CAPACITY ESTIMATION OF TWO-SIDED TYPE C WEAVES ON FREEWAYS

by

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ABSTRACT

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A weaving section is a common design on major highway facilities that has been an interest to researchers for years. Weaving areas are characterized by frequent lane changes, which significantly reduce the capacity of the freeway system. The 2000 Highway Capacity Manual (HCM) defined weaving capacity as “any combination of flows that causes the density to reach the LOS E/F boundary condition of 43 pcpmpl for freeways” based on configuration, number of lanes in the weaving section, free-flow speed, length of the weave, and volume ratio (VR). However, no research has been focused on the two-sided type C weaves, which is the most difficult weave to maneuver, where one weaving movement must cross all the freeway lanes.

Simulation is the key element for this research and the VISSIM model was calibrated using the data collected in San Antonio to make sure that the model behaves the same as the observed traffic in the field. Moreover, distributions of vehicles on each lane and/or the lane changing patterns of traffic and/or gap characteristics within the weaving area were included in the simulation model. The regression model described in this dissertation is not a speed-based model, as is the model in the current HCM, but is developed to predict capacity of two-sided type C weave for the entire range of flow combinations based on simulation runs. The result found in this research is a reliable technique for evaluating a weaving performance and should be easy to explain to practitioners.

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CHAPTER 1

INTRODUCTION

1.1 Background

Traffic congestion on freeway systems is a significant concern in urban areas throughout the United State of America. Building new freeways to reduce congestion is not feasible due to the high capital and social cost. Thus, the effective management and operation of existing freeway facilities has become a preferred approach to reduce traffic congestion. Weaving areas are common design elements in urban freeway systems. They are defined in the 2000 edition of the Highway Capacity Manual (HCM) as “the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway without the aid of traffic control devices.” [Ref. 5] Weaving areas are characterized by frequent lane changes, which significantly reduce the capacity of the freeway system.

Weaving areas, categorized by their lane configuration, consist of three kinds: Type A, Type B, and Type C. The 2000 HCM defines a Type A weaving are by two conditions: non-weaving vehicles do not change lanes, and all weaving vehicles must make at least one lane change. Thus, there is a continuous lane line from the point of the merge gore to the point of the exit gore, across which only weaving vehicle must cross. There are two sub-categories of the Type A weaves: Type A Major and Type A

Minor, shown in Figure 1.1. Type A Major weaves are used for freeway-to-freeway applications where there are two or more lanes on all entering and exiting roadways. On the other hand, Type A Minor weaves represent ramp weaves, where the entering and exiting roadways contain only single lane, as with a freeway entrance ramp and an exit ramp connected by an auxiliary lane. If there is no auxiliary lane, it is a ramp merge followed by a diverge and not a weaving area.

