Effect of On-Ramp Demand on Capacity at Merge Bottleneck Locations

By

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Effect of On-Ramp Demand on Capacity at Merge Bottleneck Locations

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ABSTRACT

Past research acknowledged the impact of ramp vehicles on the occurrence of a breakdown event, but little has been done to quantify the effect of the ramp vehicles on the resulting bottleneck capacity. The objective of this research is to explore the relationship between ramp flow and capacity and to recommend capacity values for merge bottleneck locations.

To explore the relationship between ramp flow and capacity, a capacity model has been developed using linear regression. The entire freeway network in Kansas City area was considered for the analysis. All locations which experienced “true breakdown”, breakdown because of merging operations and not due to downstream spillback, were selected for the analysis. Detector data at the six selected locations, were downloaded from KC Scout Portal from January 1, 2016 to June 30, 2016. Per lane and average speed, volume and occupancy data at 5-minute intervals were chosen for the analysis so as to detect breakdowns and find independent breakdown locations. Incident data and bad weather data were also collected for the same period and all the days with incidents and bad weather were removed from the analysis.

For this research, free flow speed was defined as the average of flows when speeds were more than 50 miles per hour and the flows were less than 800 vehicles/hour. Breakdown was said to occur when speeds drop more than 25% of the free flow speed and the reduced speeds are maintained for at least 15 minutes, i.e., three 5-minute intervals (TRB, 2016). The breakdown capacities ranged from 3,900 to 8,500 vehicles per hour (veh/h) and when averaged across all the lanes, 1,150 to 2,200 vehicles per hour per lane (veh/h/ln). The upstream breakdown flows (demand) ranged from 3,400 to 8,400 veh/h and when averaged across all the lanes, 1,050-2,100 veh/h/ln. The ramp breakdown flows (ramp demand) ranged from 150 to 2,700 veh/h and when averaged across all the lanes, 150 to 1,500 veh/h/ln.

Various variables such as freeway demand, ramp demand, free flow speed, number of lanes, ramp to freeway demand ratio, outer two-lane flow, shoulder lane flow, and remaining lane flow were considered for developing the model. Interactions between the variables were also considered. The final model was developed using 70% of the data, which were randomly selected, and the remaining 30% of the data was set aside for validation. A final model with an R^2 value of 0.689 was developed. The high R^2 value indicates that the developed model is a good predictor of capacity and this was also proven through the validation test for the developed model.

The regression model was used to predict capacity values for different ramp demand and freeway demand. By observing the calculated capacity values it was concluded that the capacity was decreasing as the ramp demand, outer two lanes flows were increasing and the capacity per lane was decreasing as the number of lanes was decreasing, which is consistent with past literature (Lu & Elefeteriadou, 2013; Kondyli et al. 2016).

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DEDICATION

I would like to dedicate this thesis to my parents, Satya Gowri Sujatha Gubbala & Ramanadha Reddy Gubbala, who have provided me with everything I ever needed. Thank you for all the sacrifices you have made to help me pursue my dream and supporting me at every step of this wonderful journey. I will always be indebted to you for being the wonderful people you are. I will always love you with all my heart.

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CHAPTER 1. INTRODUCTION

1.1 Overview

Several researchers have investigated the breakdown of flow at freeway bottlenecks as these events determine the capacity of the bottlenecks. Typical physical bottlenecks are merge junctions, weaving segments, lane reduction segments, and tunnels. All the research that has been done on this topic suggested that the probability of breakdown increases as the traffic volume increases at a bottleneck (Elefteriadou et al. 1995, Persaud et al. 1998).

Several researchers have observed that the probability of breakdown (i.e., the transition from uncongested to congested flow) increases with increasing on-ramp flow. Elefteriadou et al. (1995) developed a probabilistic model which gives the probability that a breakdown will occur at a given ramp and with a given freeway flow and is based on ramp-vehicle cluster occurrence. According to Kerner and Rehborn (1996, 1997), when on-ramp vehicles "squeeze" on to the highway or due to unexpected speed decrease and lane changing activities, a decrease in local speed and an increase in density was observed which was said to cause breakdown at a bottleneck. Yi and Mulinazzi (2007) found the ramp vehicles influence on freeway vehicles was related to the presence of persistent vehicle platoons on the on-ramps and defined it as "invasive influence". Sun et al. (2014) observed that due to increases in mainline and on-ramp flow there was an increase in lane changes which lead to forced lane changes (FLCs) that trigger congestion in the acceleration and shoulder lanes at the downstream end of the merging bottleneck.

1.2 Objective

As concluded from the literature, past research acknowledges the impact of ramp vehicles on the occurrence of a breakdown event, but little has been done to quantify the effect of the ramp vehicles on the resulting bottleneck capacity. The objective of this research is to explore the relation between ramp flow and capacity. This was done by identifying various merge bottlenecks and gathering detector data at the selected locations. The next step was to identify breakdown occurrence and obtain the capacity (i.e., the breakdown flow) and the ramp flow, at the time of breakdown. Models of capacity as a function of upstream freeway flow, ramp flow, number of lanes, ramp/flow ratio, and other variables that appeared to be significant were developed.

1.3 Organization of the document

This document is divided into six chapters. Chapter 2 presents the previous research related to capacity, the breakdown phenomenon, and the contribution of on-ramp demand on capacity. Chapter 3 presents the methodology used for this research and details the process of developing the capacity regression model. Chapter 4 presents the data collection and data filtering process. Chapter 5 provides a detailed description on the model development process and also presents the final regression model that has been developed as a part of this research and also gives the recommended capacity values based on the developed model. Chapter 6 presents the conclusion of this research and also provides recommendations for future research.

CHAPTER 2 - LITERATURE REVIEW

This chapter presents literature review findings of various researchers related to the merge capacity and breakdown definitions, the stochastic nature of breakdown and capacity, and the contribution of on-ramp flow to breakdowns at merge bottlenecks.

2.1 Definition of Capacity

Greenshields (1935) carried out test, used photographic measurement methods for the first time to measure traffic flow, traffic density, and speed. From the data gathered, he observed a linear relationship between speed and traffic density, as show in Figure 1.

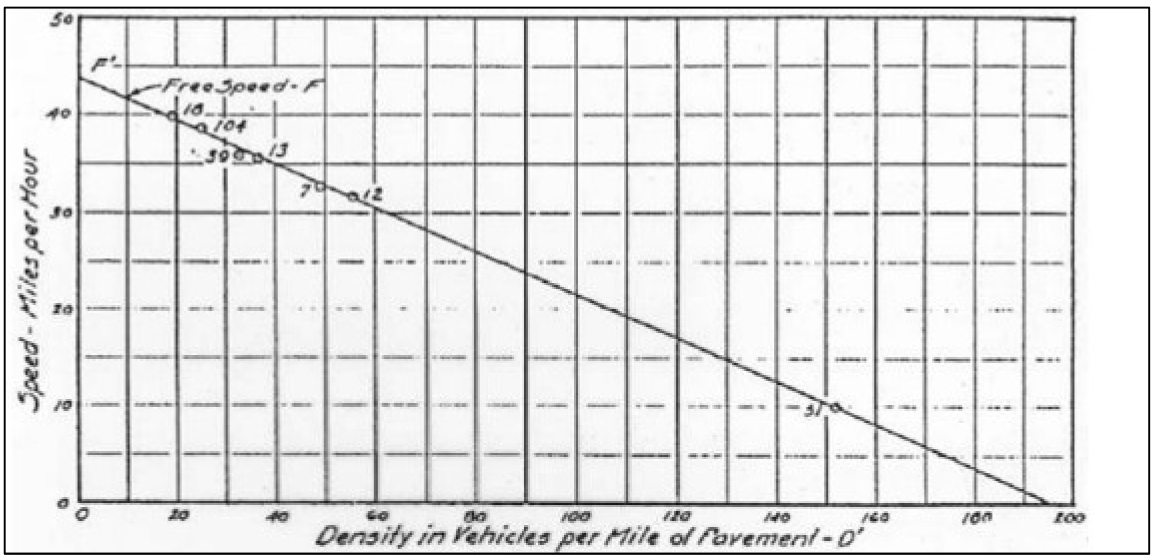

Figure 1: Speed Density Relation (Greenshield 1935)

He assumed linear relationship between speed and density, and used the fundamental equation flow = density × speed, to express the relationship between speed and flow and between flow and density in a parabolic form. Greenshields used the term "Density-vehicles per hour", as the term flow was not known at that time. This model shows a maximum traffic flow happens at the optimal traffic density. In speed (miles per hour) and density (vehicles per hour) graph (Figure 2) two regimes can be observed; that means for the same traffic flow, two different speeds are possible.

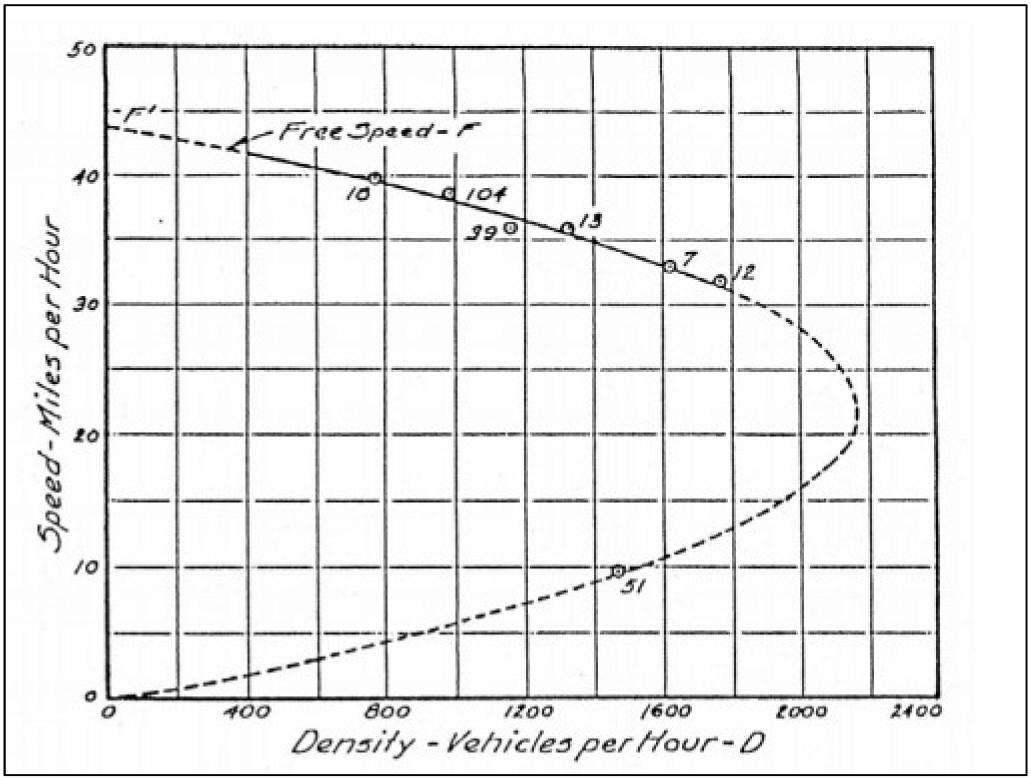

Figure 2: Speed Flow Relation (Greenshield 1935)

Hall and Agyemang-Duah (1991) recommended numerical values for capacity by investigating the capacity drop issue when the queue forms, by collecting data over 52 days at peak periods. They observed that there is a capacity drop in the bottleneck and it is the only legitimate place to look for capacity drop. The distributions of pre-queue flows and queue discharge flows in 15-minute intervals were presented, with the pre-queue flows more skewed towards higher flows. Passenger cars per hour per lane (pcphpl) is equivalent to vehicles per hour per lane (veh/h/ln) after converting the truck volumes into equivalent passenger car units, using a factor. Two capacity values, each for pre-breakdown condition (pre-queue flows) and one for post-breakdown conditions (queue discharge flows) were recommended as 2,300 pcphpl and 2,200 pcphpl, respectively. The recommended capacity values were determined by observing the mean of the distribution and converting it into passenger car equivalent and averaging over all lanes. However, they question whether the mean of the distribution is the proper value to be considered as capacity.

Persaud et al. (1998) explored capacity drop and further looked into, how metering might be able to mitigate the bottlenecks. They observed four parameters for defining capacity; flow at breakdown (Q_b) and corresponding time (T_b), and the mean queue discharge flow (Q_d) and the corresponding time (T_d). T_b and Q_b were determined when there was a sharp drop of speed at a detector which is close to the entrance and also within the bottleneck. The mean flow after the transition, from uncongested to congested conditions, until speeds recovered back to uncongested levels was determined as $Q_{d,}$ and the time where Q_d was continually exceeded by the pre-queue flow rates was determined as $T_{d,}$ through inspection.

Lorenz and Elefteriadou (2001) defined capacity as the flow rate prior to the breakdown. Data were aggregated to 1, 2, 5 and 15 minute intervals, which were originally obtained in 20 sec time intervals. Breakdown flows were recorded for each for the time intervals for selected sites and the results showed that at both sites, breakdowns occurred at different capacities all day. On some days multiple breakdowns occurred at one site, with different capacity values during each breakdown. At a particular site, breakdown would occur at a certain flow but on a different day the same flow or even higher flow would not cause a breakdown. A new definition for capacity has been suggested for inclusion into Highway Capacity Manual, "the rate of flow (expressed in pcphpl and specified for a particular time interval) along a uniform freeway segment corresponding to the expected probability of breakdown deemed acceptable under prevailing traffic and roadway conditions in a specified direction."

Elefteriadou and Lertworwanich (2002) considered three capacity definitions and compared them to each other. The definitions were:

i. Breakdown flow – 5 or 15 min flow occurring before the breakdown,

ii. Maximum pre breakdown flow – maximum 5 or 15 min flow before the breakdown,

iii. Maximum discharge flow – maximum 5 or 15 min flow during the breakdown.

Chi Square and Kolmogorov-Smirnov (K-S) tests were used to compare the distributions to the normal distribution after frequency histograms were created for all three definitions for both intervals. From the tests they found that the normal distribution was a good fit at a 5% level of significance for all definitions. To compare the mean values at each site, T-tests were performed and for almost every analysis, the breakdown flow was less than the maximum discharge flow and the maximum pre-breakdown flow. Another result of the tests was that, at one site the maximum pre-breakdown flow was less than maximum discharge flow and at the other site it was the opposite. This behavior was attributed to the geometric characteristics at the two sites. Finally, they concluded that breakdown flow should be used for future capacity analysis, reasons given as the following: i) It is a conservative value as it was the lowest value in almost every analysis, ii) for the other two definitions to be used a breakdown needs to occur, iii) it complies with the capacity definition at the boundary between congested and uncongested conditions, and iv) it can be associated with the probability of breakdown.

Polus and Pollatschek (2002) investigated the definition of freeway capacity. They presented three capacity estimation methods. By observing the data in a plane of any two of the three primary parameters of flow (e.g., flow and speed), a best-fit parabolic curve to the entire data was fitted, at both stable and unstable flows. The extreme point of this parabola was used for estimating the capacity. The authors also discussed that this method would underestimate the capacity as many observations of higher flows will be outside this curve. In the second method, they fitted a parabola to the entire free flow, but not through dense flow, and another parabola was fitted to the entire unstable flow portion. A rational unbiased estimation of the maximum reasonable flow while excluding some points was said to be obtained at the intersection points of the two curves. In the third method linear regression lines were fitted, one for dense and one for unstable flow, using only relevant pairs of flow and speed near capacity. The intersection point of these linear regression models was said to be an improved estimation method as it was based only on the relevant points in the vicinity of maximum capacity. The authors defined this as Momentary capacity. Pairs of flow-speed data obtained for three sections, for three lanes on each section for three days and each combination of section lane and day were analyzed separately and they found that shifted gamma distribution was a good fit for all the cases at a 5% level of significance.

Zhang and Levinson (2004) calculated the average percentage of flow drop at each bottleneck, on the basis of the queue discharge flow (QDF) and the pre-queue flow (PQF), and found that there was a 2% to 11% drop of average flow at all study sites. The correlation between duration of QDF and PDF was examined and results showed that heavily congested bottlenecks have shorter pre-queue transition periods. The daily average and 30-sec QDF were examined at each bottleneck and it was found that 30-sec QDFs have very high variability. They also observed long-run average QDFs and PQFs and found that they are normally distributed but the difference in mean was about 100 passenger cars per hour per lane (pc/h/ln) and they concluded that the capacity of the freeway segment, under any demand conditions should be a weighted sum of two flows, as show in Eq. 1:

$$\mathbf{C = QDF^{*}\theta + PQF^{*}(1 - \theta)\ (0 \le \theta \le 1)} \qquad \textbf{Eq. 1}$$

Brilon (2005) examined the capacity distribution by testing several known distributions. A non-parametric approach called the Product Limit Method (PLM) was applied first and this was used to obtain the distribution function for capacity, $F_c(q)$. The data were collected in 5 min increments, continuously for 3 years. Various distributions such as Gamma, Weibull, and normal distributions were investigated to fit the traffic breakdown volumes and they found that the

Weibull distribution produced the most appropriate fit for all freeway sections observed. Finally, the author concluded that capacity, defined as the volume as traffic transitions from free flowing to congested conditions, is Weibull-distributed and suggested to use the 50th percentile of the Weibull Function as a nominal capacity value.

Cassidy and Rudjanakanoknad (2005) explored the effects on capacity drop at an isolated merge. Capacity was defined as the substantial flow a freeway discharges from all exits, when merge ramps are queued, and the off-ramps are not blocked. The authors used the following equation to obtain cumulative vehicle counts vs. time curves.

$$\boldsymbol{O(t) = V(t) - q_o \times (t - t_o)} \qquad \textbf{Eq. 2}$$

Where,

$O(t) = oblique\ coordinates$

$V(t) = virtual\ vehicle\ count$

$q_o = specified\ date$

$t = time$

$t_o = start\ time$

Elefteriadou et al. (2006) discussed the breakdown occurrence at merge or lane drop bottlenecks. The authors presented six definitions of breakdown: the maximum 5- or 15- minute pre-breakdown flow, the 5- or 15- min values prior to breakdown and finally, the sustained pre-queue and queue discharge flows. The authors did not decide on a single definition for capacity, but it was agreed that the maximum pre-breakdown flow is the closest to the definition provided by the Highway Capacity Manual (HCM) (TRB, 2000), but this is difficult to obtain. They also stated that the queue discharge flow is preferred over pre-queue flow during congested conditions.

