.1, there is a decrease in the PCE values as the percentage of trucks are increased. In volume ratio of 0.2, the similar pattern is observed except at truck percentage of 25. The volume ratio of 0.3, there is no clear trend, shows how the PCE values are affected with change in percentage of trucks.
- The volume ratio of 0.2 has the maximum PCE values for all the truck percentages except 20%.
- PCE values are highest in the case of 5%.

| Table 8 PCE values at capacity for merging section |
|---------------------|---------------------|---------------------|
| VR = 0.1 | VR = 0.2 | VR = 0.3 |
| 5% | 2.08 | 2.10 | 2.02 |
| 10% | 1.95 | 1.99 | 1.96 |
| 20% | 1.93 | 1.98 | 2.00 |
| 25% | 1.92 | 2.01 | 1.99 |

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5.5 PCE values for upstream and downstream of merging section

The PCE values are computed for the upstream and downstream section adjacent to the merging section, are shown in Table 9. The flow vs. density curves for upstream and downstream section is shown in figure 11 to 16.

- For volume ratio equal to 0.1, the PCE values decreases as the percentage of trucks increases. But 0.3, the PCE value increases with the percentage of trucks. There is no clear trend of PCE values for volume ratio of 0.2 with the change in percentage of trucks.
- For upstream section, there is no clear trend which shows that PCE value are maximizes at given volume ratio for different truck percentages.
- For upstream section, for truck percentage up to 10%, as the volume ratio increases the PCE value decreases. For truck percentage equals to 25, as the volume ratio increases the PCE value increases.
- For downstream section the PCE values is highest at volume ratio of 0.2 except for truck percentage equal to 10.
- For downstream section, there is no clear trend for PCE values with increase in the truck percentage

Table 9 PCE values for upstream and downstream section at capacity

<table>
<thead>
<tr>
<th>Percentage of trucks</th>
<th>Upstream</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VR = 0.1</td>
<td>VR = 0.2</td>
</tr>
<tr>
<td>5%</td>
<td>2.11</td>
<td>2.11</td>
</tr>
<tr>
<td>10%</td>
<td>2.01</td>
<td>1.99</td>
</tr>
<tr>
<td>20%</td>
<td>1.94</td>
<td>1.99</td>
</tr>
<tr>
<td>25%</td>
<td>1.95</td>
<td>2.00</td>
</tr>
</tbody>
</table>

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Figure 13 Flow vs. density curve for upstream section with volume ratio 0.1

Figure 14 Flow vs. density curve for upstream section with volume ratio 0.2
Flow vs density upstream section (VR = 0.3)

![Graph]

Figure 15 Flow vs. density curve for upstream section with volume ratio 0.3

Flow vs density downstream section (VR = 0.1)

![Graph]

Figure 16 Flow vs. density curve for downstream section with volume ratio 0.1
Flow vs. density downstream section (VR = 0.2)

Figure 17 Flow vs. density curve for downstream section with volume ratio 0.2

Flow vs. density downstream section (VR = 0.3)

Figure 18 Flow vs. density curve for downstream section with volume ratio 0.3
5.6 PCE values for different LOS for merging section

This section discusses the PCE values based on the different level of service as shown in Table 10. The Figures from 9 to 11 are used to calculate PCE values for the different LOS. The Figure 19 shows the equivalent density for different level of service and using Equation 20, the PCE values are calculated.

![Flow vs. Density curve for VR = 0.2 for different LOS]

- The PCE values are equal to 1 for LOS A, for different volume ratios and different truck percentage.
- PCE values are function of LOS, increases with the increase in LOS for volume ratio of 0.1. But there is no clear trend for volume ratio of 0.2 and 0.3.
• PCE values are highest at truck percentage of 5% at capacity for any given volume ratio.

• PCE values for LOS B and volume ratio of 0.1, with the increase in truck percentage the PCE value increases. But for volume ratio of 0.2 and 0.3 no clear trend is present. The PCE values are highest for volume ratio of 0.1 for LOS B when compared with other volume ratio for the same percentage of trucks.