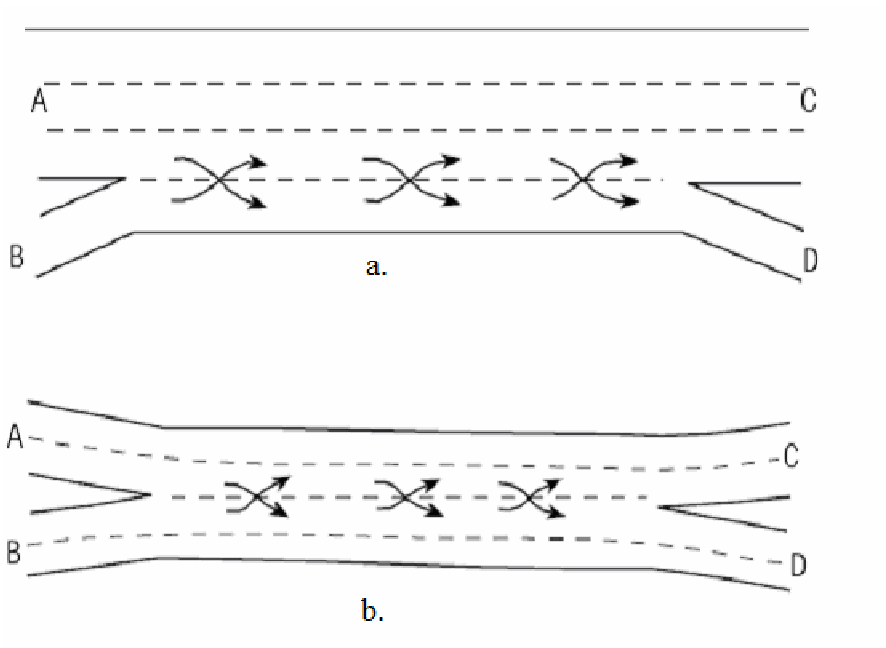

Figure 1.1 Type A Weaving Segments [Ref. 5]
a. Ramp-Weave; b. Major Weave

Types B and C weaves are characterized by having one of the lanes entering from the right roadway leave to the left, or by having one of the lanes entering from the left roadway exit to the right. Thus, not all traffic that weaves must change lanes. Type B weaves include at least three entry and exit legs with multiple lanes, and their lane changing should satisfy two following conditions: One weaving movement can be made without making any lane changes and the other weaving movement requires at least one lane change. The larger weaving movement is assumed to be the one that does not change lanes. Three basic Type B weaves are shown in Figure 1.2.

It should be noted that internal merges are shown in figure 1.2 b and c. These are not considered good design but are included in the HCM to allow analysis of existing freeway geometries.

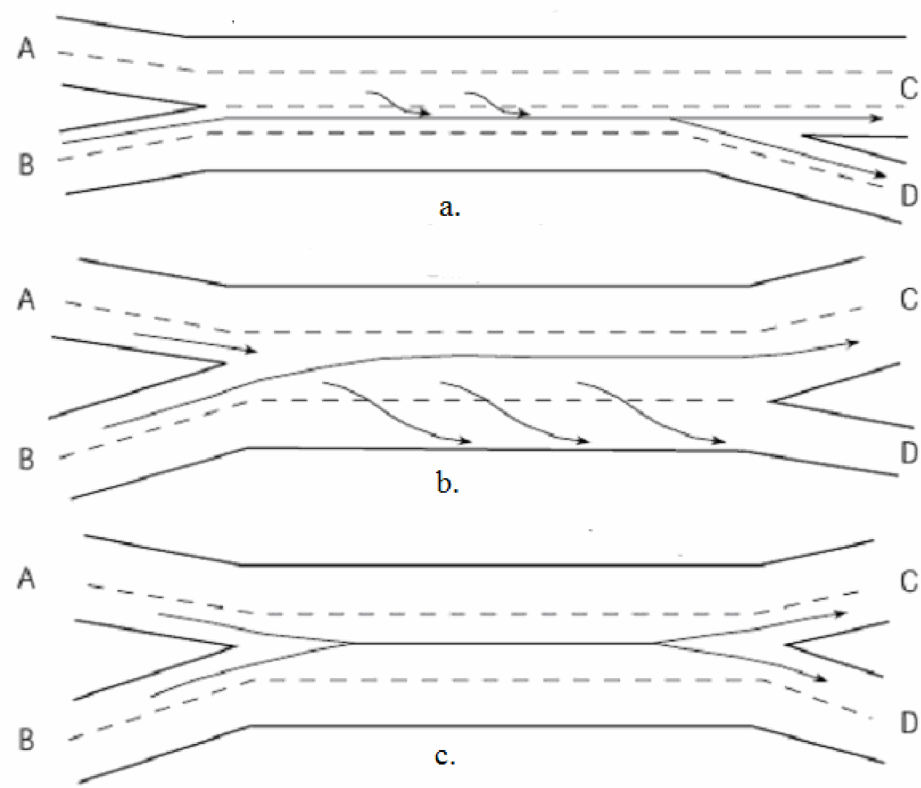

Figure 1.2 Type B Weaving Segments [Ref. 5]
a. Major Weave with Lane Balance at Exit Ramp; b. Major Weave with Merge at Entry Gore; c. Major Weave with Merge at Entry Gore and Lance Balance at Exit Gore

In Type C weaves, the traffic weaving one way does not necessarily have to change lanes while the traffic weaving the other way has to change at least two lanes (see Figure 1.3a). A final special case of Type C weaves is the two-sided weave, formed when a right-hand on-ramp is followed by a left-hand off-ramp, or vice versa (see Figure 1.3b). Again, the larger weaving movement is assumed to be the one not changing lanes. In this case, the through freeway flow operates functionally as a weaving flow. Ramp-to-ramp vehicles must cross all freeway lanes to finish their desired maneuver.