Chung et. al., (2007) considered three different breakdown type locations and investigated the capacity drop mechanism. They utilized the same equation used by Cassidy and Rujanakanoknad (2005) presented as Equation 2 above, and using the O-curves, the capacities of the three locations were found. The flow change was observed by drawing a trend line to the curve and the change was identified when the trend line changes the slope of the O-curve. Capacity drop was calculated as the difference in capacity before and after a breakdown begins, which was visually inspected from the O-curves.

Jia et al. (2010) while determining the best distribution for pre-breakdown flow, developed a new methodology for breakdown identification. For consistency, data were first converted into passenger-car equivalent flows using the default value from HCM with 5% heavy vehicles. The outlying pre-breakdown flow rates were removed using the formulas shown in Eq.3 and Eq.4 as it was important to exclude them under nonrecurring conditions:

$$\boldsymbol{q < Q_{0.25} - 1.5 \times IQR\ ,\ Q_{0.25}\ being\ the\ 25th\ percentile\ flow\ rate\ (pcphpl)} \qquad \textbf{Eq. 3}$$

$$\boldsymbol{q > Q_{0.75} + 1.5 \times IQR\ ,\ Q_{0.75}\ being\ the\ 75th\ percentile\ flow\ rate\ (pcphpl)} \qquad \textbf{Eq. 4}$$

$$IQR = Q_{0.75} - Q_{0.25}$$

$IQR\!: Inter\ Quartile\ Range$

Kolmogorov-Smirnov test for goodness of fit was applied to the data, after converting remaining data into average pre-breakdown headways, and they found that shifted lognormal distribution was the best fit for the average pre-breakdown headway probability distribution.

Oh and Yeo (2012) estimated capacity drop by observing 16 on-ramp merge bottlenecks. Capacity drop was estimated as the difference between Capacity (Q_c) and discharge flow (Q_d). Capacity was defined as “maximum number of vehicles that persist for 5 min in a free-flow state.” Capacity was determined to be sustained over 5 min interval, rather than 15-min, after reviewing several other studies. Maximum 5-min flows were used as capacity. Discharge flow was defined at the flow at which, $SD \times V_u + SD \times V_d$, was minimum during the breakdown.

$SD = Standard\ Deviation$

$V_u = Upstream\ speeds$

$V_d = Downstream\ speeds$

Kühne and Lüdtke (2012) proposed a revised definition of capacity by using statistical methods. The new definition states that "the capacity is the traffic volume which corresponds to a given probability (e.g. 15%) for a breakdown within a given observation time (e.g. 1 min)."

2.2 Definition of Breakdown

Kerner (2000), showed that the congested regime can exist under a large variation of discharge flow rates downstream of the bottleneck based on empirical study of congestion occurring at a freeway bottleneck. The discharge flow may vary between 1600 to 2700 veh/h/lane. Kerner defined three types of traffic phases: free flow, wide moving jams, and synchronized flow. It was found that a traffic pattern often consist of spatial-temporal alterations among free flow, synchronized flow and wide jams. Effective location of a bottleneck was said as a point on the highway where the spontaneous free flow to synchronized flow transition takes place with highest probability. Flow rates that can be approximately considered as controlling parameters for causing a congestion are the flow upstream of the bottleneck on the main highway, where vehicles leave the highway by the off ramp and the flow rate of the vehicles on the on-ramp. It was concluded that traffic congestion (synchronized flow), under large variation of the discharge flow rate, is self-maintained at a freeway bottleneck. The change in the traffic demand upstream of the bottleneck during the existence of the congestion and nonlinear processes inside the pinch region in synchronized flow upstream can be reasons for the variation of discharge flow rate from a congested bottleneck. The author also claims that the discharge flow rate cannot be predefined or given or predicted in reality and must be found together with other variables of congested flow upstream of the bottleneck and that is why queuing theories cannot be applied for a correct study of the dynamics of traffic patterns at bottlenecks on un-signalized highways.

Evans et al. (2001) applied Markov chains to develop the probability distribution of the time of breakdown. The area around the merge was divided into zones with approximate length of 175 feet, representative of typical two second time headway between sequential vehicles traveling at 60 miles per hour. Three zones are defined starting at the nose of the gore point leading to two zones in merge lanes and three zones in the through lanes and a total of eight zone lane pairs, as shown in the figure below.

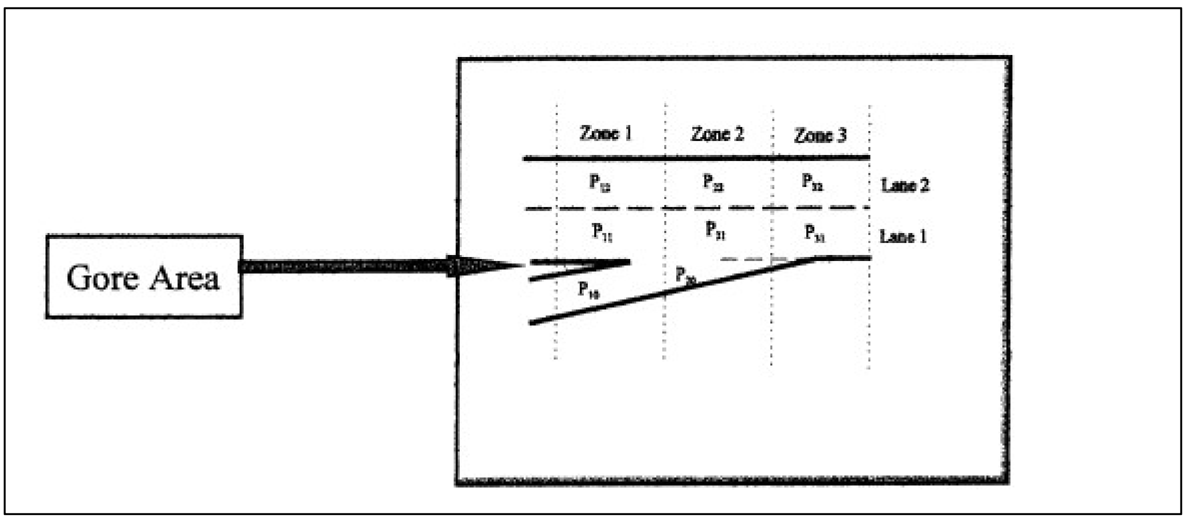

Figure 3: Zonal probabilities (Evans et al., 2001)

The probability distribution of the time of breakdown was determined based on the zonal merging probabilities with respect to vehicles travelling on the freeway. Freeway flows, available gaps, and drivers' action as they approach the merge area were considered in developing the model. Breakdown was defined as "any time there is a merging vehicle present, and one of the following occurs for a minimum time span of 6 seconds (the time to transition three states total or two states after the initial states);

i. Vehicles are present in lane 1 and vehicles continually arrive in that lane by moving forward in lane 1 and/or merging from lane 2,

ii. Vehicles are present in lane 1 and vehicles are braking in lane 1.

The potential breakdown states are called “bad states: and other states are called “good states.”

Lorenz and Elefteriadou (2001) conducted a study along the Highway 401 in Toronto, Ontario, Canada at two on ramp locations and explored breakdown occurrence and freeway capacity differences. Vehicle speed data, recorded in 20 second intervals, were collected for both sites and speed vs. time series plots were prepared. If the average speed dropped below 90 km/h (55.9 mph), across all lanes, for at least 5 min then that event is considered as breakdown. Conversely, if the average speed across all lanes was more than 90 km/h for at least 5 min, then the facility was considered to have recovered.

Polus and Pollatschek (2002) divided data into three regimes; free flow, dense flow, and unstable flow after plotting scatterplots. The traffic breakdown was defined as the change of state from dense flow to unstable flow. The threshold between dense and unstable flows was 80 km/h. Elefteriadou and Lertworawanich (2003) evaluated and compared several widely used capacity definitions. They obtained data from two on-ramp merges, which regularly experience breakdown. Breakdown was defined as occurring when speeds dropped below 90 kmph for a period of 15 min. They also pointed out that different freeways have different thresholds and the geometry and free flow of that freeway will affect these thresholds.

Zhang and Levinson (2004) identified some properties of traffic flows at active freeway bottlenecks. Flows during pre-queue transitions and queue discharge periods were measured and studied. Two occupancy threshold values were used for diagnosis in this study. If in an interval of

30 seconds, the minimum occupancy reading was larger than 25% from all the detectors, then the station was said to be in congestion region. If the occupancy reading was smaller than 20%, then it was considered as uncongested. The intermediated period was said to exist if neither of the conditions were satisfied.

Brilon (2005) defined breakdown as occurring when speed dropped below 70 km/h (43.5 mph) and the drop was at least 10km/h to avoid intervals of recovery within large congestion periods.

Jia et al. (2010) considered various freeway segments and each individual segment's critical speed and density values were found for determining breakdown. Breakdown was defined to occur when speeds on the freeway segment drop below the critical value of 55 mph and at the same time its density was greater than equal to the boundary between C and D level of service (LOS). The downstream sensors data were eliminated if the same breakdown features were observed.

Kühne and Lüdtke (2012) defined breakdown based on queue length, denoted by n_{crit}, though they recognize other characteristics associated with breakdown such as pre breakdown and queue discharge flow and defined speed drops. They also stated that this value was arbitrarily similar to speed drops, which classifies breakdown. The theory was performed with and without traffic controls and the values of n_{crit} were found to be good fit for the data.

Oh and Yeo (2012) considered several on-ramp locations each location with different number of lanes, and estimated the capacity drop values. They also explained that four states of traffic can exist on a freeway; free flow, transition to bottleneck, bottleneck, and recovery period. The free flow conditions are said to exist when both upstream (V_u) and downstream (V_d) speeds are greater than 50 mph. The transitions and recovery period both were said to have same characteristics which were, V_u exits between 40 to 50 mph and the V_d is greater than or equal to 50 mph. Finally, the breakdown conditions were said to occur when the following equations are satisfied;

$V_u \leq 40$ mph **Eq. 5**

$V_d \geq 50$ mph **Eq. 6**

(SD V_u + SD V_d) ≤ 5 mph for 15 min, SD = standard deviation **Eq. 7**

Shawky and Nakamura (2012) investigated the breakdown phenomenon at on-ramp merging sections of urban expressways in Japan. The breakdown event was defined as a reduction in speed at the bottleneck detector below the critical value of 55 km/h, and this condition must be sustained for at least 15 min while the downstream speeds were over this critical value. They treated breakdowns as a probabilistic phenomenon. Three different aspects of merging capacity; the maximum flow rate before the breakdown, the flow rate when the breakdown starts to occur, and the flow rate during the queue discharge period were discussed. They investigated the probabilities of breakdown as a function of flow rates and observed that the breakdown probabilities followed the Weibull distribution function at all sections. They used occupancy measurements as an indicator of the breakdown phenomenon instead of traffic flowed, rated as occupancy observations, cover the effect of all vehicles classes, in regard to vehicle length and they do not have to adjust the raw occupancy observations unlike flow rates.

2.3 Stochasticity of Capacity and Breakdown

Elefteriadou et al. (1995) studied the probabilistic nature of breakdown at freeway merge junctions. To study this characteristics, the sites were selected such that the section downstream of the merge is free of constraints, the merge area must regularly experience breakdown mainly because of ramp volume and not because of geometric design deficiencies, and all the sites selected should have similar geometry and general environment. Breakdown was defined to occur when there was a speed reduction of 16 km/h for at least one vehicle on the mainline. The probability of breakdown was computed as a function of the cluster size, which is dependent on ramp flow rate and the freeway flow. The data they observed demonstrated that breakdown does not occur when the capacity is reached and the high flows do not result in breakdown. They finally concluded that the breakdown is a probabilistic event and not deterministic. It was also observed that the on-ramp vehicle cluster size plays an important role in breakdown but large cluster always does not guarantee breakdown; instead, the probability of breakdown increases.

Evans et al. (2001) determined the arrival distribution of the merging vehicles and the probability of transitioning from state to state, the probability distribution of the time of breakdown occurring was determined before any time, t, by applying Markov chains. Higher probabilities of breakdown were expected because of the larger arrival rates of the vehicles traveling on the freeway main line, with respect to application of Markov chains and the results of the model also predicted a higher probability of breakdown for higher arrival rates.

Persaud et al. (2001) conducted research to better understand the breakdown phenomenon. Available data of 20 seconds and 30 seconds were aggregated into one minute counts to estimate the probability of breakdown curve. Time-speed and time-flow diagrams were plotted for each day and breakdown was identified as the point at which a flow was followed by a sharp drop in speed and flow. They observed this this point was not necessarily at the highest observed flows and could occur at any flow values, and in all the observed cases, drop in speed and flow continued until the end of the peak period.

Polus and Pollatschek (2002) found the stochastic nature of the capacity by observing the variabilities of flow-speed pairs. The fluctuations in momentary capacity were mainly observed during the peak period and in the approximate vicinity of the maximum flow.

Brilon (2005) understood capacity as “traffic volume below which traffic still flows and above which the flow breaks down into stop-and-go or even standing traffic.” And by no means is a constant value. Rather than using the traditional concept of single value capacities, the concept of stochastic capacities was said to be more realistic and more useful. An improved understanding of traffic flow observations and typical dynamics in different traffic states on a freeway was provided by this probabilistic approach. But one drawback was the enormous amount of data needed to get the capacity distribution function.

Chow et al. (2009) analyzed the probability of breakdown by using bivariate Weibull distribution. The probability of breakdown was considered as a function of the combination of mean speed and occupancy of the approaching traffic. They plotted contours of breakdown probability with respect to speed and occupancy. From this plots it was observed that the probability of breakdown increased with an increase in speed and occupancy. It was also found from the speeds and occupancies leading to breakdown that these are highly negatively correlated to each other, i.e. traffic stream traveling at high (low) speed will be associated with a low (high) occupancy.

2.4 On-Ramp Flow Contribution to Breakdown

Elefteriadou et al. (1995) prepared a probabilistic model and described the process of breakdown at ramp-freeway junction. According to this model, “the probability of breakdown will occur at given ramp and freeway flow and is based on ramp-vehicle cluster occurrence.” They examined video tapes placed near on-ramps and saw that queues were created on the ramp or on the freeway due to the large cluster of vehicles entering the freeway from the ramp. As the number of the vehicle platoons entering the freeway from the ramp increased the impact also was said to be increased. They also observed the shift of freeway traffic to the left lane to avoid the turbulence and conflicts in the merge area.

Kerner and Rehborn (1996, 1997) defined the breakdown phenomenon as the transition from free-flow to synchronized flow. The abrupt changes in the average speed were observed to detect the transition from the free-flow to synchronized flow (average speeds are almost synchronized in different lanes), using the data from German highways. When on-ramp vehicles "squeeze" on the highway or due to unexpected speed decrease and lane changing activity the decrease in local speed and increase in density was observed which was said to cause a breakdown in a bottleneck.

Daganzo et al. (1999) presented a model describing traffic behavior. When a vehicle in the platoon decelerated to grow a gap in order for a vehicle to merge in front of it, "deceleration disturbance" occurred, which caused the following vehicles to decelerate as well. Further instabilities and perturbations resulted due to the entire platoon returning back to its original speed which causes a wave to travel within the platoon and propagate upstream. This led to higher densities and to development of jam states. When a vehicle accelerates and closes the gap in front of it, it is called an "acceleration disturbance" and if these are persistent the queue disturbances could propagate forward and reduce the flow through the bottleneck.

Daganzo (2002) modeled the freeway breakdown event at freeway merging segments, assuming two types of drivers, fast- and slow-moving, as follows: While on-ramp vehicles enter and stay in the passing (left) lane, fast moving vehicles stay in left lane, willing to accept shorter headways. The fast-moving vehicles that had entered the freeway from the on-ramp merge into the passing lane, leaving the shoulder lane, thus increasing the passing-lane flow, which is defined as "pumping mechanism" as the drivers are willing to accept reduced headways and let the on-ramp vehicles merge. Fast moving vehicles follow each other in small headways which is a suggestion that they are "motivated" by their desire to pass, in highway and uncongested flow conditions. Because of the merging fast-moving vehicles, the passing lane becomes saturated downstream of the merge and a shockwave will move further upstream. This happens when the through and/or merging flow is high. This causes a speed decrease in passing lanes near the merge, causing the fast-moving vehicles to lose their "motivation" to follow closely and change lanes to equalize speeds leading the queues on the passing lanes eventually spill over to shoulder lane. Banks et al. (2003) conducted an evaluation study of Daganzo's theoretical model and concluded that some phenomena described in Daganzo's theory do occur, but not at all locations, and that the underlying assumption is oversimplified. Only at one site, increase in time gaps (loss of motivation) was verified but contrary to Daganzo's mode; the speed equalization does not take place at all locations. As predicated by Daganzo, distinction between capacity and discharge flow were not observed downstream from the queues, and even though the speeds were not equalized, they did observe redistribution of flows among lanes.

Yi and Mulinazzi (2007) found that ramp vehicles influence on freeway vehicles was related to the presence of persistent vehicle platoons on the on-ramps and defined it as "invasive-influence". They found that the number of evasive events increased and the standard deviation decreased with the merging platoon size. The brake light indication and lane change maneuvers were observed to account for the evasive behavior of the freeway vehicles that travel on the shoulder lane, for developing the model. Three merge situations were also defined as the following: (i) Free Merge (FM): random arrival of ramp vehicle that does not interact with the freeway vehicle. (ii) Challenged Merge (CM): ramp vehicle conflicts with freeway vehicles on the shoulder lane before merging and, (iii) Platoon Merge (PM): cluster of ramp vehicles force their way ignoring the priority order, and trigger invasive-influences to the freeway vehicles.