• For LOS C, D and at capacity, there is no clear trend of variation of PCE values with respect to volume ratios and truck percentage.

Table 10 PCE values for different LOS for merging

<table>
<thead>
<tr>
<th>LOS</th>
<th>% of trucks</th>
<th>VR = 0.1</th>
<th>VR = 0.2</th>
<th>VR = 0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>B</td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.03</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.32</td>
<td>1.20</td>
<td>1.18</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.40</td>
<td>1.16</td>
<td>1.11</td>
</tr>
<tr>
<td>C</td>
<td>5%</td>
<td>1.87</td>
<td>2.09</td>
<td>2.09</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.72</td>
<td>2.13</td>
<td>1.98</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.81</td>
<td>1.84</td>
<td>1.87</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.83</td>
<td>1.86</td>
<td>1.86</td>
</tr>
<tr>
<td>D</td>
<td>5%</td>
<td>2.00</td>
<td>2.03</td>
<td>1.91</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.95</td>
<td>1.88</td>
<td>1.82</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.90</td>
<td>1.91</td>
<td>1.93</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.90</td>
<td>1.94</td>
<td>1.93</td>
</tr>
<tr>
<td>Capacity</td>
<td>5%</td>
<td>2.08</td>
<td>2.10</td>
<td>2.02</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.95</td>
<td>1.99</td>
<td>1.96</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.93</td>
<td>1.98</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.92</td>
<td>2.01</td>
<td>1.99</td>
</tr>
</tbody>
</table>
Table 11 PCE values for different volume ratios merging section

<table>
<thead>
<tr>
<th>VR</th>
<th>% of trucks</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>CAPACITY</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.87</td>
<td>2.00</td>
<td>2.08</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.00</td>
<td>1.03</td>
<td>1.72</td>
<td>1.95</td>
<td>1.95</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.00</td>
<td>1.32</td>
<td>1.81</td>
<td>1.90</td>
<td>1.93</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.00</td>
<td>1.40</td>
<td>1.83</td>
<td>1.90</td>
<td>1.92</td>
</tr>
<tr>
<td></td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>2.09</td>
<td>2.03</td>
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<td>1.86</td>
<td>1.94</td>
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</tr>
<tr>
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<td>1.00</td>
<td>2.09</td>
<td>1.91</td>
<td>2.02</td>
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<tr>
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<td>1.98</td>
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<td>1.93</td>
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</tr>
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<td>25%</td>
<td>1.00</td>
<td>1.11</td>
<td>1.86</td>
<td>1.93</td>
<td>1.99</td>
</tr>
</tbody>
</table>

5.7 PCE values for different LOS for upstream and downstream

To calculate the PCE values based on LOS, Figures 13 to 18 are used. The PCE values are reported for the different level of service for upstream and downstream section adjacent to merging section in Table 12 to Table 14. The different LOS for basic freeway segment are shown in Figure 20 for upstream VR = 0.2 and truck percentage 10.

For upstream section

- The PCE value is equal to 1 for LOS A and B for different volume ratios and different truck percentages except for VR = 0.3 and truck percentage = 25.
- For volume ratio of 0.2, increase in the PCE value with increase in LOS for same truck percentage.
- For LOS C, PCE value is increasing for volume ratio 0.1 and 0.3 as the truck percentage increases. For VR of 0.2, there is no clear trend of PCE values when the percentage of trucks increases.
• For LOS C, the PCE values are highest for VR of 0.3 for any given percentage of trucks.
• For LOS D, the PCE values increases in all the volume ratios when the truck percentage is increases.
• For LOS D, there is no clear trend for which VR the PCE values are highest.
• For LOS E, there is no clear trend for change in the PCE values with different volume ratio and percentage of trucks.

Figure 20 Flow vs. Density for upstream section VR = 0.2

For Downstream section

• The PCE value is equal to 1 for LOS A for different volume ratios and different truck percentages except for VR = 0.3 and truck percentage = 25.
• For LOS B, PCE value increases as the truck percentage increases for volume ratio 0.2 and 0.3. But there is no clear trend for PCE values for VR of 0.1 with the change in percentage of trucks.