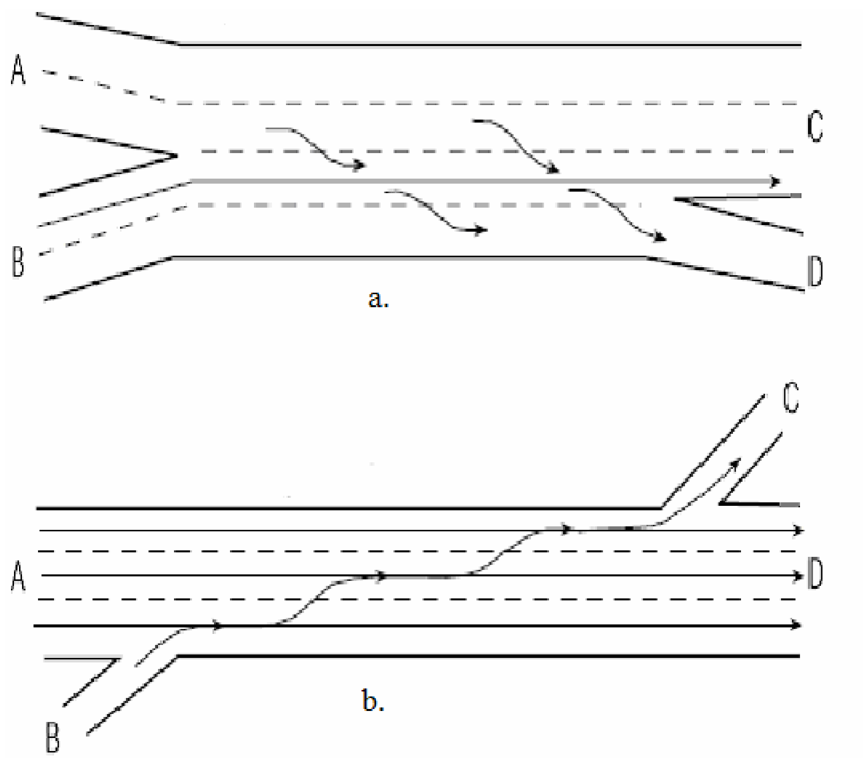

Figure 1.3 Type C weaving segments [Ref. 5]
a. Major Weave Without Lane Balance or Merging; b. Two-Sided Weave

Typically, a weaving section always has four flows: freeway to freeway (F-F), freeway to ramp (F-R), ramp to freeway (R-F), and ramp to ramp (R-R). In a two-sided Type C Weave (figure 1.3 b), weaving flows are F-F (A to D) and R-R (B to C). Non-weaving flows in a two-sided Type C Weave are F-R (A to C) and R-F (B to D).

1.2 Problems Associated with Existing Technique

Most traditional weaving analysis methods use roadway geometry and traffic volumes as inputs, and provide an estimate of speed as an output. The use of speed for assessing the capacity of weaving areas has proven to be a poor choice. Previous studies from California showed that average speed appears to be rather insensitive to flow up to 1,600 passenger cars per hour per lane (pcphpl) [Ref. 30]. Initial observations suggested that speed might be the result of many factors, only one of which is proximity to capacity. Throughout the typical weaving zone, speed may vary dramatically, and characterizing speed does not consider the motorist's perception of acceptable operation in a weaving zone. These problems have been noted in other areas of highway capacity, and the trend in level of service calculation has been toward using density as a service measure. Density is the result of both speed and flow, and therefore acts as a single measure of congestion.