By considering the distribution of traffic on the shoulder lane (less traffic on the shoulder lane indicates high invasive influence) and the persistent platoons which cause the speed decrease in shoulder lanes the significance of invasive influence on the shoulder lane was estimated. An alternative LOS indicator was proposed that corresponds to these relationships between invasive-influence with volume shift and speed reduction.

Shawky and Nakamura (2012) studied the factors affecting the breakdown probability and found that the probability of breakdown increases with increasing acceleration lane length at the same value of outflow, a right on-ramp section (the traffic system in Japan is left-hand and the right lanes carry more traffic), the higher on-ramp flow ratio (relative percentage of on-ramp flow rate to outflow rates) at the same outflow rate, and the significant increase in interval length of the data at same outflow rate. Suggesting that they are “motivated” by the desire to pass, the fast moving vehicles follow each other in small headways, in high and uncongested flow.

Kondyli et al. (2013) developed probabilistic models to predict breakdown of flow by using data from five freeway-ramp merging segments. The breakdown prediction model was developed for the critical ramps, i.e., the ramps where congestion starts due to merging operations. Depending on the type of data available three types of models were developed. The first type provides the probability of breakdown as a function of ramp demand and the upstream freeway demand, considered individually. This type of model was said to consider the effect of the combination of different levels of ramp demand and freeway demand on the breakdown probability. The probability of breakdown was provided as a function of the sum of the ramp and freeway demands upstream of the critical ramp junction by the second probability models, and the third type provided the probability of breakdown as a function of the occupancy upstream of the critical ramp.

Sun et al. (2014) studied the mechanism of early-onset of breakdowns at on-ramp bottlenecks on expressways in Shanghai, China. Key parameters like pre-queue flow, queue discharge flow, speed variation per minute, lane change (LC) times in the mainline lanes and the acceleration lanes, LC types and LC locations (longitudinal and lateral). It was observed that during the early breakdown, most lane changes were forced LCs that occurred near the downstream end of the bottleneck, which spread laterally rather quickly. The authors examined the traffic flow process at two merge bottlenecks, with macroscopic and microscopic analysis, to determine the causes for early-onset congestion. One minute speeds were also estimated which were used to detect breakdown time and location as well as its evolution in space and over time. The breakdown process was summarized as congestion occurring at the merging point first, migrating laterally (at the breakdown point lanes) and then longitudinally soon afterwards.

LCs were categorized as normal free LC (NLC), cooperative (CLC), or forced LC (FLC). The LC type distribution vary spatially and temporally and to capture the distribution difference and its effects an indicator called degree of disruption (DOD) was used. The higher the DOD, the greater the disruptions lane changes cause. DOD = a*CLC+b*FLC, where CLC and FLC are the number of LC events per unit time, respectively, and a and b are the weight coefficients corresponding to the two LC types. It was concluded that due to increase in mainline and on-ramp flow there was an increase in LCs which lead to FLCs which trigger congestion in the acceleration and shoulder lanes at the downstream end of the merging bottleneck. Due to secondary lane changes congestion spreads to other lanes and then to upstream section of the bottleneck. Because of this intense LC activities congestion might occur even before the expected capacity is reached.

2.5 Summary of Literature Review

From the above literature review it can be seen that various researchers have investigated the capacity and breakdown at merge junctions and provided different definitions, but no consensus has been reached on the definitions of breakdown and capacity. They have also concluded that the occurrence of the breakdown and the resulting capacity is stochastic in nature. Research has also investigated how the breakdown is triggered at merge junctions and various researchers showed that the ramp flow is a major factor that causes breakdown at a merge junction. However, research has not quantified the exact effect of ramp demand on the capacity of the merge junction.

The main objective of this research is to observe various merge junctions and develop models that estimate the bottleneck capacity as a function of the mainline flow and the ramp flow. Additional parameters such as the number of lanes and the free-flow speed will be considered for the models.

CHAPTER 3. METHODOLOGY

This chapter discusses the methodology that is used for this research. First the selection process of various merge bottleneck locations is presented. Then, the identification of the breakdown occurrences is discussed. Next, a description of the model development considering various parameters is presented. Finally, the calculation of recommended capacity values is explained.

3.1 Selection of Critical Merge Junctions

The merge junctions to be considered for analysis should be "critical junctions," where breakdowns occur due to merging operations, while (near) free-flowing conditions occur downstream to eliminate the impact of downstream congestion. The selection of critical merge locations is done in two steps.

Step 1: Selecting locations based on presence of detectors; the detectors should be present upstream, downstream and at the merge.

Step 2: Selecting critical junctions by analyzing the detectors selected in step 1 using time series plots.

Both steps are explained in detail in sections 4.2 and 4.3 of chapter 4.

3.1 Identification of Breakdown Events

The first step is the identification of the breakdown events for each site studied and for all days available. This would be done by using the breakdown identification algorithm presented in Highway Capacity Manual 6th Edition (TRB, 2016). The algorithm is explained in detail, below.

Step 1: Check for a sudden drop in speed, at least 25% below the free flow speeds, at any 5-min time interval.

Step 2: Observe if this speed drop is consistent over a period of 15-minutes, which is three consecutive 5-minute intervals.

For this research free flow speed has been defined as the average of all the speeds, when speeds were greater than 50 mph and the flows less than 800 veh/hr. Once the above steps are performed and the breakdown time interval is identified, the 5-minute average flow rate measured downstream of the bottleneck, that corresponds to the 5-minute interval right before the breakdown, is the capacity of the bottleneck. Additionally, during the same time interval, the 5-minute average flow rate upstream of the merge is also obtained (i.e., upstream breakdown flow), as well as the 5-minute average ramp flow (i.e., ramp breakdown flow). All data obtained above are used to develop capacity estimation model discussed in the following section.

3.2 Developing Capacity Model

Once all the capacity values mentioned in the previous section are obtained, a regression model is developed. Several variables are considered for the capacity model development. The final selection of the variables depends on the reasonableness of the model and its goodness of fit. The variables that are considered are: upstream breakdown flow, ramp breakdown flow, number of lanes, free-flow speed, subsets of freeway demand i.e., outer two lane flows, shoulder lane flows, remaining lane flows, and all interaction variables. All variables and their relation with capacity are discussed in detail in chapter 5, model development. All parameters significant at a 95% level were retained in the final model.

The regression model was developed using 70% of the data (randomly selected in SPSS software), and the remaining 30% of the data was set aside for validation.

3.3 Recommending Capacity Values

Based on the derived model, this thesis proposes recommended capacity values for merge bottleneck locations as a function of the number of lanes, the ramp and freeway flow, and other parameters that appear to be statistically significant.

CHAPTER 4. DATA COLLECTION AND ANALYSIS

In this chapter the data collection and analysis is described in detail. The first section details the requirements for the data collection. In the next section the initial screening for selecting the critical junctions is explained. Later, the final screening process for selecting the critical merge locations is presented. The final section of this chapter presents detailed description of all selected critical merge locations.

4.1 Data Description

Data need to be collected at specific locations to ensure that breakdown events and capacities are identified and obtained. The data collection requirements are:

- Detectors present downstream, upstream of the merge and at the on-ramp, as shown in Figure 4;
- Freeway volume, occupancy and speed data available at all detector stations at 5-minute increments;
- Data available for at least six months, excluding holidays, weekends, and construction/work zone periods; and
- Weather and incident data available at the same sites.

All the detector data that were used for the analysis were downloaded from KC Scout portal (http://www.kcscout.net/KcDataPortal). Various intervals were available in the portal but 5-minute intervals were chosen for analysis purposes. The 5-minute intervals were chosen to detect breakdowns and to also find the independent bottleneck locations (locations where breakdown occurs independently due to the interaction between on-ramp flow and mainline flow and not due to the spillback of downstream congestion).

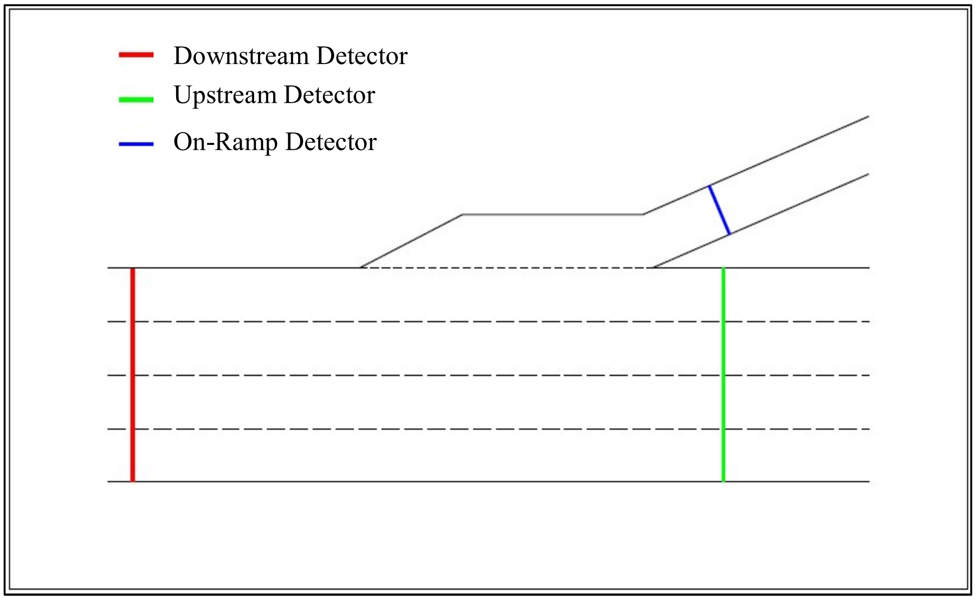

Figure 4: Detector Positions

Data of only peak periods and weekdays were considered for the analysis. Days with incidents at the study merge locations and days with adverse weather conditions were removed from the analysis, to ensure that congestion occurs due to merging and not due to these factors.

Incident data were obtained from KC Scout Portal as well. Incident data, from January 1, 2016 to June 30, 2016, were gathered. The incident data included details about the time of the incidents, the place the incident happened, the type of incident that happened (minor, major, stalled vehicle, etc.). All the days with incidents which might affect the peak hour traffic were removed from the analysis. If the peak period at a location was from 4:00 PM to 6:00 PM and the incident happened before 3:30 PM or after 6:30 PM then those days were retained in the data set.

Weather data for this research were obtained from www.wunderground.com. The weather data in the Kansas City area were collected by using the airport code MCI. Data from January 1, 2016 to June 30, 2016 were obtained. The precipitation was considered critical only when it was more than 0.02 inches. All days with precipitation which might affect the peak hour traffic were removed from the analysis. If the peak period at a location was from 4:00 PM to 6:00 PM and the precipitation occurred before 3:30 PM or after 6:30 PM then those days were retained in the data set.

4.2 Initial Screening Process

The initial screening process was based on the presence of detectors upstream, downstream and at the on-ramp. All merge segments along I-35, I-435, I-70, I-635, I-29 freeways in the Kansas City area were screened to identify candidate sites. After identifying potential locations based on detector availability, average speed, volume, and occupancy data were obtained. Table 1 lists all the merge locations which were screened based on the detector locations.

Table 1: Initial Site Selection

Location	Interstate
Parallel Parkway NB	I-635
North of State Ave NB	I-635
Shawnee Drive NB	I-635
Kansas Avenue NB	I-635
Leavenworth Road NB	I-635
38th Street SB	I-635
Parallel Parkway SB	I-635
Armour Road	I-29
Front Street	I-29
South West Boulevard SB	I-35
18th Street Express Way NB	I-35
Antioch Road NB	I-35
Antioch Road SB	I-35
Johnson Drive NB	I-35
67th Street NB	I-35
75th Street NB	I-35
75th Street SB	I-35
87th Street NB	I-35
87th Street SB	I-35
95th Street NB	I-35
95th Street SB	I-35
Bedford Road	I-35

Table 1: Continued

Location	Interstate
Lee's Summit WB	I-70
Blue Ridge Cut Off EB	I-70
Jackson Avenue EB	I-70
Lee's Summit EB	I-70
Little Blue Parkway EB	I-70
Van Brunt Boulevard EB	I-70
Woods Chapel EB	I-70
Little Blue Parkway WB	I-70
Noland Road WB	I-70
US-40 Highway WB	I-70
West of MO7 WB	I-70
Woods Chapel WB	I-70
Metcalf Avenue EB	I-435
Roe Boulevard	I-435
Holmes Road EB	I-435
104th Street EB	I-435

4.3 Secondary Screening Process

The secondary screening process was based on time series plots. A breakdown, which happens due to the interaction between freeway flow and on-ramp flow, was considered to be a true breakdown. In this step, data from all the locations, selected in the process above, were investigated to check for merge locations which experience true breakdowns. For this analysis, speed time-series diagrams were developed. The merge junctions that did not experience congestion due to downstream queue spillback, but mainly experienced congestion due to interaction between freeway flow and on-ramp flow were selected for further analysis.

Figure 5 shows an example of a time series plot created using the data collected at the merge location at Little Blue Parkway on I-70. From the time series plot it is observed that the speeds at downstream detector, I-70 W @ LITTLE BLUE RIVER were dropping before the speeds at the upstream detector (I-70 W @ LITTLE BLUE PKW). As the speed drop was first observed further downstream from this location, it was removed from the analysis. All locations which show similar results were also excluded from the analysis.

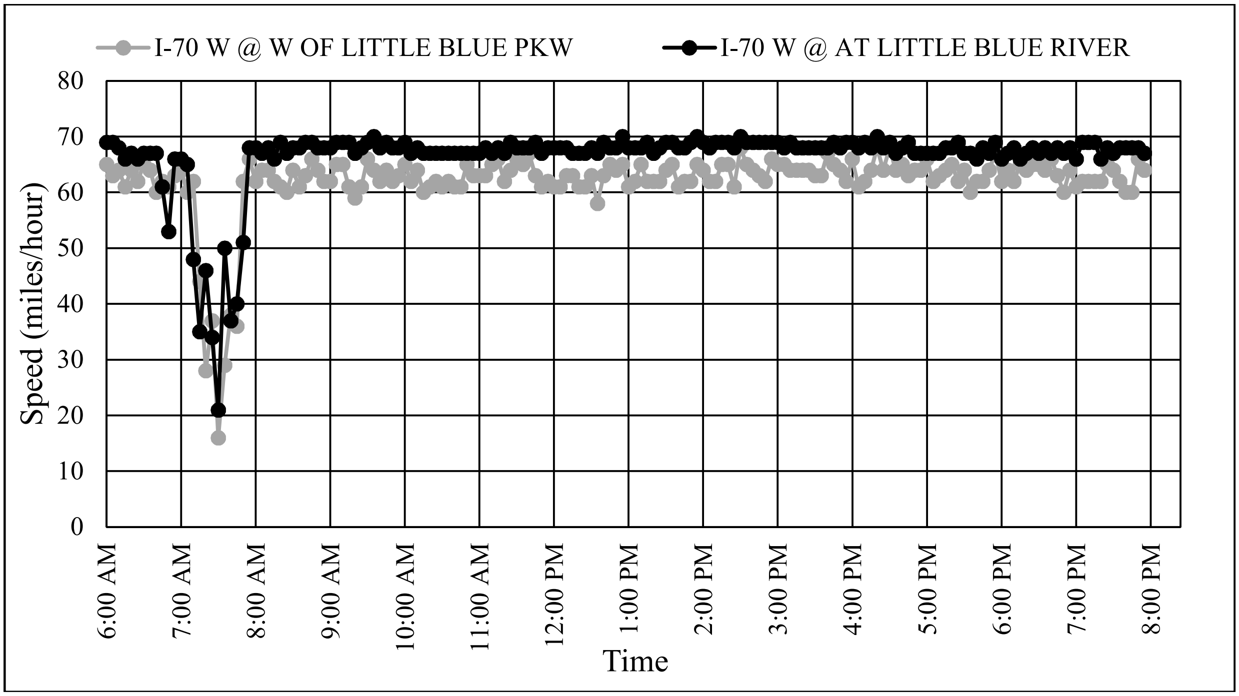

Figure 5: Time series plot for I-70 at Little Blue Parkway Merge Junction

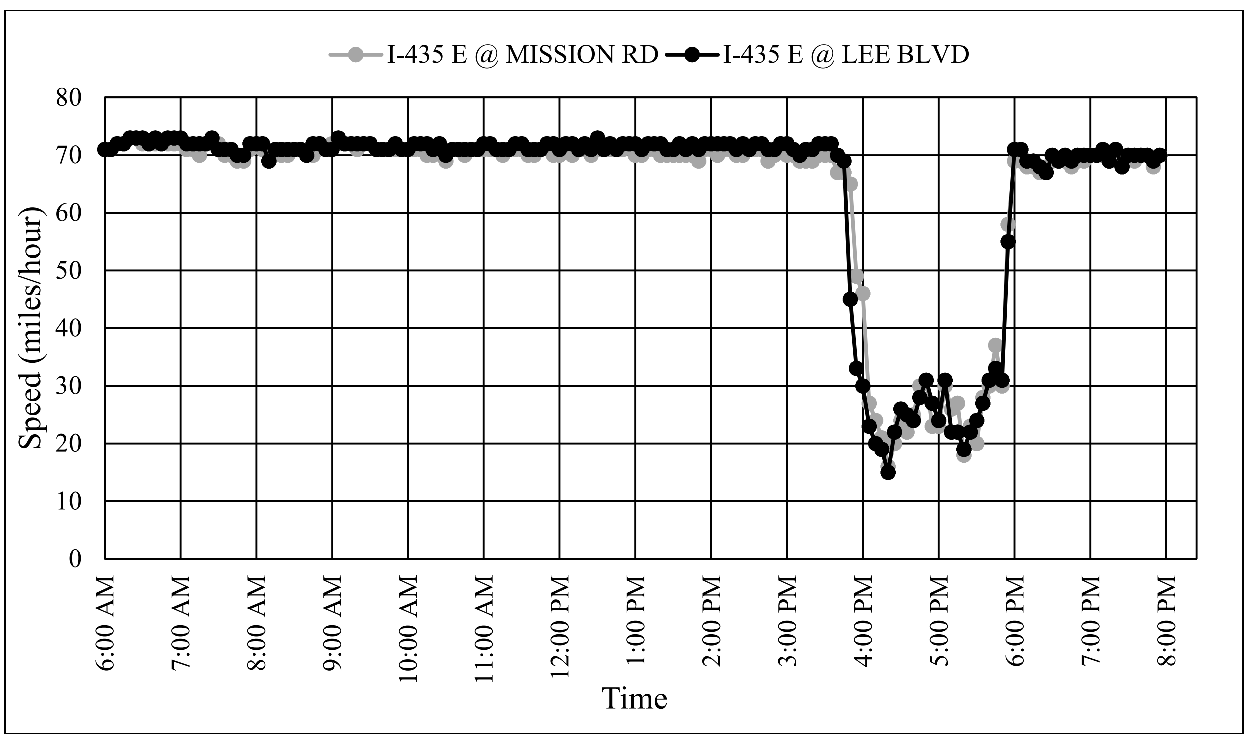

Figure 6: Time series plot for I-435 at Roe Boulevard

Figure 6 shows the time series plot at the merge location I-435 at Roe Boulevard. From the time series plot it was observed that the speeds at upstream detector, I-435 E @ MISSION ROAD were dropping before the speeds at downstream detector, I-435 E @ LEE BLVD. As the speed drops were first observed at the upstream detectors, near the merge, this location was selected for further analysis. All locations which show similar trends were also selected for analysis.