• For LOS C, PCE value is increasing for volume ratio 0.1 and 0.3 as the truck percentage increases. For VR of 0.2 there is no clear trend of PCE values when the percentage of trucks increases.

• For LOS C, the PCE values are highest for VR of 0.2 for any given percentage of trucks.

• For LOS D, there is no clear trend for PCE values, as the volume ratio increase and truck percentage increases.

• For LOS D, the PCE values are highest for VR of 0.2 for any given percentage of trucks.

• For LOS E, there is no clear trend for change in the PCE values with different volume ratio and percentage of trucks.
Table 12 PCE values for different volume ratios upstream section

<table>
<thead>
<tr>
<th>VR</th>
<th>% of trucks</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>5%</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.59</td>
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<td>1.00</td>
<td>1.00</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.03</td>
<td>1.74</td>
<td>1.94</td>
</tr>
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<td>25%</td>
<td>1.00</td>
<td>1.00</td>
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<td>1.80</td>
<td>1.95</td>
</tr>
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<td>0.2</td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.92</td>
<td>2.11</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.30</td>
<td>1.93</td>
<td>1.99</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.19</td>
<td>1.94</td>
<td>1.99</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.35</td>
<td>1.94</td>
<td>2.00</td>
</tr>
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<td>5%</td>
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<td>1.00</td>
<td>1.59</td>
<td>1.89</td>
<td>1.95</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.62</td>
<td>1.94</td>
<td>1.96</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.80</td>
<td>1.97</td>
<td>1.98</td>
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<tr>
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<td>25%</td>
<td>1.03</td>
<td>1.09</td>
<td>1.87</td>
<td>2.02</td>
<td>2.03</td>
</tr>
</tbody>
</table>

Table 13 PCE value for different LOS for upstream

<table>
<thead>
<tr>
<th>LOS</th>
<th>% of Trucks</th>
<th>VR = 0.1</th>
<th>VR = 0.2</th>
<th>VR = 0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.00</td>
<td>1.00</td>
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</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.03</td>
</tr>
<tr>
<td>B</td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.00</td>
<td>1.00</td>
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<td></td>
<td>20%</td>
<td>1.00</td>
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<td>1.00</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.09</td>
</tr>
<tr>
<td>C</td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.59</td>
</tr>
<tr>
<td></td>
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<td>1.00</td>
<td>1.30</td>
<td>1.62</td>
</tr>
<tr>
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<td>25%</td>
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<td>2.11</td>
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<td>1.99</td>
<td>1.96</td>
</tr>
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<td>20%</td>
<td>1.94</td>
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</tr>
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<td></td>
<td>25%</td>
<td>1.95</td>
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<td>2.03</td>
</tr>
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</table>
Table 14 PCE values for different volume ratios downstream section

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<th>VR</th>
<th>% of trucks</th>
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<th>B</th>
<th>C</th>
<th>D</th>
<th>CAPACITY</th>
</tr>
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<td>1.99</td>
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<td>20%</td>
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<td>1.02</td>
<td>1.08</td>
<td>1.57</td>
<td>1.91</td>
</tr>
<tr>
<td></td>
<td>25%</td>
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<td>1.04</td>
<td>1.14</td>
<td>1.55</td>
<td>1.94</td>
</tr>
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<td>1.00</td>
<td>1.41</td>
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</tr>
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<td>10%</td>
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<td>1.00</td>
<td>1.13</td>
<td>1.72</td>
<td>1.99</td>
</tr>
<tr>
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<td>20%</td>
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<td>1.02</td>
<td>1.10</td>
<td>1.59</td>
<td>2.00</td>
</tr>
<tr>
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<td>25%</td>
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<td>1.36</td>
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<td>1.00</td>
<td>1.61</td>
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</tr>
<tr>
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<td>20%</td>
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<td>1.02</td>
<td>1.10</td>
<td>1.57</td>
<td>2.00</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.03</td>
<td>1.08</td>
<td>1.11</td>
<td>1.57</td>
<td>2.03</td>
</tr>
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Table 15 PCE values for different LOS for downstream section