In addition, an even more significant problem associated with existing speed-prediction procedures is the determination of capacity. Typically, capacity is defined as the maximum flow in which vehicles can be reasonably expected to travel through a given facility. However, the maximum number of vehicles which can traverse the facility will vary with different combinations of the four traffic movements. As a result, the existing speed-prediction procedures do not directly compute weaving area capacity [Ref. 30].

Another problem with existing methods has been that they are difficult to justify or explain to practitioners. They depend on regression analysis to fit complex curves to widely scattered data points usually resulting in usually poor correlation and physical dimensions taken to unrealistic fractional powers [Ref. 26].

1.3 Proposed Research Approach

Existing speed prediction procedures cannot estimate the operational performance of freeway weave sections. As a result, the objective of this research is to develop new methodologies to estimate the capacity of the two-sided Type C weave on freeways. Since the two-sided Type C weave is the most difficult weave to maneuver, where one weaving movement must cross all the freeway lanes, all factors potentially affecting the performance of the weaving zone will be considered in the model. The flows in the weaving and non-weaving traffic streams create conflicts in a weaving zone; these conflicts will be considered in the model. Moreover, distributions in vehicles on each lane and/or the lane changing patterns of traffic and/or gap characteristics within the weaving area will be included in the model. The location of the field study for this research was southbound I-35/410 between the Rittiman entrance and southbound I-410 exit in San Antonio, Texas.

Simulation is a key element of this research and VISSIM was selected because of its capability. VISSIM is a microscopic, time-step-and-behavior based simulation model developed to model urban traffic. In contrast to less complex simulation software using constant speeds and deterministic car following logic, VISSIM uses a psycho-physical driver behavior model. In addition, VISSIM simulates traffic flow by moving "driver-vehicle-units" through a network. Every driver with his/her specific behavior characteristic is assigned to a specific vehicle. As a consequence, the driver behavior corresponds to the technical capabilities of his/her vehicle. Core algorithms are well documented; moreover, the open interfaces provide compatibility with external software. VISSIM has become widely used by the intensive research and a large user community worldwide since it was distributed to the market in 1992 [Ref. 29].

In the remainder of this dissertation, the literature review of previous studies related to weaving areas is presented in chapter 2. Chapter 3 describes the collection of field data in San Antonio. In chapter 4, a VISSIM overview is provided, followed by the VISSIM inputs and outputs needed for this research. Chapter 5 discusses the calibration and validation of the simulation model. Chapter 6 presents simulation runs and results for this research. In addition, a regression model for estimating the capacity of two-sided Type C weaves is developed in this chapter. In addition, the density obtained from simulation outputs is compared with the values presented in the 2000 HCM at the end of chapter 6. Chapter 7 examines the regression model to see how well this model can predict the capacity of a weaving section with various numbers of lanes in the mainlane. Chapter 8 presents the conclusions and recommendations for this research. The appendixes provide a comprehensive overview of the simulation runs made for this research.

CHAPTER 2

LITERATURE REVIEW

An extensive literature review related to weaving area is presented in this chapter. Weaving areas can be found on all types of highway facilities, ranging from freeways to arterials. The current procedure in the 2000 Highway Capacity Manual (HCM) was developed for weaving areas on freeways, and little has been done for other facility types. This literature review begins with a history of weaving procedures as prescribed in the various editions of the Highway Capacity Manual and related documents. Next, other freeway models are reviewed.

2.1 Weaving Areas by the Book

The 1950 edition of the Highway Capacity Manual [Ref. 1] defined *weaving* as "the act performed by a vehicle moving obliquely from one lane to another, thus crossing the path of other vehicles moving in the same direction." Further, a *weaving section* was defined as "a length of one-way roadway serving as an elongated intersection of two one-way roads crossing each other at an acute angle in such a manner that the interference between cross traffic is minimized through substitution of weaving for direct crossing of vehicle pathways."