Using this process of the speed time-series plots all merge bottleneck locations in the Kansas City area were checked. Table 2 shows all the locations which were selected during the initial screening process and gives the final locations which were selected for developing the capacity regression model.

Table 2: Secondary Screening Process

Location	Interstate	Congestion	Selected/Not-Selected
Parallel Parkway NB	I-635	Downstream Spillback	Not-Selected
North of State Ave NB	I-635	Downstream Spillback	Not-Selected
Shawnee Drive NB	I-635	Downstream Spillback	Not-Selected
Kansas Avenue NB	I-635	Downstream Spillback	Not-Selected
Leavenworth Road NB	I-635	Downstream Spillback	Not-Selected
38^{th} Street SB	I-635	Downstream Spillback	Not-Selected
Parallel Parkway SB	I-635	Downstream Spillback	Not-Selected
Armour Road	I-29	Downstream Spillback	Not-Selected
Front Street	I-29	Downstream Spillback	Not-Selected
South West Boulevard SB	I-35	Downstream Spillback	Not-Selected
18^{th} Street Express Way NB	I-35	Downstream Spillback	Not-Selected
Antioch Road NB	I-35	Downstream Spillback	Not-Selected
Antioch Road SB	I-35	Downstream Spillback	Not-Selected
Johnson Drive NB	I-35	Downstream Spillback	Not-Selected
67^{th} Street SB	I-35	At Merge	Selected
75^{th} Street NB	I-35	At Merge	Selected
75^{th} Street SB	I-35	At Merge	Selected
87^{th} Street NB	I-35	Downstream Spillback	Not-Selected
87^{th} Street SB	I-35	Downstream Spillback	Not-Selected
95^{th} Street NB	I-35	Downstream Spillback	Not-Selected

Table 2: Continued

Location	Interstate	Congestion	Selected/Not-Selected
95th Street SB	I-35	Downstream Spillback	Not-Selected
Bedford Road	I-35	At Merge	Selected
Lee's Summit WB	I-70	Downstream Spillback	Not-Selected
Blue Ridge Cut Off EB	I-70	Downstream Spillback	Not-Selected
Jackson Avenue EB	I-70	Downstream Spillback	Not-Selected
Lee's Summit EB	I-70	Downstream Spillback	Not-Selected
Little Blue Parkway EB	I-70	Downstream Spillback	Not-Selected
Van Brunt Boulevard EB	I-70	Downstream Spillback	Not-Selected
Woods Chapel EB	I-70	Downstream Spillback	Not-Selected
Little Blue Parkway WB	I-70	Downstream Spillback	Not-Selected
Noland Road WB	I-70	At Merge	Selected
US-40 Highway WB	I-70	At Merge	Selected
West of MO7 WB	I-70	Downstream Spillback	Not-Selected
Woods Chapel WB	I-70	Downstream Spillback	Not-Selected
Metcalf Avenue EB	I-435	Downstream Spillback	Not-Selected
Roe Boulevard	I-435	At Merge	Selected
Holmes Road EB	I-435	At Merge	Selected
104th Street EB	I-435	At Merge	Selected

4.4 Selected Locations

Based on the selection process the following nine merge locations were selected for the final analysis.

1. I-35 @ 67th ST (South Bound)
2. I-35 @ 75th ST (North Bound)
3. I-35 @ 75th ST (South Bound)
4. I-35 @ Bedford Road (North Bound)
5. I-70 @ Noland Road (West Bound)
6. I-70 @ US-40 HWY (West Bound)
7. I-435 @ 104th ST (East Bound)
8. I-435 @ East Holmes Road (East Bound)

9. I-435 @ Roe Avenue (East Bound)

Out of these nine selected location three, locations 2, 3 and 4, were removed because of the bad detector data. The data from the remaining six locations were used for further analysis. The selected locations are described below.

4.4.1 I-35 @ 67th ST (Southbound):

This is a parallel type merge location with four lanes on the mainline and one lane on the ramp. The peak periods at this location were recorded during morning, between 7:00 AM to 9:00 AM and evening, between 4:00 PM to 6:30 PM. The free flow speed (average speed during low flow conditions) at this location is 64 mph and the speed limit is 60 mph. Figure 7 shows the schematic and Figure 8 shows the google maps image of I-35 South Bound at 67th Street.

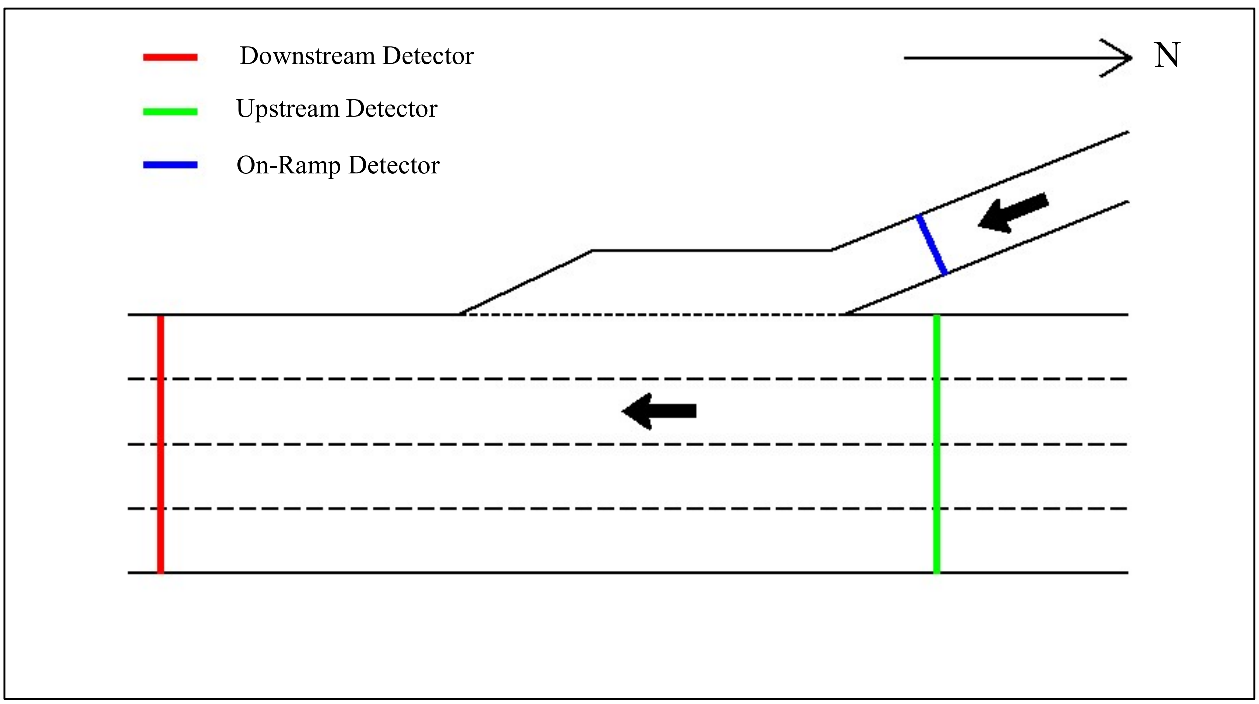

Figure 7: I-35 South Bound at 67th Street

Figure 8: I-35 South Bound at 67th Street (Google Maps)

4.4.2 I-70 @ Noland ST (Westbound):

This a parallel type merge location with three lanes on the mainline and one lane on the ramp. The peak period at this location were recorded during morning, between 6:00 AM to 8:30 AM. The free flow speed at this location was 64 mph and the speed limit was 65 mph. Figure 9 shows the schematic and Figure 10 shows the google maps image of I-70 West Bound at Noland Road.

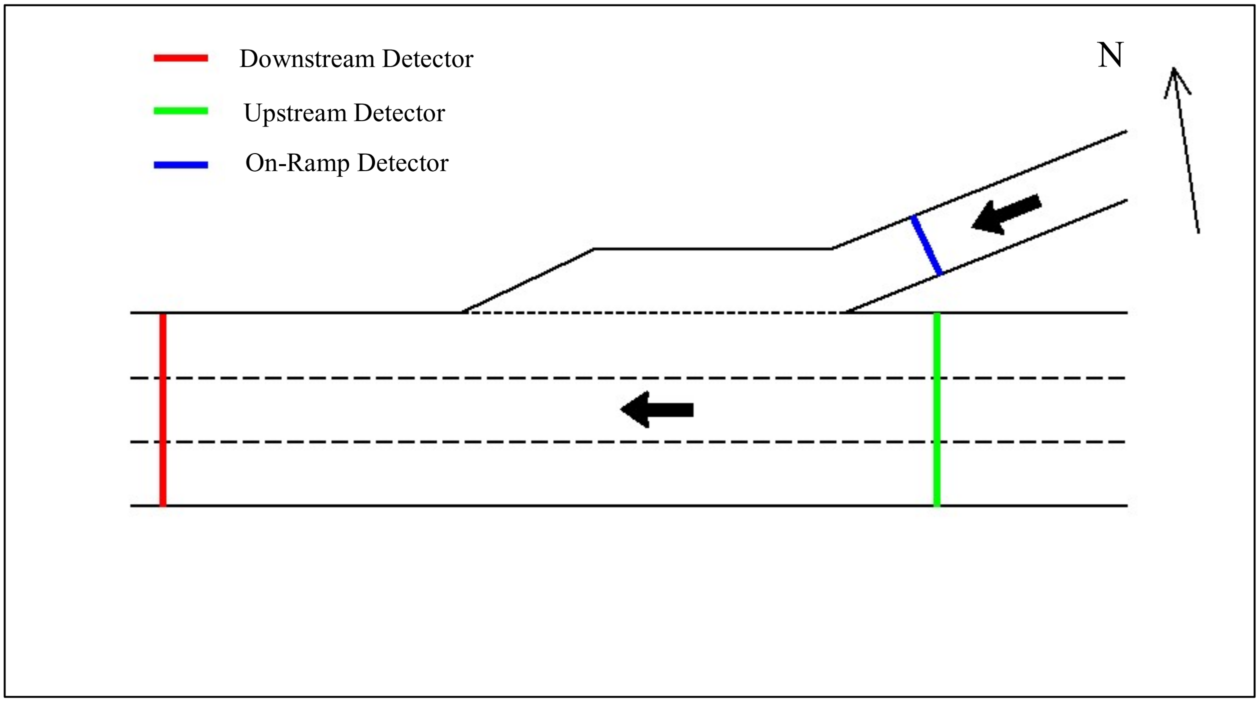

Figure 9: I-70 West Bound at Noland Road

Figure 10: I-70 West Bound at Noland Road (Google Maps)

4.4.3 I-70 @ US-40 HWY (Westbound):

This a parallel type merge location with three lanes on the mainline and one lane on the ramp. The peak period at this location were recorded during morning, between 6:00 AM to 8:30 AM. The free flow speed at this location was 61 mph and the speed limit was 65 mph. Figure 11 shows the schematic and Figure 12 shows the google maps image of I-70 West Bound at US-40 Highway.

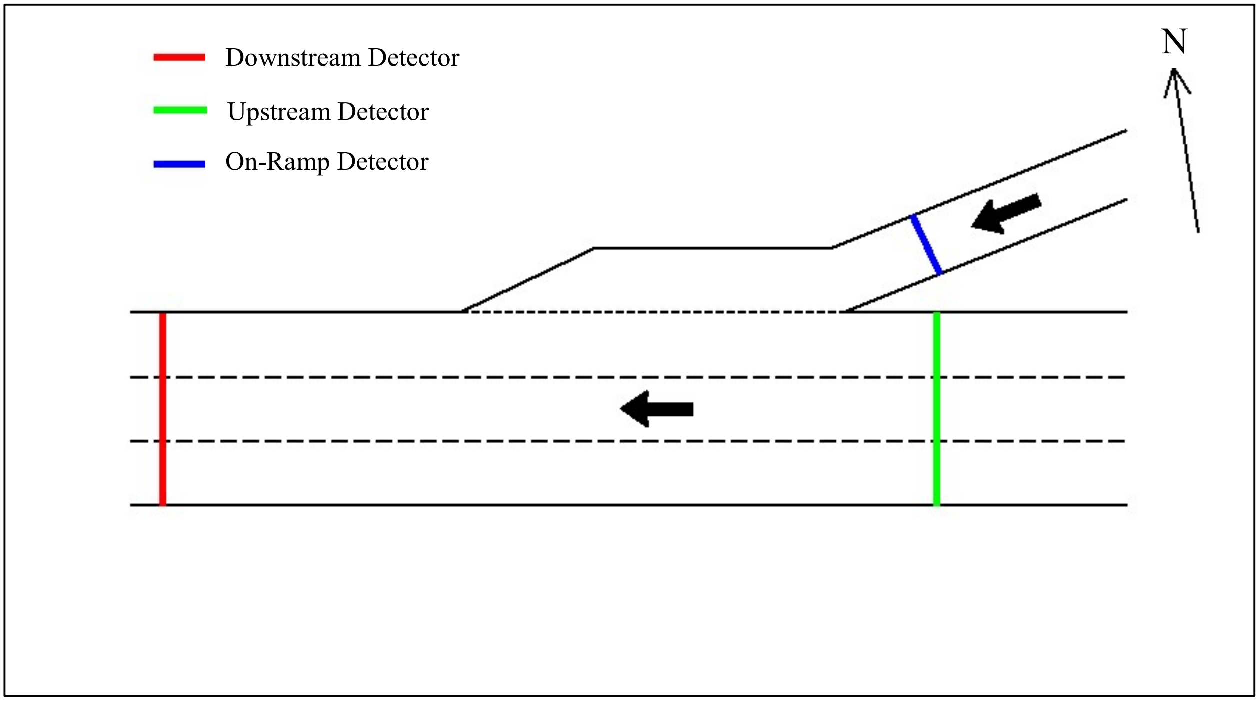

Figure 11: I-70 West Bound at US-40 Highway

Figure 12: I-70 West Bound at US-40 Highway (Google Maps)

4.4.4 I-435 @ 104th ST (Eastbound):

This a parallel type merge location with four lanes on the mainline and one lane on the ramp. The peak period at this location were recorded during evening, between 3:30 PM to 5:00 PM. The free flow speed at this location was 71 mph and the speed limit was 65 mph. Figure 13 shows the schematic and Figure 14 shows the google maps image of I-435 East Bound at 104th Street.

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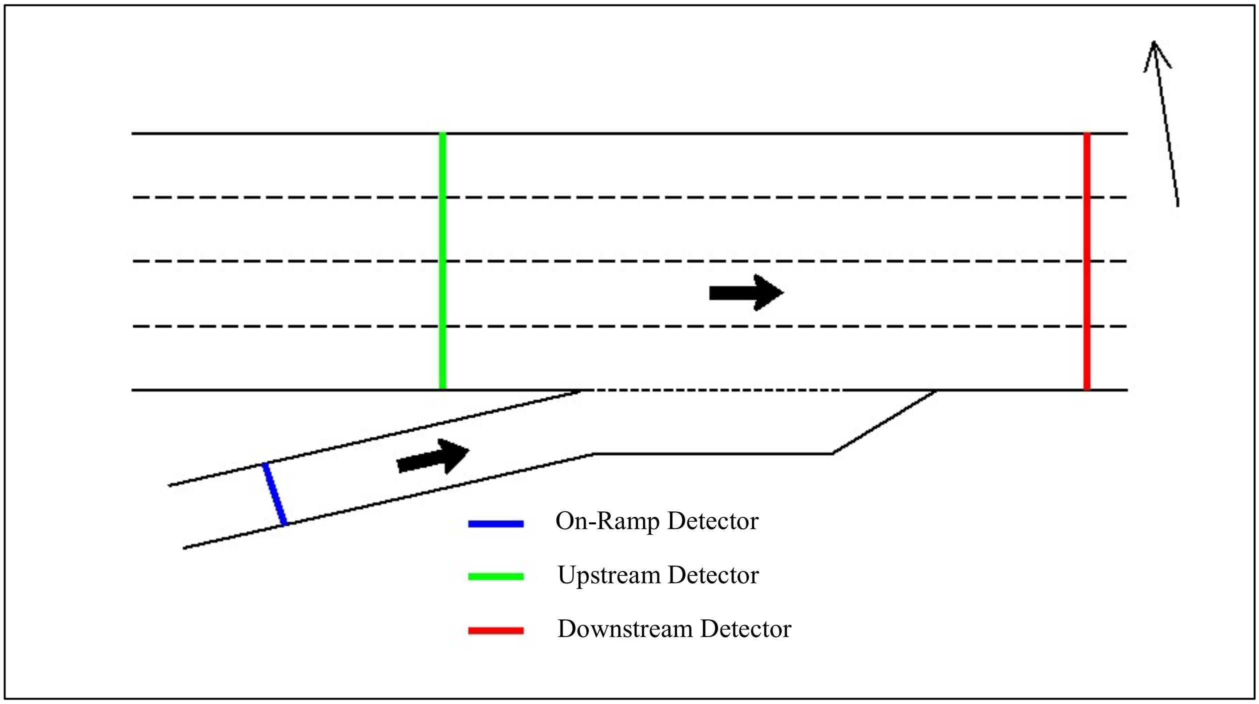

Figure 13: I-435 East Bound at 104th Street


Figure 14: I-435 East Bound at 104th Street (Google Maps)

4.4.5 I-435 @ East Holmes Road (Eastbound):

This a parallel type merge location with four lanes on the mainline and one lane on the ramp. The peak period at this location were recorded during afternoon, between 3:30 PM to 5:00 PM. The free flow speed at this location was 68 mph and the speed limit was 65 mph. Figure 15 shows the schematic and Figure 16 shows the google maps image of I-435 East Bound at East Holmes Road.