<table>
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<tr>
<th>LOS</th>
<th>% of Trucks</th>
<th>VR = 0.1</th>
<th>VR = 0.2</th>
<th>VR = 0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
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<td>10%</td>
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<tr>
<td></td>
<td>25%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.03</td>
</tr>
<tr>
<td>B</td>
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<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>10%</td>
<td>1.05</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>20%</td>
<td>1.02</td>
<td>1.02</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>25%</td>
<td>1.04</td>
<td>1.04</td>
<td>1.08</td>
</tr>
<tr>
<td>C</td>
<td>5%</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
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<td></td>
<td>25%</td>
<td>1.14</td>
<td>1.41</td>
<td>1.11</td>
</tr>
<tr>
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<td>5%</td>
<td>1.41</td>
<td>1.41</td>
<td>1.36</td>
</tr>
<tr>
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<td>10%</td>
<td>1.61</td>
<td>1.72</td>
<td>1.61</td>
</tr>
<tr>
<td></td>
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<td>1.55</td>
<td>1.64</td>
<td>1.57</td>
</tr>
</tbody>
</table>

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Table 15 PCE values for different LOS for downstream section (contd.)

<table>
<thead>
<tr>
<th>Capacity</th>
<th>5%</th>
<th>10%</th>
<th>20%</th>
<th>25%</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.06</td>
<td>1.99</td>
<td>1.91</td>
<td>1.94</td>
</tr>
<tr>
<td></td>
<td>2.15</td>
<td>1.99</td>
<td>2.00</td>
<td>2.05</td>
</tr>
<tr>
<td></td>
<td>2.04</td>
<td>2.04</td>
<td>2.00</td>
<td>2.03</td>
</tr>
</tbody>
</table>

5.8 PCE values for trucks coming only from on-ramp in merging section

The results mentioned in Table 16 are for the case when at particular volume ratio all the trucks that are coming from on-ramp have that much percent of trucks. So, as the volume ratio increases the actual truck percentage in total flow increases.

- The low values for PCE values are attributed to the fact that in this case, heavy vehicles are coming from the on-ramp only, the total volume of truck is small.
- There is no clear trend for PCE values with the change in volume ratio and truck percentage.

Table 16 PCE value at capacity for merging section (trucks on-ramp only)

<table>
<thead>
<tr>
<th>% of trucks</th>
<th>VR = 0.1</th>
<th>VR = 0.2</th>
<th>VR = 0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>1.10</td>
<td>1.04</td>
<td>1.14</td>
</tr>
<tr>
<td>10%</td>
<td>1.09</td>
<td>1.05</td>
<td>1.12</td>
</tr>
<tr>
<td>20%</td>
<td>1.03</td>
<td>1.05</td>
<td>1.14</td>
</tr>
<tr>
<td>25%</td>
<td>1.03</td>
<td>1.04</td>
<td>1.15</td>
</tr>
</tbody>
</table>

5.9 PCE values for trucks coming only from freeway in merging conditions

The results mentioned in Table 17are for the case when at particular volume ratio all the traffic that is coming for freeway has that much percent of trucks. So, as the volume ratio increases, the actual truck percentage in total flow decreases. The lower volume
ratio means more number of trucks is coming from freeway section for a given percentage of trucks.

- The heavy vehicles coming from freeway suffer turbulence even from the passenger car coming from the on-ramps which explains the fact that the PCE values are very high. The interaction between the car-truck, truck-truck and truck-car are not easy to comprehend.

- There is no clear trend for PCE values with the change in volume ratio and truck percentage.