Traffic in the weaving section was divided into weaving and non-weaving flows, and only Type A configurations were considered. The weaving capacity (i.e., the maximum number of weaving vehicles) was then taken to be equivalent to the flow in a single lane (since the number of vehicles crossing the crown line could be no greater than the number that could crowd into a single lane). For very short weaving sections (less than 100 feet), the capacity was 1200 vehicles per hour, with many vehicles stopping before entering the weaving section. A 900-foot weaving section could accommodate about 1500 passenger cars per hour at about 40 mph, and a 450-foot weaving section could accommodate the same number at 30 mph. These flows represent *possible* capacities for the weaving traffic. The 1950 HCM recommended that lanes be added to each side of the weave to fully accommodate the non-weaving traffic.

The design and analysis tools for weaving sections consisted primarily of a single graph relating the length of the weaving section (defined as the distance between the merge and diverge gores), the sum of the weaving flows (up to 3500 vehicles per hour), and the operating speed of the weaving section (see Figure 2.1). A weaving section length of up to 3400 feet is indicated in the figure. An equation was provided to estimate the number of lanes required for the weaving section, which divided the sum of the non-weaving traffic, the larger weaving flow, and three times the smaller weaving flow by the single lane capacity on the approach and exit roadways.

$$N = \frac{W_1 + 3W_2 + F_1 + F_2}{C} \tag{2.1}$$

where N= number of lanes

W_1= larger weaving volume (vph)

W_2= smaller weaving volume (vph)

F_1, F_2= non-weaving volumes (vph)

C= normal uninterrupted flow capacity for approach and exit roadways (vphpl)

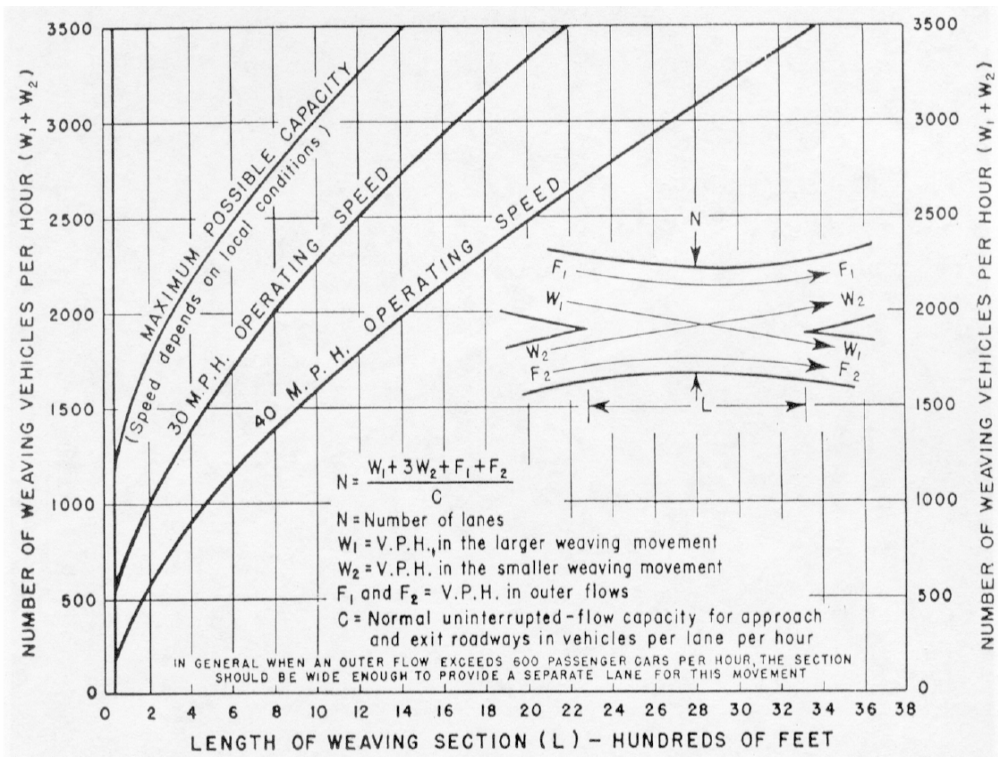

Figure 2.1 Weaving Analysis, 1950 Highway Capacity Manual [Ref. 1]