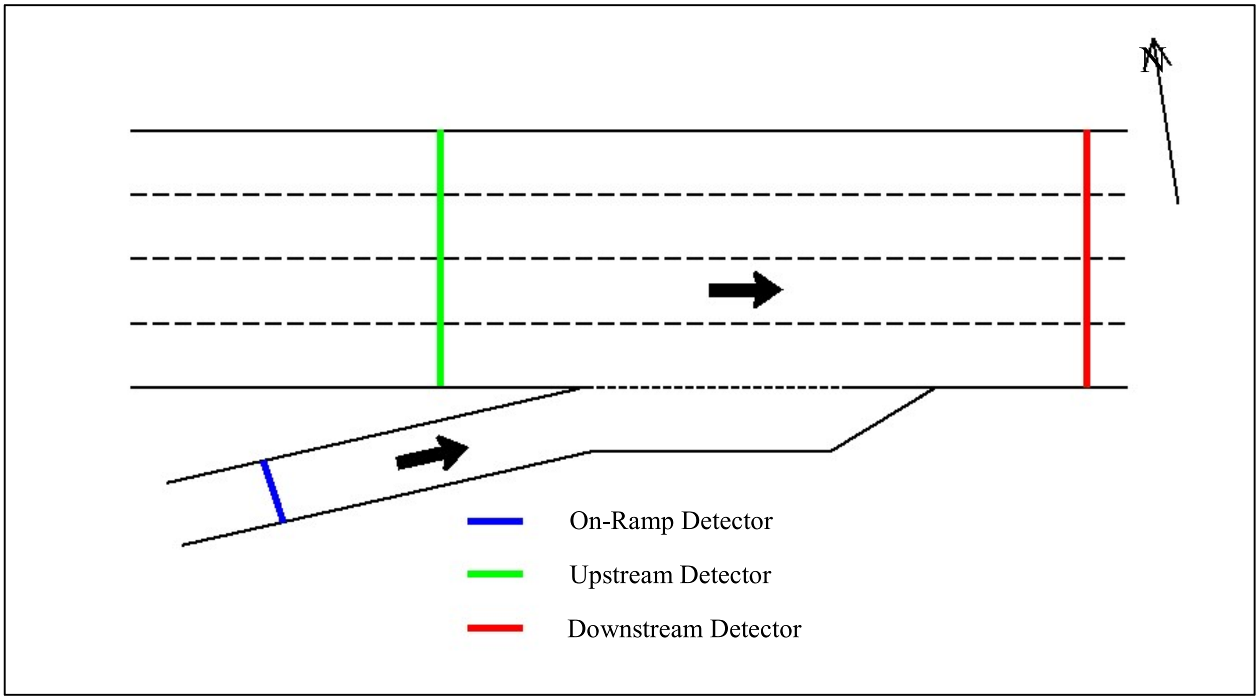

Figure 15: I-435 East Bound at East Holmes Road

Figure 16: I-435 East Bound at East Holmes Road (Google Maps)

4.4.6 I-435 @ Roe Avenue (East Bound):

This a parallel type merge location with four lanes before and five lanes on the mainline and two lanes on the ramp. The peak period at this location was recorded during afternoon, between 3:30 PM to 5:00 PM. The free flow speed at this location is 70 mph and the speed limit is 65 mph. Figure 17 shows the schematic and Figure 18 shows the google maps image of I-435 East Bound at Roe Avenue.

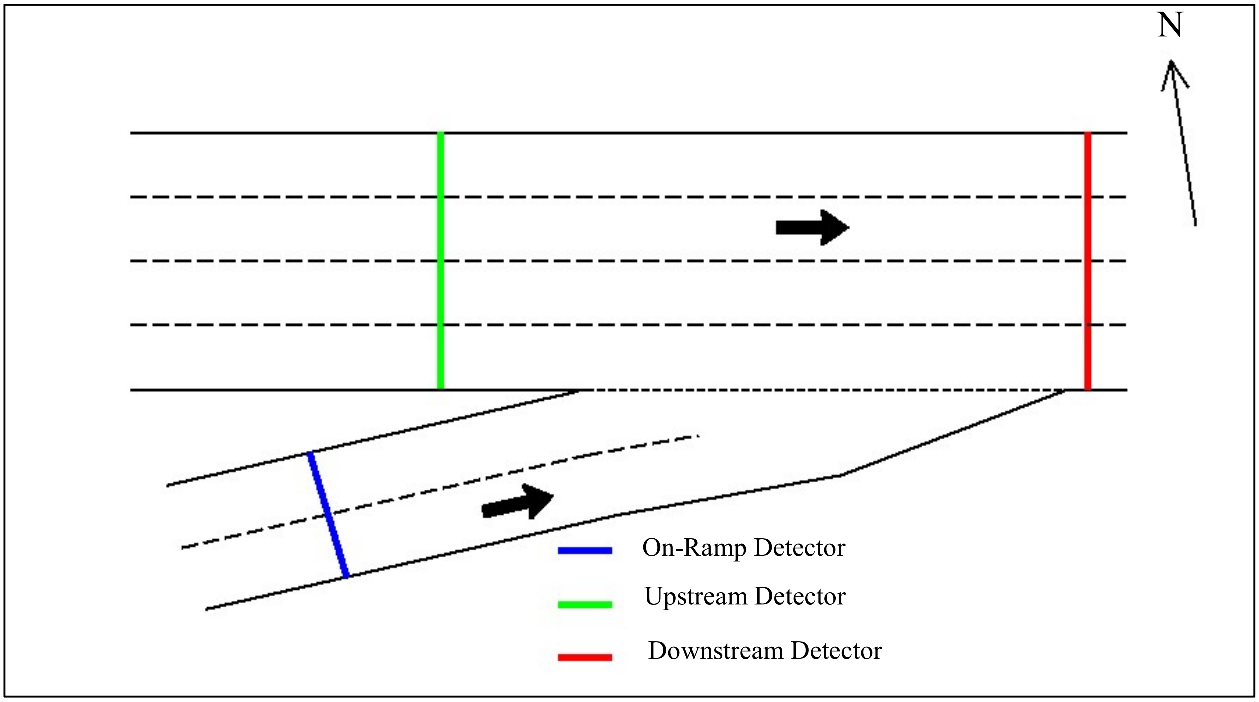

Figure 17: I-435 East Bound at Roe Avenue

Figure 18: I-435 East Bound at Roe Avenue (Google Maps)

CHAPTER 5. MODEL DEVELOPMENT

This chapter discusses the model development process. The first section discusses the breakdown identification at the selected merge locations. In the second section, various independent variables considered for the model are explained in detail. Later, the regression model and its validation are explained. Finally, recommended capacity values at merge junctions are listed.

5.1 Identification of Breakdown

After selecting all the critical merge locations, six months' worth of data were collected at all the locations. Days with bad weather and incidents were removed, as described in section 3.1, in order to focus on true breakdowns due to merging operations. Free Flow Speeds (FFS) were calculated for each location. FFS was calculated by averaging all the speeds higher than 50 mph and flows lower than 800 veh/hr. Breakdown was said to occur when speed drops below 25% of the FFS for at least 15 minutes (three consecutive five minute intervals) (TRB, 2016). The FFS and thresholds for each site are listed in Table 3.

Table 3: Free Flow Speeds and thresholds for selected locations

	Merge Junctions	Direction	Number of lanes	FFS (mph)	Speed Threshold (mi/h)
1	I-70 @ Noland Road	West Bound	3	64	48
2	I-70 @ US 40 HWY	West Bound	3	61	46
3	I-35 @ 67th Street	South Bound	4	64	48
4	I-435 @ 104th Street	East Bound	4	71	53
5	I-435 @ East Holmes Road	East Bound	4	68	51
6	I-435 @ Roe Avenue	East Bound	4	70	53

Using these thresholds all breakdowns were identified at each location. Once the above steps were performed and the breakdown times were identified, the 5-minute average flow rate, measured downstream of the bottleneck, right before the speeds dropped below the threshold level, is the capacity of the bottleneck and all these values were extracted. Additionally, the upstream breakdown flow, which is the 5-minute average flow rate upstream of the merge, and the ramp breakdown flow, which is the 5-minute average ramp flow were also extracted at the same time interval. A total of 593 breakdown events were recorded.

5.2 Selecting Independent Variables

In this section all the variables that were initially selected for developing the model are described. This section also shows the relationship between capacity and the independent variables. All the variables with significant correlation with the dependent variable are retained in the model.

Demand (D) (veh/h/ln):

The upstream freeway flow recorded by the detectors right before the breakdown is the demand. Demand data were available for all the 593 breakdown events recorded. Figure 19 exhibits the relationship between downstream capacity (in veh/h/ln) and upstream demand (in veh/h/ln).

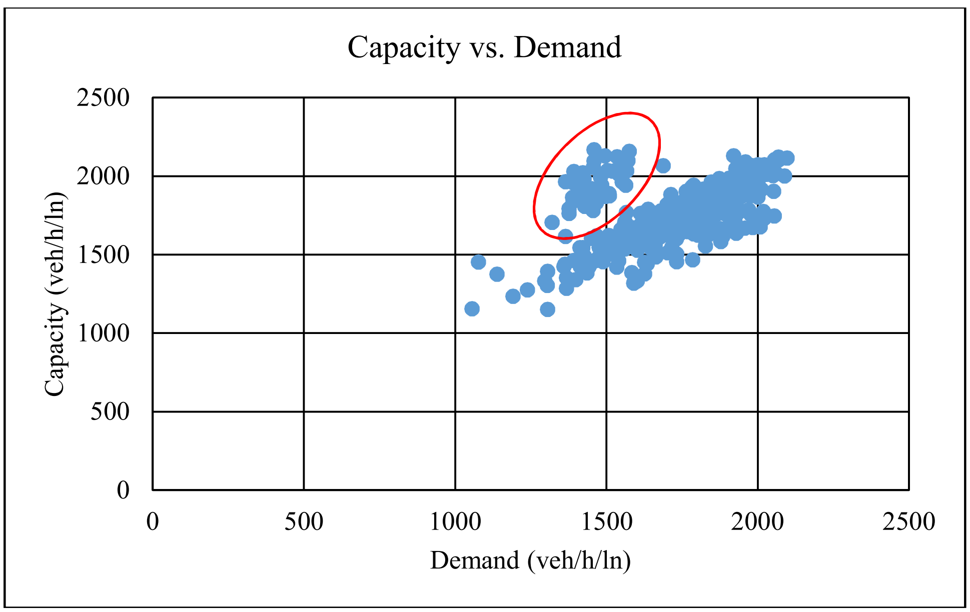

Figure 19: Relationship between Capacity and Demand

From the graph, it is clear that there is a positive correlation between capacity and demand, which means that there will be an increase in capacity as the demand increases. The data in the red circle shown in Figure 19, appear to correspond to 3-lane merge segments; while the remaining points capture capacity values at both 3-lane and 4-lane segments. This suggests that 3-lane segments have generally higher capacity values than 4-lane segments, as this is also discussed in a later section of this thesis.

Ramp Demand (RD) (veh/h/ln):

The ramp flow recorded by the detectors right before the breakdown is the ramp demand. The ramp demand data were available for all the 593 breakdown events recorded. Figure 20 exhibits the relationship between the capacity (veh/hr/lane) and ramp demand (veh/hr/lane).

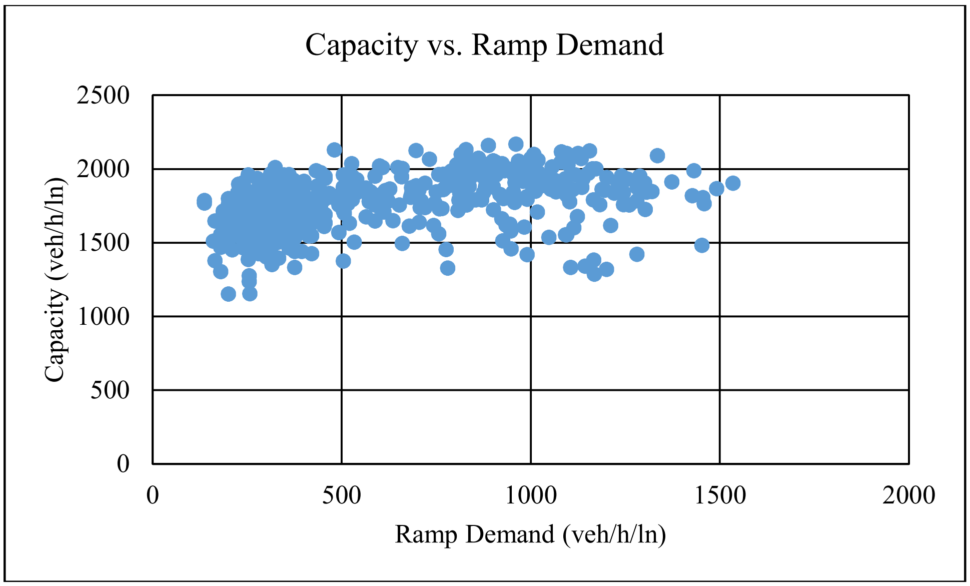

Figure 20: Relationship between Capacity and Ramp Demand

From the graph the correlation between ramp demand and capacity is not clear. The relation will be further examined while developing the regression model.

Free Flow Speed (FFS) (mi/h):

The average of all the speeds when flows were less than 800 veh/h and the speeds were more than 50 mph is defined as Free Flow Speed for this research. The free flow speeds at all segments considered for this research range from 64 mph to 71 mph. Figure 21 exhibits the relationship between the capacity (in veh/h/ln) and free flow speed (in mph). Although the graph reveals a strong correlation between capacity and FFS, the range of free flow speeds at the study sites observed was very small. Hence proper judgment cannot be made about the relationship between capacity and free flow speed. That is why this variable had not been considered in developing the regression model.

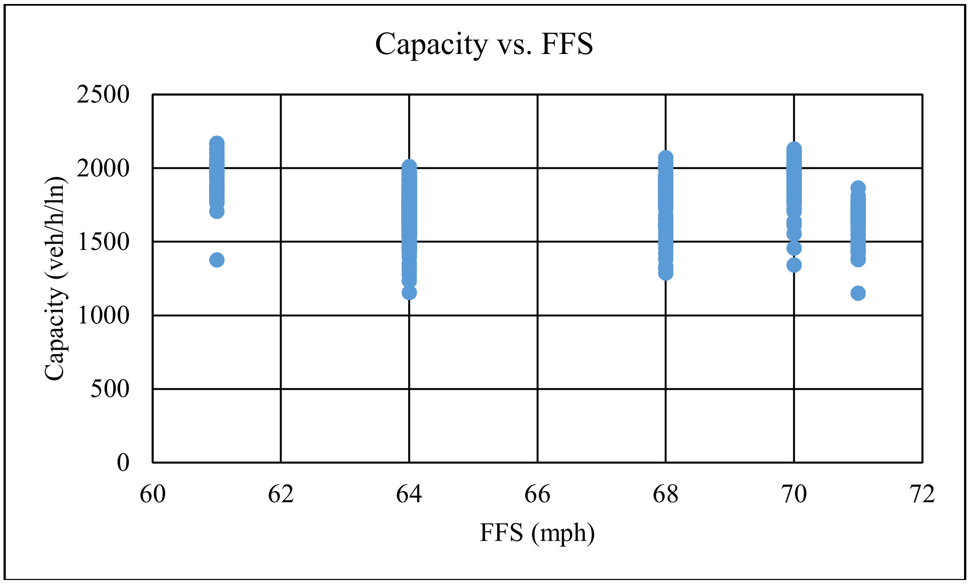

Figure 21: Relationship between Capacity and Free Flow Speed

Number of Lanes (N):

This variable represents the total number of lanes at the merge junction. The number of lanes at each location considered for the analysis were checked using Google maps, satellite imagery. There were only two variations in this variable, three and four lanes. The locations I-70 at Noland road and I-70 at US-40 highway have three lanes at the merge and the remaining locations have four lanes. Figure 22 exhibits the relationship between the capacity (veh/h/ln) and number of lanes. From the graph it is observed that there is a negative correlation between capacity and number of lanes which means that there will be a decrease in capacity as the number of lanes increases. This is consistent with finding from the research by Lu & Elefteriadou, 2013 and Kondyli et al., 2016.