**Table 17 PCE value at capacity for merging section (trucks on freeway only)**

<table>
<thead>
<tr>
<th>Percentage of Trucks</th>
<th>VR = 0.1</th>
<th>VR = 0.2</th>
<th>VR = 0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>2.24</td>
<td>2.18</td>
<td>2.09</td>
</tr>
<tr>
<td>10%</td>
<td>1.99</td>
<td>2.05</td>
<td>2.12</td>
</tr>
<tr>
<td>20%</td>
<td>1.93</td>
<td>2.02</td>
<td>2.08</td>
</tr>
<tr>
<td>25%</td>
<td>1.96</td>
<td>2.04</td>
<td>2.10</td>
</tr>
</tbody>
</table>

5.10 Summary of the results

The results of this study have shown that the unlike the HCM recommendations:

- Capacity of the merging section decreases as the volume ratio increases.
- PCE values for merging section varies with different volume ratios and different truck percentage.
- PCE values for merging section varies with truck percentage.
- PCE values for merging section varies with different LOS.
- PCE values for upstream and downstream section varies with different volume ratios.
- PCE values for upstream and downstream section varies with different truck percentage.
- PCE values for upstream and downstream section varies with different LOS.
CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1. Effect of flows rates on PCE values:

Results obtained for this study are similar to the results of study conducted by Elefteriadou (1999). Elefteriadou reported that the PCE values for basic freeway segment for level segments were equal to 1.0 for flow up to 1500 vphpl for truck percentages up to 15%. When the flow was increased to the 2000 vphpl, the PCE value increased to 1.5 for 5% trucks and for 15 to 25% trucks the PCE value was 2.0. In our study, the results showed the PCE values vary with the LOS for downstream and upstream section a low as 1.0 for low flow rates (LOS A) and as high as 2.15 at capacity.

2. PCE values in merging sections:

The study showed that the PCE values are different for the merging section as compared to the basic freeway segment. The results are consistent with results of the study conducted by Vermijis (1998) which showed the PCE values for weaving sections can vary from 2.5 to 3.6 based on the lane configuration for weaving sections. The results from that study showed that the PCE values are different from the values recommended by HCM (2000). In this study also for
weaving sections the PCE values varies with the LOS, volume ratios and percentage of trucks.

3. PCE values for upstream and downstream section adjacent to merging sections:
   From this study the upstream sections adjacent to the merging sections doesn't behave as basic freeway segment the capacities are influenced by merging operations. The PCE value for upstream and downstream section can be as low as 1.0 for LOS A and as high as 2.15 for LOS E. The HCM recommended value is 1.5 for truck percentage from 2 to 25%.

4. Effect of volume ratio on capacities:
   The study showed that the capacity of the merging sections is affected by the volume ratios. High volume ratios lead to the lower capacity of the section.

5. The HCM recommends the use of same PCE value for different volume ratios, different percentage of trucks and different LOS. This study showed that PCE values for level section vary with the LOS, percentage of truck and volume ratio.

6. Effect of PCE values when heavy vehicles are on ramp only:
   The PCE values for trucks that are only coming from the on-ramp vary from 1.03 to 1.15, based on volume ratio and truck percentage. PCE values are lower when compared to the scenario when trucks are coming only from freeway and from the scenario when trucks are present both on freeway and ramp.

7. Effect on PCE values when heavy vehicles are on freeway only:
   The PCE values for trucks that are only coming from the freeway vary from 1.93 to 2.24, based on volume ratio and truck percentage. PCE values are higher when
compared to the scenario when trucks are coming only from on-ramp and from
the scenario when trucks are present both on freeway and ramp.

6.2 Recommendations

- One of the main drawbacks of this study is that the model is not validated; the
  validated model can help in a more legitimate estimation of PCE values
determination. However, this study showed that the PCE values for merging
section area is different from basic freeway segment. The similar results are
expected after the model validation.

- For future studies, impact of the acceleration lane length on the capacity of
  merging sections and potential impact on PCE values.

- The effect of upgrades and downgrades on the PCE values and capacities of the
  merging sections.

- The number of lanes may affect the PCE values; the more lanes will allow
  vehicles to travel in outer lanes away from merging lanes which may change the
  PCE values.
CHAPTER 7

LIMITATIONS OF THE SOFTWARE AND STUDY

7.1 Limitations of software and study

- One of the main drawbacks of this study is that the model was not validated. Therefore, although the trends and relationships of the PCE values with respect to various parameters appear to be reasonable, the actual computed PCE values may no be accurate.