The development of the graph and equation was based on data from six sites: four on the roadways surrounding the Pentagon in Arlington, Virginia, and two on the San Francisco Bay Bridge distribution system. The writers also recognized that, at high weaving volumes, some weaving traffic would have to use the lanes adjacent to the lanes on both sides of the crown line, creating a *compound weaving section* and requiring a longer weaving section. As a practical matter, though the 1950 HCM recommended that weaving sections should only be used when the two approach roadways each carried “less than the normal capacity of two lanes of a one-way roadway and the total number of vehicles required to weave [did] not exceed 1500 per hour.” [Ref. 1] This volume restriction applied when the two entering roadways each consisted of a single lane. Additional lanes could be added to accommodate non-weaving flows.

The 1965 edition of the Highway Capacity Manual [Ref. 2] defined a *weaving section* as “a length of one-way roadway at one end of which two one-way roadways merge and at the other end of which they separate. A multiple weaving section involves more than two entrance and/or exit roadways.” The basic design and analysis tools from the 1950 HCM were carried over into the new HCM, but considerably amplified with additional data (33 observations at 27 sites are listed in the appendix, and are selected from the 1963 BPR urban weaving area capacity study). Lane configuration is not specifically considered; all sketches showing lane lines are Type A, but the crown line is defined as a real or imaginary line connecting the merge and diverge gores. The weaving methodology is considered applicable to simple and multiple weaves, as well as one- and two-sided weaving, although the reader is referred to the material in the chapter on ramps when one-sided weaving is formed by an entrance ramp followed by an exit ramp.

The length of the weaving section is revised to begin at a point before the merge gore where the roadways are two feet apart to end at a point after the diverge gore where the roadways are twelve feet apart.

The equation for estimating the required width (number of lanes) was modified from the previous edition by (1) dividing the sum of volumes by a service volume (corresponding to a specific level of service on a basic freeway segment) rather than the single lane capacity and (2) allowing the multiplier of the smaller weaving volume (in the numerator and designated *k*) to vary from one to three rather than defining it as three.

$$N = \frac{V_{w1} + kV_{w2} + V_{01} + V_{02}}{SV} \tag{2.2}$$

where N= number of lanes

V_{w1}= larger weaving volume (vph)

V_{w2}= smaller weaving volume (vph)

V_{01}, V_{02}= non-weaving (outer) volumes (vph)

SV= appropriate service volume or capacity on approach and exit roadways (vphpl)

k= weaving influence factor

The concept of quality of flow (distinct from the level of service) was introduced, and five levels were identified, designated as I, II, III, IV, and V. Maximum lane service volume ranged from 2000 passenger cars per hour for level I down to 1600 passenger cars per hour for level V. The quality of flow was related to levels of service, but differed by the type of facility (freeways and multilane rural highways, two-lane rural highways, and urban and suburban arterials).

The graph relating weaving section length to the total weaving volume used the quality of flow rather than operating speed as in the 1950 HCM (see Figure 2.2). However, levels III, IV, and V correspond to speeds of (approximately) 40, 30, and 20 mph, and the lines corresponding to these three levels are very similar to those in the earlier edition. The width calculation sets k equal to 3 for quality of flow levels III, IV, and V, as in the earlier edition. Smaller values of k, corresponding to quality of flow levels I and II, result in longer and narrower weaving sections.

The third edition of the Highway Capacity Manual [Ref. 3] was initially published in 1985 and reflected the extensive research on weaving areas conducted since the release of the 1965 HCM. Weaving was defined as "the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway, without the aid of traffic control devices. Weaving areas are formed when a merge area is closely followed by a diverge area, or when an on-ramp is closely followed by an off-ramp and the two are joined by an auxiliary lane." Weaving areas are defined in terms of three principal geometric characteristics: weaving length (defined as in the 1965 HCM), lane configuration (relative placement and number of entry and exit lanes, generalized to three types), and width (number of lanes).