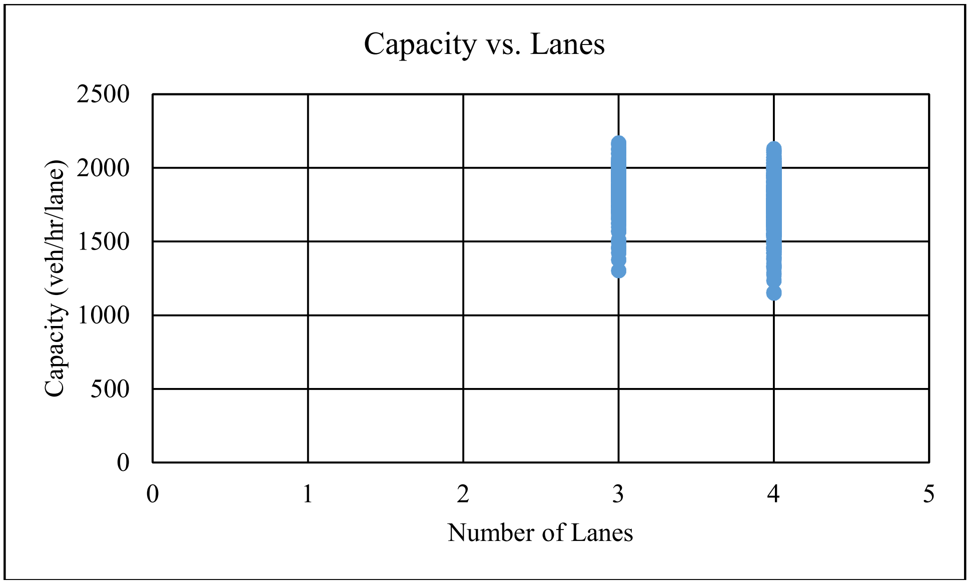

Figure 22: Relationship between Capacity and Number of Lanes

Ramp to Freeway Demand Ratio (R):

This is the ratio of ramp demand to the freeway demand. As the freeway and ramp demand data were available for all the 593 breakdown events, the ratio was also calculated for all the 593 events. Figure 23 exhibits the relationship between capacity (veh/h/ln) and Ratio.

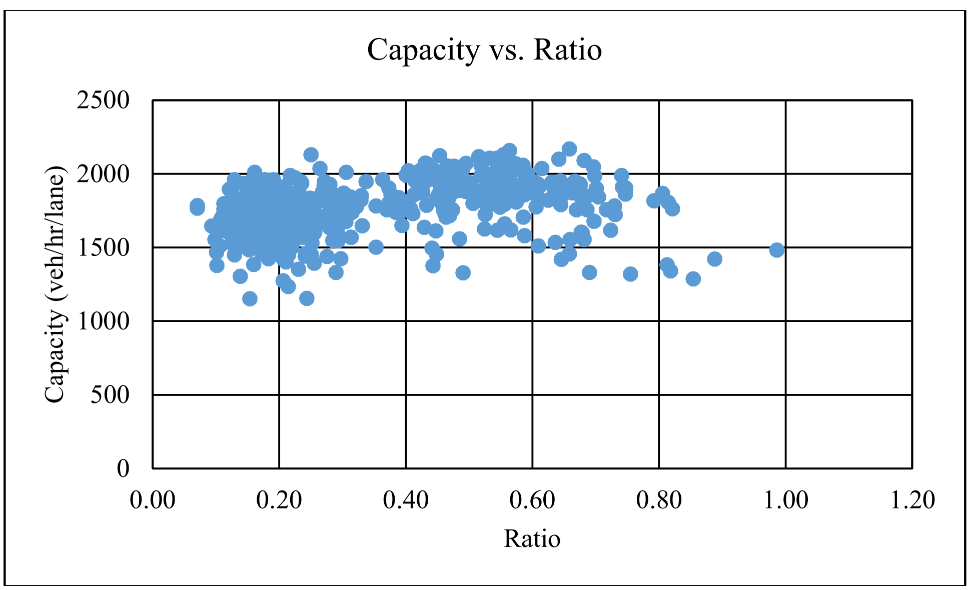

Figure 23: Relationship between Capacity and Ratio

From the graph the correlation between ratio and capacity is still not clear. When the ratio was low the capacities were scattered between 1200 veh/h/ln to 2200 veh/h/ln but when the ratios were increasing the capacities were falling. This relation will be further examined while developing the regression model.

Outer two lane flow (OTLF) (veh/h/ln):

The average flow at the outer two (right) lanes measured upstream of the merge junction recorded by the detectors right before the breakdown was collected. The detector data obtained from the KC Scout portal have individual lane data. The OTLF is calculated by adding the flows of the outer two lanes and averaging then across both lanes. This variable is mainly selected to check whether the outer two lane flows, which corresponds to the ramp influence area, are majorly affecting the capacity of the merge junction. Figure 24 exhibits the relation between the capacity (in veh/h/ln) and Outer Two Lane Flow (in veh/h/ln).

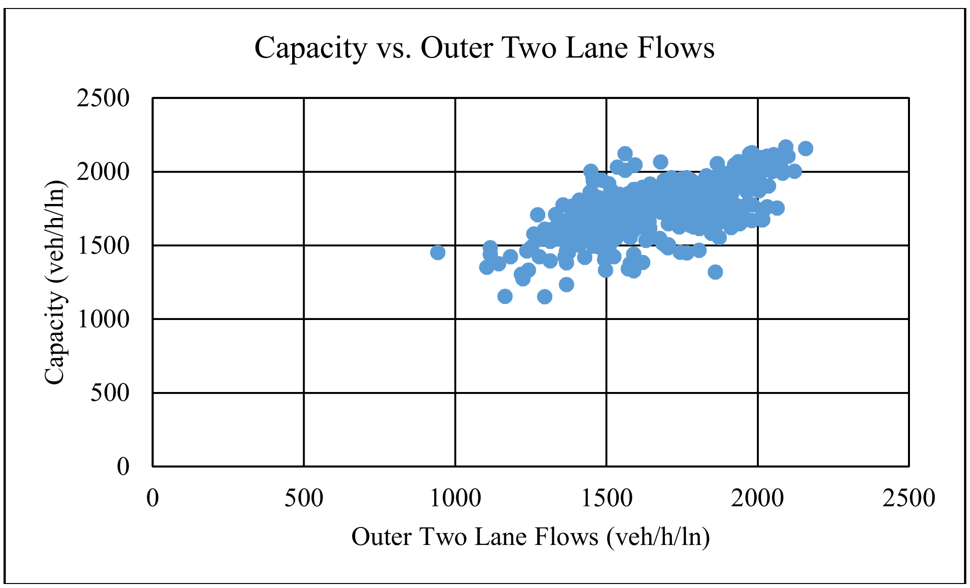

Figure 24: Relationship between Capacity and Outer Two Lane Flow

From the graph it is clear that there is a positive correlation between capacity and outer two lane flows, which means that there will be an increase in capacity as the outer two lane flows increases.

Shoulder lane flow (SLF) (veh/hr/ln):

The average flow on the outer shoulder lane measured upstream of the merge junction recorded by the detector right before the breakdown was collected. Figure 25 exhibits the relationship between the capacity (in veh/h/ln) and shoulder lane flow (in veh/h/ln). From the graph it is observed that there is a positive correlation between capacity and shoulder lane flow. This relation will be further examined while developing the model.

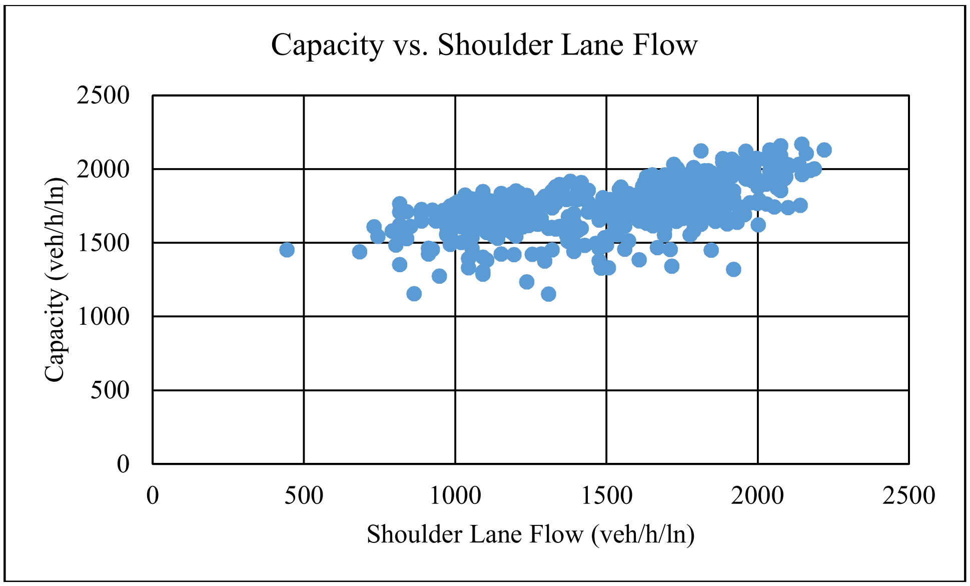

Figure 25: Relationship between Capacity and Shoulder Lane Flow

Remaining lanes flow (RLF) (veh/h/ln):

There are two types of remaining lane flows in this research. The first one would be RLF1, when the shoulder lane flow is chosen as one of the variable in the regression model. For a four lane segment this is calculated by averaging the remaining three inner lane flows. For a three lane segment this this is calculated by averaging the remaining two inner lane flows. The second one would be RLF2, when the outer two lane flow is chosen as one of the variable in the regression model. For a four lane segment this is calculated by averaging the remaining two inner lane flows. For a three lane segment this just the inner lane flow. It should also be noted that, it is redundant to choose both outer two lane flows and shoulder lanes flows in one model as they both overlap.

Figure 26 and Figure 27 exhibits the relationship between the capacity (in veh/h/ln) and remaining lane flow (in veh/h/ln). From the graph it is observed that there is a positive correlation between capacity and remaining lane flows. This relationship will be further examined while developing the model.

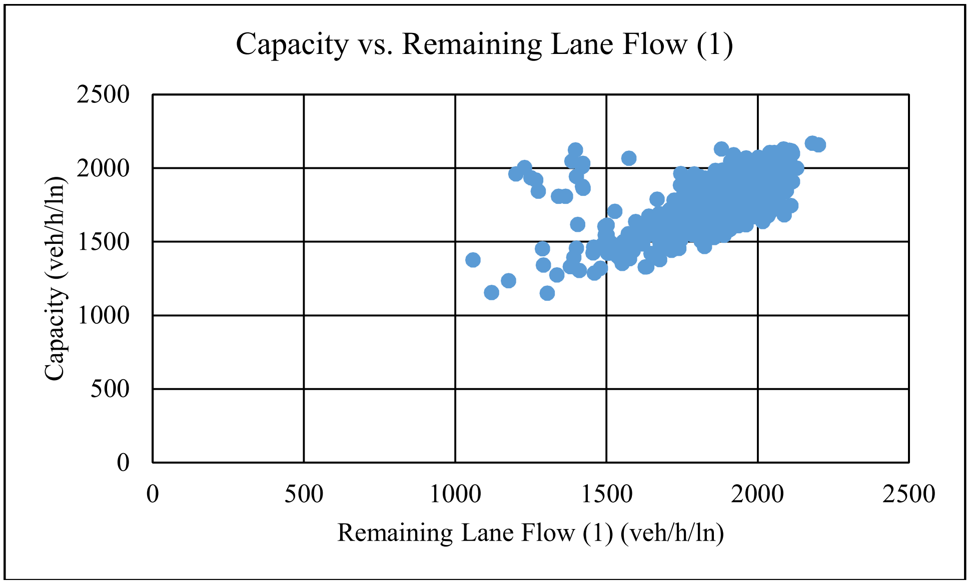

Figure 26: Relationship between Capacity and Remaining Lane Flow (1)

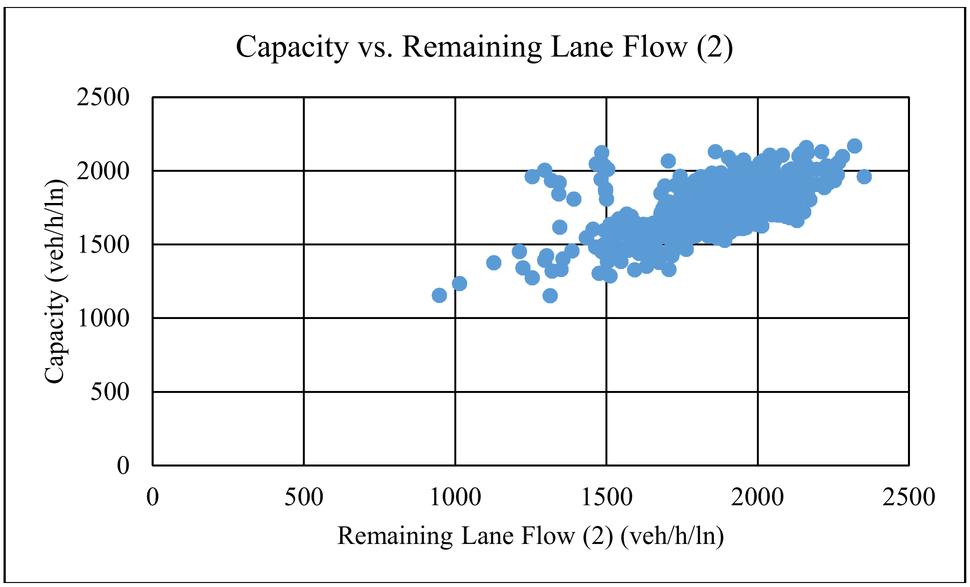

Figure 27: Relationship between Capacity and Remaining Lane Flow (2)

Interaction Variables:

Pearson's correlation was calculated between variables and if the correlation between two variables was more than 0.5, then the interaction term was also included in the variables which were used to develop the model. The correlation between two independent variables was checked using SPSS and Table 4 shows all correlations between the variables.

From the previous discussion it is noticed that some of the variables include demand and its subsets, such as outer two lane flow, shoulder lane flow and the remaining lane flow. Demand and its subsets were not included at once while developing a model as it would not make any sense to include variables with overlapping values. Similarly ramp demand and ratios of ramp demand to freeway demand are also not included in the model cocnurrently, to avoid overlap.

Table 4: Pearson's Correlation between independent variables

	C	D	RD	R	RLF1	RLF2	SLF	OTLF	FFS	L
C	1	0.564	0.428	0.334	0.585	0.612	0.623	0.676	0.011	-0.256
D	0.564	1	0.193	-0.004	0.699	0.603	0.514	0.689	0.542	0.181
RD	0.428	0.193	1	0.975	0.009	-0.002	0.54	0.448	0.33	0.08
R	0.334	-0.004	0.975	1	-0.119	-0.106	0.468	0.336	0.222	0.014
RLF1	0.585	0.699	0.009	-0.119	1	0.928	0.224	0.525	0.071	0.026
RLF2	0.612	0.603	-0.002	-0.106	0.928	1	0.237	0.424	-0.106	-0.237
SLF	0.623	0.514	0.54	0.468	0.224	0.237	1	0.911	0.483	-0.254
OTLF	0.676	0.689	0.448	0.336	0.525	0.424	0.911	1	0.528	-0.076
FFS	0.011	0.542	0.33	0.222	0.071	-0.106	0.483	0.528	1	0.547
L	-0.256	0.181	0.08	0.014	0.026	-0.237	-0.254	-0.076	0.547	1

From the above table all the interaction variables with high correlation are RDxSLF, DxFFS, FFSxL and RDxOTLF.

5.3 Capacity Model Development

Various models were developed to predict the capacity of merge bottleneck locations. SPSS statistical software was used to run the regression analysis. The stepwise regression method was used to select the variables for the model. A significance level of 95% was used for all the variables. FFS was thought to be a good contributor to the model but from Table 4 it became clear that there was a negligible correlation between FFS and capacity. Regression analysis was done using only FSS as the independent variable and the goodness of the fit (R^2) of the test was found to be zero. Hence FFS was not included in developing any of the models.

Different models using different combinations of variables were analyzed. Care was to avoid overlap between the included variables. For example, if D was selected as one of the variables, any of its subsets, SLF, RLF1, RLF2, OTLF, were not included in the model.

The final model was developed using 70% of the data. The data used for developing the model are selected randomly. The remaining 30% was used to validate the model. From the final model it was found that the ramp demand negatively affects the capacity i.e., the higher the ramp demand the lesser the capacity will be. It also estimates capacity as a function of the ramp demand, remaining lane flows, number of lanes, outer two lane flows, and the interaction between ramp demand and outer two lane flows.

The final model is:

$$\boldsymbol{Capacity = 1170.58 - 0.53RD + 0.31RLF2 + 0.12\mathrm{OTLF} - 70.06 \quad + \\ + 0.19(RD \times OTLF)} \qquad \textbf{Eq. 9}$$

Where, RD = Ramp Demand (veh/h/ln), RLF2 = Remaining Lane Flow (veh/h/ln), OTLF – Outer Two Lane Flow (veh/h/ln), N - Number of Lanes.

Table 5 and Table 6 provide details about the goodness of fit for the model, and the results of the regression analysis.

Table 5: Goodness of Fit

R	R^2	Adjusted R^2	Std. Error of the Estimate
0.82	0.689	0.675	96.08339

R^2 gives the goodness of fit of the model. Since R^2 is, high for the model shown in Eq. 9 it can be said that the model adequately describes the field data.

Table 6: Regression analysis results

	Unstandardized Coefficients, B	Std. Error	Standardized Coefficients, B	t	Sig.
(Constant)	1170.58	109.278		10.712	<0.01
RD	-0.53	0.135	-1.034	-3.913	<0.01
RLF2	0.31	0.027	0.383	11.639	<0.01
OTLF	0.12	0.048	0.156	2.584	<0.05
L	-70.06	11.874	-0.174	-5.9	<0.01
RDxOTLF	0.19	0.039	1.419	4.903	<0.01

5.3.1 Validation

After developing the capacity model shown in Eq. 9 predicted capacity values for both 70% of the data, with which the model was developed and 30% of the data, which were set aside for validation, were calculated. Next, 45 degree plots between field capacity values and predicted capacity were constructed for both data sets.Table 7 shows the correlation between field capacity values and predicted capacity values for both 70% and 30% data. It can be observed that the correlation between field capacity and predicted capacity values for both data sets are very close. Based on this observation we can concluded that the data is not over fit, which means that the model will have good predictive performance and it will not overreact to minor fluctuations in the training dataset.