- The PCE values developed in this study are for under saturated conditions and the results may not hold good for over saturated conditions.
APPENDIX I

DATA COLLECTION

Table 1: Data collection results from freeway

<table>
<thead>
<tr>
<th>Start time</th>
<th>Finish time</th>
<th>Heavy vehicles in lane 1 and 2</th>
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<th>Motor Bikes</th>
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<th>Shoulder lane</th>
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Table 2: Data collection results from on-ramp

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<th>Motorbikes</th>
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Table 3 Simulation Results from the varying each parameter

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<th>Acceleration lag</th>
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Table 3 Simulation Results from the varying each parameter (contd.)

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<th>Downstream Speed</th>
<th>Ramp Speed</th>
<th>Acceleration lag</th>
<th>FFS</th>
<th>DSS</th>
<th>Deceleration lag</th>
<th>% yield</th>
<th>Lane change</th>
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Sample Calculation: HCM merging section capacity analysis for 25%

### Freeway Data

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<td>Free-flow speed on freeway</td>
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<tr>
<td>Volume on freeway</td>
<td>3680 vph</td>
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### On Ramp Data

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<th>Side of freeway</th>
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<td>Number of lanes in ramp</td>
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<tr>
<td>Free-flow speed on ramp</td>
<td>50.0 mph</td>
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<td>Volume on ramp</td>
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<td>Length of first accel/decel lane</td>
<td>2500 ft</td>
</tr>
<tr>
<td>Length of second accel/decel lane</td>
<td>ft</td>
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### Adjacent Ramp Data (if one exists)

| Does adjacent ramp exist? | No |
| Volume on adjacent Ramp | vph |
| Position of adjacent Ramp | ft |
| Type of adjacent Ramp | ft |

### Conversion to pc/h Under Base Conditions

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<th>Freeway</th>
<th>Ramp</th>
<th>Adjacent Ramp</th>
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<td>Recreational vehicles %</td>
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### Estimation of V12 Merge Areas

\[ L = \text{Equation } 25-2 \text{ or } 25-3 \]

\[ P = 1.000 \text{ Using Equation 0} \]
\[ V = V(P_{12}) = 4140 \text{ pc/h} \]

### Capacity Checks

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<td>4599</td>
<td>4700</td>
<td>No</td>
</tr>
<tr>
<td>v</td>
<td>4599</td>
<td>4600</td>
<td>No</td>
</tr>
</tbody>
</table>

#### Level of Service Determination (if not F)

Density, \( D = 5.475 + 0.00734 v + 0.0078 v^2 - 0.00627 L = 25.5 \text{ pc/mi/ln} \)

Level of service for ramp-freeway junction areas of influence C

#### Speed Estimation

Intermediate speed variable, \( M = 0.459 \)

Space mean speed in ramp influence area, \( S = 54.5 \text{ mph} \)

Space mean speed in outer lanes, \( S = N/A \text{ mph} \)

Space mean speed for all vehicles, \( S = 54.5 \text{ mph} \)
| Site Names: | 030390...IR-15 | County: | Clark |
| Location: | S/B on ramp of the Cheyenne Av Inch Exit 46 | Sector Class; | Urban Principal Arterial - Interstate |

Table 4: Traffic count by NDOT on case study location on-ramp location

**Nevada Department of Transportation**

**Daily Volume from 01/31/2006 through 02/07/2006**

<table>
<thead>
<tr>
<th>Road</th>
<th>N</th>
<th>S</th>
<th>Road</th>
<th>N</th>
<th>S</th>
<th>Road</th>
<th>N</th>
<th>S</th>
<th>Road</th>
<th>N</th>
<th>S</th>
<th>Road</th>
<th>N</th>
<th>S</th>
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</thead>
<tbody>
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<td>370</td>
<td>15:00</td>
<td>390</td>
<td>390</td>
<td>19:00</td>
<td>884</td>
<td>304</td>
<td>23:00</td>
<td>370</td>
<td>370</td>
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<tr>
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<td>370</td>
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</table>

| Seasonal Factor Type: | 05 | Daily Factor Type: | 05 |
| Axle Factor Type: | | Growth Factor Type: | |

| Table 4: Traffic count by NDOT on case study location on-ramp location |

| Seasonal Factor Type: | 05 | Daily Factor Type: | 05 |
| Axle Factor Type: | | Growth Factor Type: | |