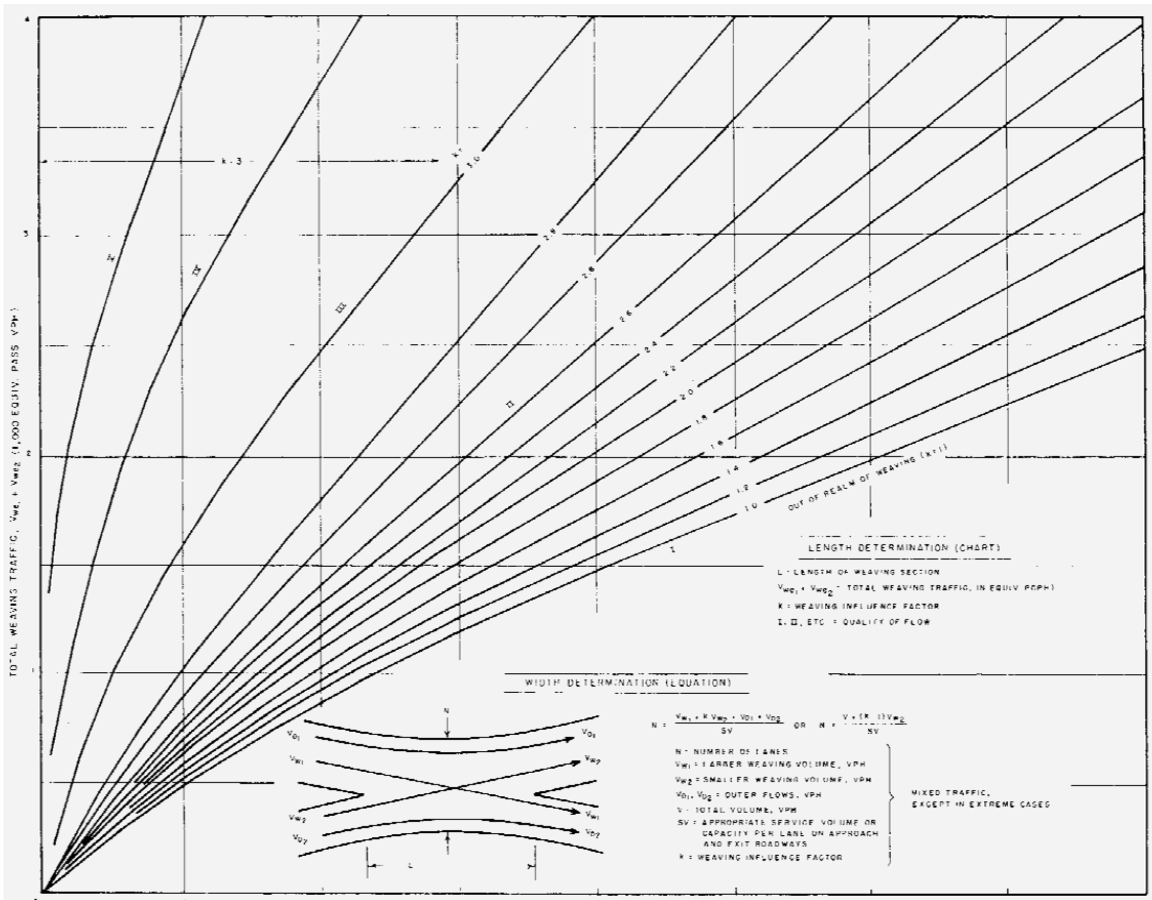

Figure 2.2 Weaving Analysis, 1965 Highway Capacity Manual [Ref. 2]

The two major components of the weaving analysis procedure were a model to predict the speed of weaving and non-weaving vehicles for a given weaving length, width and volume conditions, and a model to determine if the weaving traffic in a particular weaving area is constrained by its geometry. The speed estimation equation (eq. 2.3) was developed through regression techniques on over 207 observations, with different regression constants for twelve categories (weaving and non-weaving traffic, configuration, and unconstrained and constrained operation) [Ref. 3]. The same equation was used for weaving and non-weaving speeds, with unconstrained operation initially assumed [Ref. 3].