In the next step of the validation the root mean squared error (RMSE), for field capacity values and predicted capacity values for the 30% data was calculated. The RMSE was found to be

95, which shows a small deviation in number of vehicles per hour per lane between the filed observed and predicted capacity values. Figure 28 show the 45 degree plot between the field and predicted values for the 30% data.

Table 7: Correlation Table

Correlations				
Sample			Observed Capacity Values	Predicted Capacity Values
30% Data		N	182	182
	Pearson's Correlation	Observed Capacity Values	1	.819**
		Predicted Capacity Values	.819**	1
		N	411	411
70% Data	Pearson's Correlation	Observed Capacity Values	1	.824**
		Predicted Capacity Values	.824**	1
		Sig. (2-tailed)	0	
**Correlation is significant at the 0.01 level (2-tailed).				

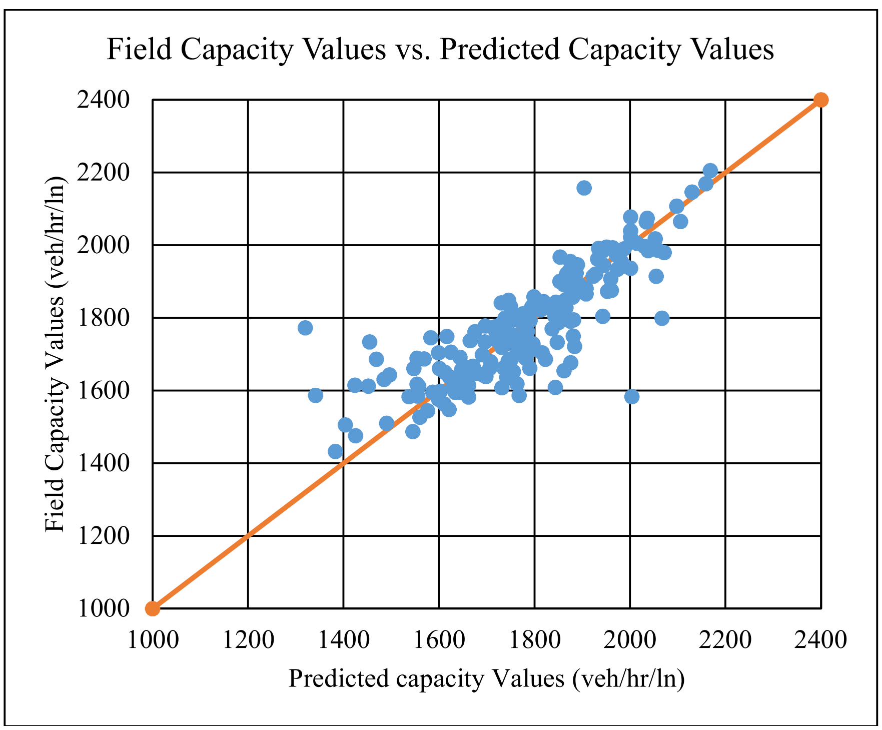

Figure 28: Field Capacity Values vs. Predicted Capacity Value

5.4 Recommended Capacity Values

The model developed for this research, which is shown in Equation 9, is used to propose capacity values as a function of ramp demand, upstream freeway demand (outer two lane flows and remaining lane flows), and number of lanes. Although upstream demand is not a direct variable in the model, an upstream demand was assumed and it was spilt into outer two lane flows and remaining lane flows. The assumed values were within the range of the observed field values. The following tables list capacity values for three and four lane freeways for a range of upstream breakdown flow between 1000 veh/h/ln to 2000 veh/h/ln and ramp breakdown flows between 100 veh/h/ln to 1500 veh/h/ln for 3 lane and 4 lane segments.

Table 8 give the capacity range for different freeway demand and ramp demand values. The detailed list of capacity values for different outer two lanes flows, remaining lane flows, and ramp demand are listed in APPENDIX A. Figure 29 to Figure 34 shows the relationship graphs between the predicted capacity values and ramp demand and also how the capacity varies depending upon the ramp demand, outer two lane flows, remaining lane flows and number of lanes.

From Table 8 and from Figure 29 to Figure 34 listed below various observations can be made.

i. Capacity decreases as ramp demand increases.
ii. Capacity decreases as the outer two lane flow increases.
iii. Capacity per lane decreases as the number of lanes increases.

Table 8: Capacity Ranges for Different Freeway Demand and Ramp Demand

D (veh/h)	3000	4000	8000	7000	6000	5000	4000	6000
N	3	3	4	4	4	4	4	3
RD (veh/h/ln)	Capacity Ranges (veh/h/ln)							
100	1355-1409	1415-1684	1692-1775	1528-1715	1405-1614	1345-1449	1285-1306	1696-1857
200	1322-1379	1382-1669	1673-1764	1500-1704	1372-1599	1312-1425	1252-1274	1674-1846
300	1288-1349	1348-1654	1655-1753	1472-1693	1338-1584	1278-1401	1218-1242	1651-1835
400	1254-1319	1314-1639	1636-1742	1443-1682	1304-1569	1244-1376	1184-1210	1629-1824
500	1220-1289	1280-1624	1617-1731	1415-1671	1270-1554	1210-1352	1150-1178	1607-1813
600	1186-1259	1246-1610	1599-1720	1387-1660	1236-1540	1176-1328	1116-1146	1584-1802
700	1152-1229	1212-1595	1580-1709	1359-1649	1202-1525	1142-1303	1082-1114	1562-1791
800	1118-1199	1178-1580	1562-1698	1331-1638	1168-1510	1108-1279	1048-1082	1539-1780
900	1084-1168	1144-1565	1543-1687	1302-1627	1134-1495	1074-1255	1014-1050	1517-1769
1000	1050-1138	1110-1550	1524-1676	1274-1616	1100-1480	1040-1230	980-1018	1494-1758
1100	1016-1108	1076-1536	1506-1665	1246-1605	1066-1466	1006-1206	946-986	1472-1747
1200	983-1078	1043-1521	1487-1654	1218-1594	1033-1451	973-1182	913-954	1450-1736
1300	949-1048	1009-1506	1468-1643	1190-1583	999-1436	939-1157	879-922	1427-1725
1400	915-1018	975-1491	1450-1632	1162-1572	965-1421	905-1133	845-890	1405-1714
1500	881-988	941-1476	1431-1621	1133-1561	931-1406	871-1109	811-858	1382-1703

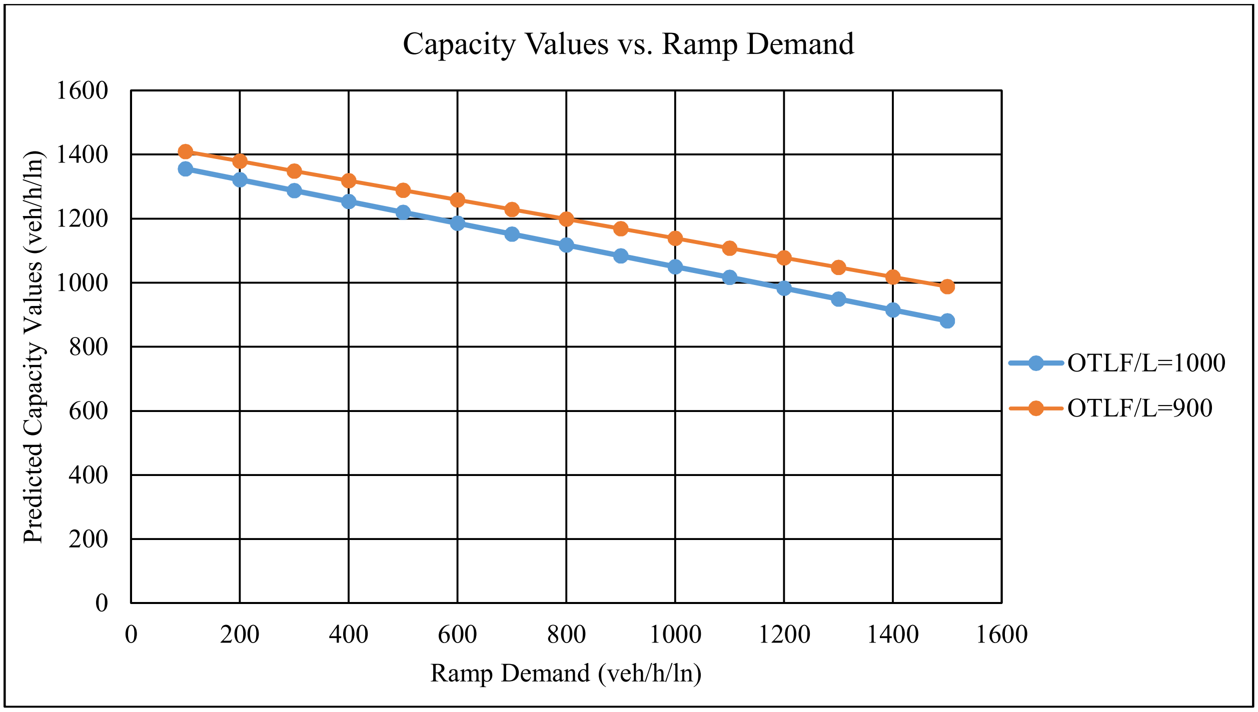

Figure 29: Relationship graph between predicted capacity values and ramp demand for D=3000 and N=3 and different proportions for OTLF and RLF2

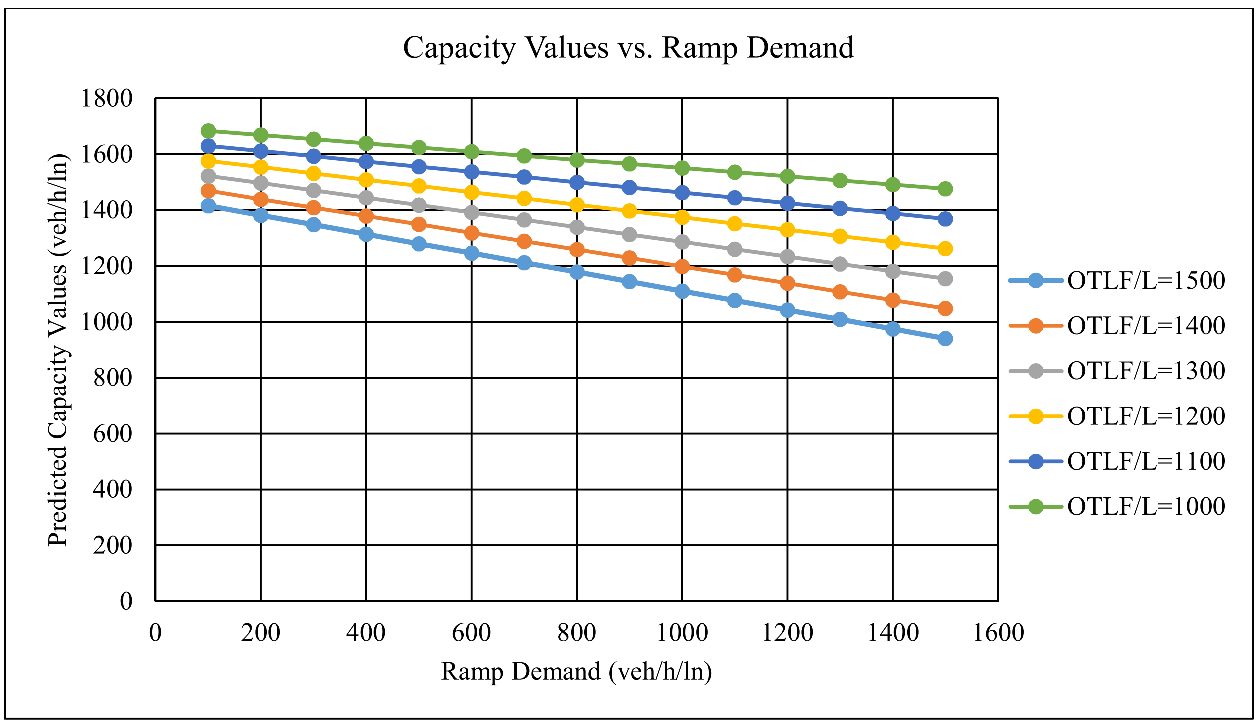

Figure 30: Relationship graph between predicted capacity values and ramp demand for D=4000 and N=3 and different proportions for OTLF and RLF2

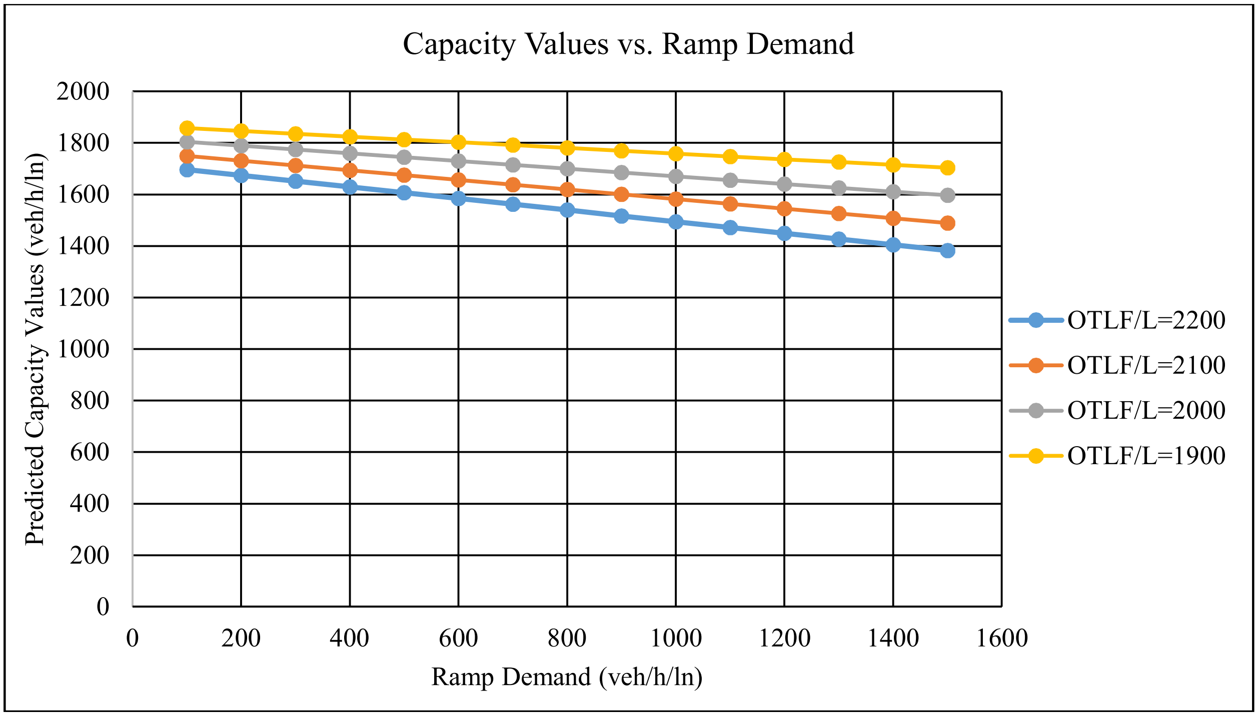

Figure 31: Relationship graph between predicted capacity values and ramp demand for D=6000 and N=3 and different proportions for OTLF and RLF2

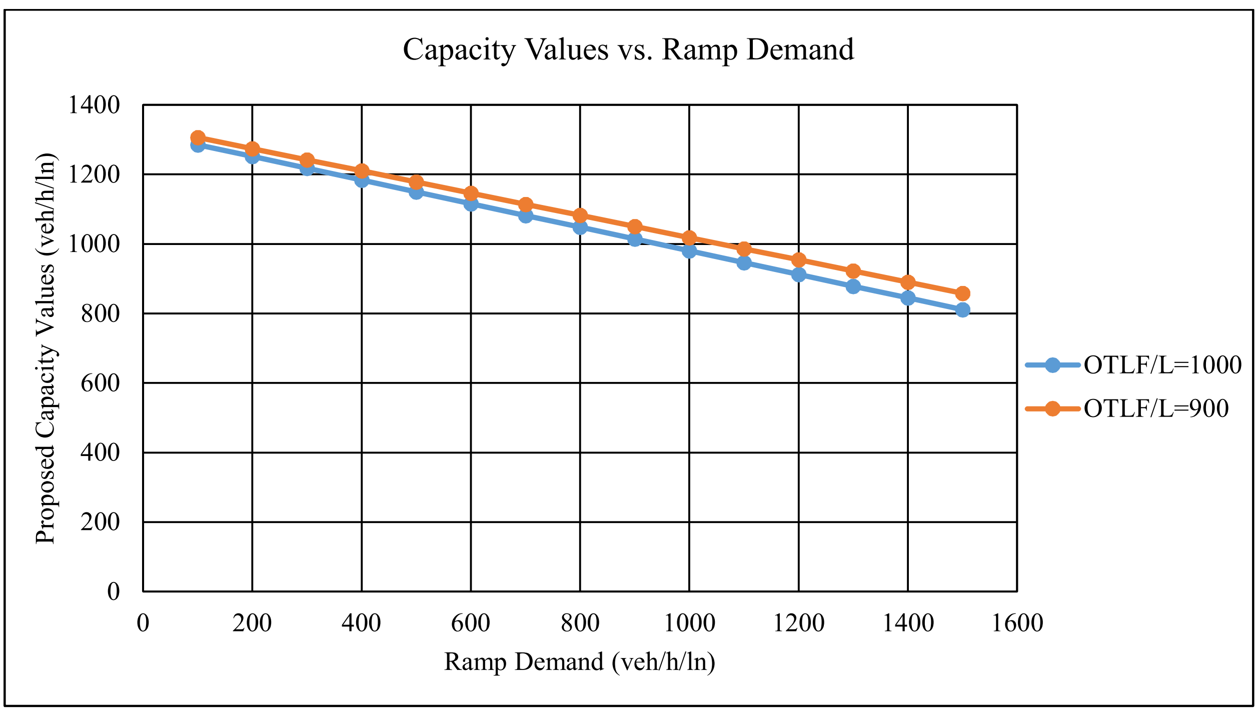

Figure 32: Relationship graph between predicted capacity values and ramp demand for D=4000 and N=4 and different proportions for OTLF and RLF2