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<th>02/08/2006</th>
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<td>188</td>
<td>358</td>
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# Nevada Department of Transportation

## Daily Volume from 01/31/2006 through 02/07/2006

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<th>02/03/2006</th>
<th>02/04/2006</th>
<th>02/05/2006</th>
<th>02/06/2006</th>
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<table>
<thead>
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<tr>
<td>23:00</td>
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## Volume

- Minimum: 10,500
- Maximum: 18,500
- Range: 23,004 - 23,004
- Average: 16,705

## Summary

- **Annual Peak Vol**: 1,031
- **Monthly Peak Vol**: 1,021
- **Weekly Peak Vol**: 1,021
- **Daily Peak Vol**: 1,021
- **Seasonal Peak Vol**: 1,021
- **Daily Veh**: 1,021
- **Axle Veh**: 1,021
- **Pulse Veh**: 1,021

**Data Information**

- Date: 03/17/2006
- Time: 7:47:33 AM
- Road: AADT 22,982
- NEG AADT 32,982
- POS AADT 0
- DVD: Page 2 of 2
### Nevada Department of Transportation

#### Daily Volume from 01/30/2006 through 02/05/2006

**Site Name:** 030387, 6021056, IR-15  
**County:** Clark  
**Function Class:** Urban Principal Arterial - Interstate  
**Location:** 6 mi N of the Cheyenne Av Interch Exit 46

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<th>Date</th>
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<th>AM Peak Hr</th>
<th>PM Peak Vel</th>
<th>PM Peak Hr</th>
<th>Seasonal Fct</th>
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</table>

**Volume:**

- **AM Peak Vel:** 432
- **PM Peak Vel:** 2342
- **Seasonal Fct:** 1.00

**Volume Calculation:**

\[ \text{Volume} = \text{AM Peak Vel} \times \text{PM Peak Vel} \times \text{Seasonal Fct} \]

**Example Calculation:**

\[ \text{Volume} = 432 \times 2342 \times 1.00 = 1,009,428 \text{ vehicles/day} \]
Nevada Department of Transportation
Daily Volume from 01/30/2006 through 02/05/2006

Site Names: 030387. 602603 . IR-15
County: Clark
Function Class: Urban Principal Arterial - Interstate
Location: 6 mi S of the Cheyenne Av Interch Exit 46

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<th>02/03/2006</th>
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<td>07:00</td>
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Counted: 03/17/2006 7:47:12AM
ROAD AADT 78,100
NEG AADT 87,692
POS AADT 40,402
DV03: Page 2 of 2
APPENDIX II

SIMULATION RESULTS

VR = 0.1 and upstream section

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<thead>
<tr>
<th>0% trucks</th>
<th>Discharge (vph) Mean</th>
<th>95% C.I.</th>
<th>Speed (mph) Mean</th>
<th>95% C.I.</th>
<th>Density (vpmpl) Mean</th>
<th>95% C.I.</th>
</tr>
</thead>
<tbody>
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<td>449.96</td>
<td>63.90</td>
<td>63.79</td>
<td>64.00</td>
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<td>900.64</td>
<td>63.32</td>
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<td>63.40</td>
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<th>Speed (mph) Mean</th>
<th>95% C.I.</th>
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<td>Speed (mph)</td>
<td>Density (vpmpl)</td>
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<tr>
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### 5% trucks

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VR = 0.3 upstream section

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### 10% trucks

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VR = 0.3 Merging Section

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REFERENCES


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Amanpreet Singh Ahuja

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NIT Kurukshetra, Haryana, India

Honors:
Second prize in zone 2, big beam competition 2005.

Thesis title:
Development of Passenger Car Equivalents for Freeway Merging Sections

Thesis Examination Committee:
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Committee Member, Dr. Hualiang (Harry) Teng, Ph.D.
Committee Member, Dr. Edward S Neumann, Ph.D.
Graduate College Representative, Dr. Ashok K. Singh, Ph.D.