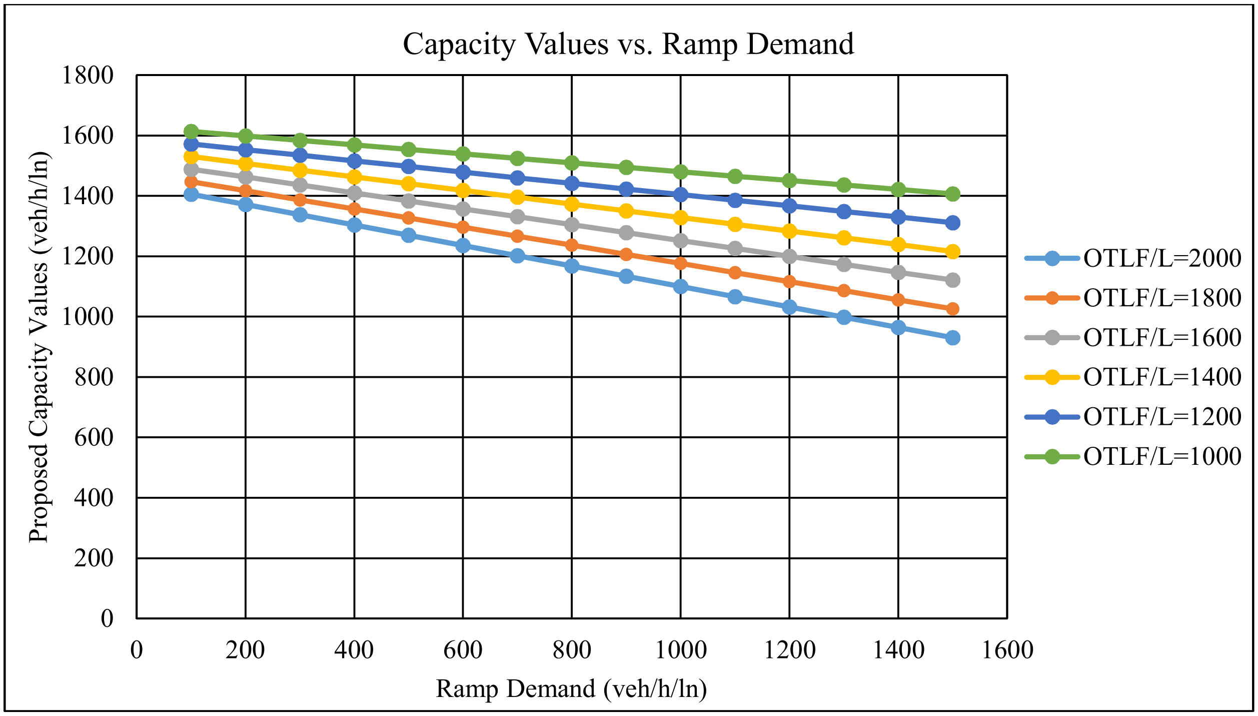

Figure 33: Relationship graph between predicted capacity values and ramp demand for D=6000 and N=4 and different proportions for OTLF and RLF2

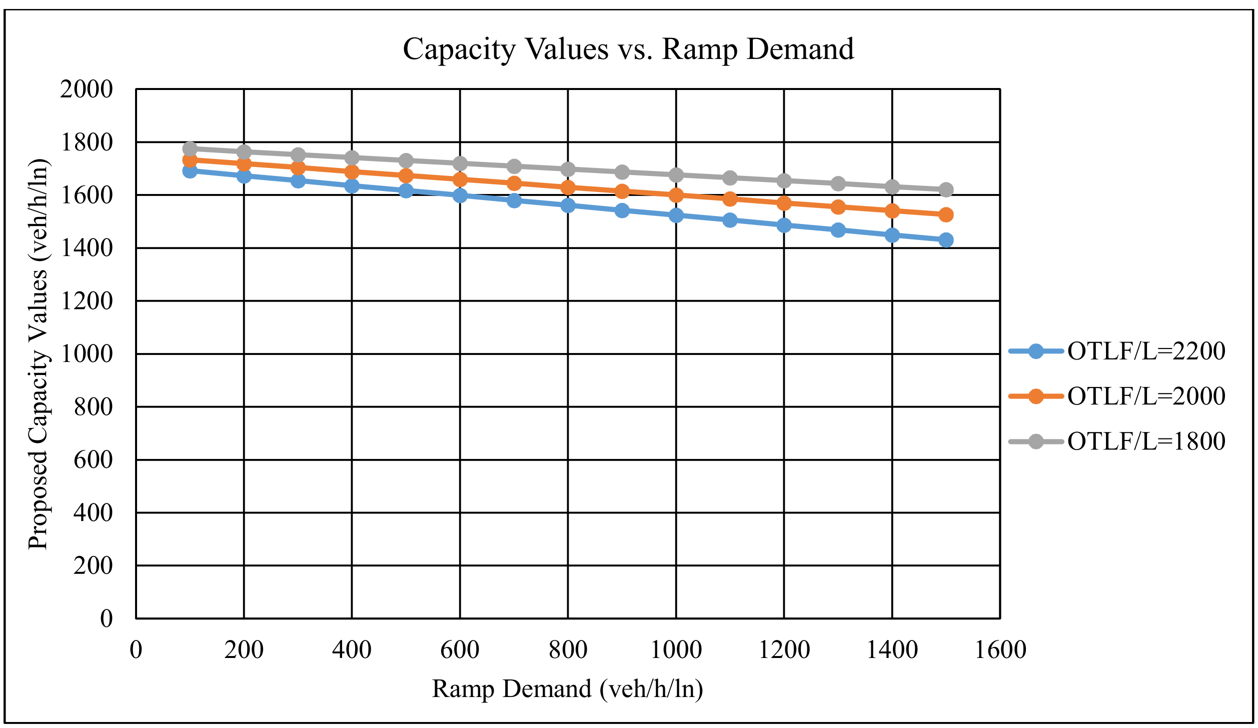

Figure 34: Relationship graph between predicted capacity values and ramp demand for D=8000 and N=4 and different proportions for OTLF and RLF2

5.5 Limitations

Although the model developed in this research captures significant factors that affect capacity, it still has some limitations.

1. Free flow speed was considered to be a possible contributor for capacity. But while developing the final model it was observed that there was no correlation between the observed field capacity values and free flow speeds. This was assumed to be because of the small range of field measured FFS in the data collection sites.
2. Truck volume percentages were also assumed to affect capacity. However, truck volume data were not available in the KC Scout portal. Therefore, this variable was not considered for developing the model.
3. The model does not consider the impact of the ramp number of lanes on capacity. Given that there was only one such ramp location with two lanes, it was not possible to quantify this impact on the final capacity estimation model.

CHAPTER 6. CONCLUSION AND RECOMMENDATIONS

This chapter summarizes the results of this work and provides recommendation for future work.

6.1 Conclusions

This thesis developed capacity estimation model at freeway merge bottleneck locations in Kansas City area. The model was developed considering the data collected during the peak periods, from a period of 6 months, from January 1, 2016 to June 30, 2016, at six different merge bottleneck locations. Days with bad detectors data, as well as days with incidents and bad weather were removed from the analysis.

A regression model was developed to give recommended capacity values for merge bottleneck locations. The model was developed considering the impacts of ramp breakdown flow, remaining lanes breakdown flow (flow of freeway, excluding the flow of shoulder lanes), number of lanes, and the interaction between ramp demand and shoulder lane flow. All the variables retained in the model were significant at 95% confidence level. The R^2 value for this model was 0.689, which is considered significant, and hence the model will be good predictor of the capacity values. Free flow speed was expected to be a contributor to the model but as it did not show any correlation with field capacity values it was removed from the model. Truck percentages were also considered as one of the contributing factors but were not included in the model as truck volumes were not available. The final model, presented as Equation 9, is also shown below.

$$\boldsymbol{Capacity = 1170.58 - 0.53RD + 0.31RLF2 + 0.12\mathrm{OTLF} - 70.06N +}$$
$$\boldsymbol{+ 0.19(RD \times OTLF)}$$

This model suggests that the ramp demand negatively affects the capacity at merge bottleneck locations. This indicates that, even if the total demand (sum of freeway demand and ramp demand) is the same, a higher proportion of ramp demand will lead to reduced capacity near merge locations. Some recommendations for capacity values at merge bottleneck locations at different on-ramp flows, freeway flows and number of lanes were presented in section 5.4. It is also observed that, as the outer two lane flow (influence area flow) increases there is a decrease in capacity, which is reasonable considering the increasing interaction between ramp vehicles and freeway vehicles. In addition, it is also observed that as the number of lanes increases the capacity per lane decreases, which is consistent with the past research (Lu & Elefteriadou, 2013; Kondyli et al., 2016).

6.2 Recommendations and Future Research

The model developed in this research was based on data from the Interstate network in the Kansas City area. As such, it captures macroscopic impacts of driver behavior and driver interactions in the specific area. Therefore, it cannot be said that this model can be used universally for all the bottleneck locations throughout the country. Further research has to be done to evaluate whether the contributing factors for capacity at merge bottlenecks remain the same or change, depending upon the location.

The analysis for this research was only restricted to merge junctions. The contributing factors for the capacity might remain same or alter at different bottleneck segments. Hence, similar analysis must be done at other bottleneck segments i.e., weaving and diverging segments.

The HCM 2016 states that the capacity is a function of speed. Because of the small range of free flow speeds available a weak correlation was observed between free flow speed and capacities and hence final model was developed without using FFS as a variable. It is recommended to gather data from a wide range of locations with different free flow speeds and check for its contributions to capacity. It is also recommended to gather truck volumes and check whether it is a significant factor for capacity. As the truck volumes were not present, the capacity values were not converted into equivalent volumes of passenger cars.

The capacity model developed only considered the three lane and four lane merge junctions. Future research should also look at merge junctions with two, five and six lanes for enhancing the model.

The freeway merge segment methodology in HCM 2016 use capacity values ranging from 2,400 passenger cars per hour per lane (pc/h/ln) for FFS 70 to 75 mi/h, to 2,250 pc/h/ln for FFS 55 mi/h. It does not consider variations in capacity due to the variations in ramp demand. The capacity model developed in this research can be considered as capacity estimation model while designing merge bottleneck locations, or for evaluating the operational efficiency of these segments.

It is also necessary to obtain data at merge junctions with two on-ramp lanes, and assess if the ramp number of lanes affects the merge capacity.

As this model was exclusively developed using the data gather from the Kansas City freeway network, Kansas Department of Transportation can use this model to estimate capacity for designing merge junctions in the Kansas City area.

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APPENDIX A

RECOMMENDED CAPACITY VALUES

Table 9: Capacity values based on ramp demand, number of lanes, outer two lane flows, and remaining lane flows for a 3 lane freeway merge segment

D (veh/h)	3000	3000	4000	4000	4000	4000	4000	4000	6000	6000	6000	6000
OTLF (veh/h/ln)	1000	900	1500	1400	1300	1200	1100	1000	2200	2100	2000	1900
RLF2 (veh/h/ln)	1000	1200	1000	1200	1400	1600	1800	2000	1600	1800	2000	2200
RD (veh/h/ln)	**Capacity (veh/h/ln)**											
100	1355	1409	1415	1469	1523	1576	1630	1684	1696	1750	1804	1857
200	1322	1379	1382	1439	1496	1554	1611	1669	1674	1731	1789	1846
300	1288	1349	1348	1409	1470	1531	1593	1654	1651	1713	1774	1835
400	1254	1319	1314	1379	1444	1509	1574	1639	1629	1694	1759	1824
500	1220	1289	1280	1349	1418	1487	1555	1624	1607	1675	1744	1813
600	1186	1259	1246	1319	1391	1464	1537	1610	1584	1657	1730	1802
700	1152	1229	1212	1289	1365	1442	1518	1595	1562	1638	1715	1791
800	1118	1199	1178	1259	1339	1419	1500	1580	1539	1620	1700	1780
900	1084	1168	1144	1228	1313	1397	1481	1565	1517	1601	1685	1769
1000	1050	1138	1110	1198	1286	1374	1462	1550	1494	1582	1670	1758
1100	1016	1108	1076	1168	1260	1352	1444	1536	1472	1564	1656	1747
1200	983	1078	1043	1138	1234	1330	1425	1521	1450	1545	1641	1736
1300	949	1048	1009	1108	1208	1307	1407	1506	1427	1527	1626	1725
1400	915	1018	975	1078	1181	1285	1388	1491	1405	1508	1611	1714
1500	881	988	941	1048	1155	1262	1369	1476	1382	1489	1596	1703

Table 10: Capacity values based on ramp demand, number of lanes, outer two lane flows, and remaining lane flows for a 4 lane freeway merge segment

D (veh/h/ln)	4000	4000	5000	5000	5000	5000	5000	6000	6000	6000	6000	6000	6000
OTLF (veh/h/ln)	1000	900	1400	1300	1200	1100	1000	2000	1900	1800	1700	1600	1500
RLF2 (veh/h/ln)	1000	1100	1100	1200	1300	1400	1500	1000	1100	1200	1300	1400	1500
RD	**Capacity (veh/h/ln)**												
100	1285	1306	1366	1387	1408	1429	1449	1405	1426	1447	1468	1489	1509
200	1252	1274	1334	1357	1380	1402	1425	1372	1394	1417	1440	1462	1485
300	1218	1242	1302	1327	1352	1376	1401	1338	1362	1387	1412	1436	1461
400	1184	1210	1270	1297	1323	1350	1376	1304	1330	1357	1383	1410	1436
500	1150	1178	1238	1267	1295	1324	1352	1270	1298	1327	1355	1384	1412
600	1116	1146	1206	1237	1267	1297	1328	1236	1266	1297	1327	1357	1388
700	1082	1114	1174	1207	1239	1271	1303	1202	1234	1267	1299	1331	1363
800	1048	1082	1142	1176	1211	1245	1279	1168	1202	1236	1271	1305	1339
900	1014	1050	1110	1146	1182	1219	1255	1134	1170	1206	1242	1279	1315
1000	980	1018	1078	1116	1154	1192	1230	1100	1138	1176	1214	1252	1290
1100	946	986	1046	1086	1126	1166	1206	1066	1106	1146	1186	1226	1266
1200	913	954	1014	1056	1098	1140	1182	1033	1074	1116	1158	1200	1242
1300	879	922	982	1026	1070	1114	1157	999	1042	1086	1130	1174	1217
1400	845	890	950	996	1042	1087	1133	965	1010	1056	1102	1147	1193
1500	811	858	918	966	1013	1061	1109	931	978	1026	1073	1121	1169

Table 10: Continued

D	6000	6000	6000	6000	6000	7000	7000	7000	7000	7000	7000	7000	7000	7000
OTLF/Lane	1400	1300	1200	1100	1000	2200	2100	2000	1900	1800	1700	1600	1500	1400
RLF2/Lane	1600	1700	1800	1900	2000	1300	1400	1500	1600	1700	1800	1900	2000	2100
RD	**Capacity (veh/h/ln)**													
100	1530	1551	1572	1593	1614	1528	1549	1569	1590	1611	1632	1653	1674	1694
200	1508	1531	1553	1576	1599	1500	1522	1545	1568	1591	1613	1636	1659	1681
300	1485	1510	1535	1559	1584	1472	1496	1521	1545	1570	1595	1619	1644	1669
400	1463	1490	1516	1543	1569	1443	1470	1496	1523	1550	1576	1603	1629	1656
500	1441	1469	1497	1526	1554	1415	1444	1472	1501	1529	1557	1586	1614	1643
600	1418	1448	1479	1509	1540	1387	1417	1448	1478	1508	1539	1569	1600	1630
700	1396	1428	1460	1492	1525	1359	1391	1423	1456	1488	1520	1552	1585	1617
800	1373	1407	1442	1476	1510	1331	1365	1399	1433	1467	1502	1536	1570	1604
900	1351	1387	1423	1459	1495	1302	1339	1375	1411	1447	1483	1519	1555	1591
1000	1328	1366	1404	1442	1480	1274	1312	1350	1388	1426	1464	1502	1540	1578
1100	1306	1346	1386	1426	1466	1246	1286	1326	1366	1406	1446	1486	1526	1565
1200	1283	1325	1367	1409	1451	1218	1260	1302	1343	1385	1427	1469	1511	1553
1300	1261	1305	1348	1392	1436	1190	1234	1277	1321	1365	1408	1452	1496	1540
1400	1239	1284	1330	1375	1421	1162	1207	1253	1299	1344	1390	1435	1481	1527
1500	1216	1264	1311	1359	1406	1133	1181	1229	1276	1324	1371	1419	1466	1514

Table 10: Continued

D	7000	8000	8000	8000	8000	8000
OTLF/Lane	1300	2200	2100	2000	1900	1800
RLF2/Lane	2200	1800	1900	2000	2100	2200
RD	**Capacity (veh/h/ln)**					
100	1715	1692	1713	1734	1754	1775
200	1704	1673	1696	1719	1741	1764
300	1693	1655	1679	1704	1729	1753
400	1682	1636	1663	1689	1716	1742
500	1671	1617	1646	1674	1703	1731
600	1660	1599	1629	1660	1690	1720
700	1649	1580	1612	1645	1677	1709
800	1638	1562	1596	1630	1664	1698
900	1627	1543	1579	1615	1651	1687
1000	1616	1524	1562	1600	1638	1676
1100	1605	1506	1546	1586	1625	1665
1200	1594	1487	1529	1571	1613	1654
1300	1583	1468	1512	1556	1600	1643
1400	1572	1450	1495	1541	1587	1632
1500	1561	1431	1479	1526	1574	1621