$$S_w or S_{nw} = 15 + \frac{50}{1 + a(1+VR)^b (v/N)^c / L^d} \qquad (2.3)$$

where S_w= average running speed for weaving traffic (mph)

S_{nw}= average running speed for non-weaving traffic (mph)

VR= volume ratio (fraction of traffic weaving)

v= total flow in weaving area (passenger cars per hour)

N= number of lanes in the weaving area

L= length of weaving area (feet)

a, b, c, d= regression constants

Traffic was deemed constrained if the estimated proportion of the width used by the weaving traffic exceeded a specified maximum for each configuration [Ref. 3].

$$Type\ A: N_w = 2.19 N\, VR^{0.571}\ L_H^{\ 0.234} / S_w^{\ 0.438}$$

$$Type\ B: N_w = N[0.085 + 0.703VR + (234.8/L) - 0.018(S_{nw} - S_w)] \qquad (2.4)$$

$$Type\ C: N_w = N[0.761 - 0.011L_H - 0.005(S_{nw} - S_w)] + 0.047VR$$

where N_w= number of lanes required for weaving

N= number of lanes in the weaving area

VR= volume ratio (weaving volume to total volume)

L= weaving length (feet)

L_H= weaving length (stations)

S_w, S_{nw}= weaving and non-weaving speed (mph)

If N_w exceeded specified maximums (1.4, 3.5, or 3.0 lanes for Types A, B, or C, respectively), the weaving section was considered constrained, and the speeds were re-estimated appropriately. Finally, levels of service for weaving and non-weaving flows were directly related to the predicted speeds.

While this procedure represented a major improvement, particularly with the explicit consideration of the lane configuration, which has a major effect on the number of required lane changes in the weaving area, its use is awkward in design. Both the model for speed estimation and the test for constrained operation require an assumption of the weaving area's length, width, and configuration, thus requiring a trial-and-error approach in design. When the revised edition of the 1985 HCM was released in 1994 [Ref. 4], the weaving area chapter was unchanged.

The fourth edition of the Highway Capacity Manual [Ref. 5] was published in 2000. In this edition, the definition of the weaving section is unchanged from that in the third edition. Also, "the heart of weaving analysis procedure is the prediction of space mean speeds of weaving and non-weaving flows within the weaving segment." [Ref. 5] However, the new equation estimated average weaving and non-weaving speed is introduced (eq. 2.5), with unconstrained operation initially assumed. This assumption is later tested, and speeds are recomputed if operations turn out to be constrained. Essentially the same equation as in the 1985 & 1994 edition; however, free flow speed is directly inserted instead of assumed to be 60 mph. Also, the model was re-estimated resulting with different values for a, b, c, and d.

$$S_i = 15 + \frac{S_{FF} - 10}{1 + W_i} \tag{2.5}$$

where S_i= average speed of weaving (i=w) or non-weaving (i=nw) vehicles (mph),

S_{FF}= is the average free-flow speed of the freeway segments entering and leaving the weaving segment (mph),

W_i= weaving intensity factor for weaving (i=w) and non-weaving (i=nw) flows.

$$W_i = \frac{a(1+VR)^b \left[\frac{v}{N}\right]^c}{L^d} \tag{2.6}$$

where

VR= volume ratio (weaving flow/total flow),

v= total flow rate in the weaving segment (pc/h),

N= total number of lanes in the weaving segment,

L= length of the weaving segment (ft),

a,b,c,d= constants of calibration.

Traffic was deemed constrained if the estimated proportion of the width used by the weaving traffic exceeded a specified maximum for each configuration [Ref. 5].

$$\begin{aligned}
&Type\ A: N_w = 0.74(N)VR^{0.571}\ L^{0.234} / S_w^{0.438} \\
&Type\ B: N_w = N[0.085 + 0.703VR + (234.8/L) - 0.018(S_{nw} - S_w)] \qquad (2.7) \\
&Type\ C: N_w = N[0.761 + 0.047VR - 0.00011L - 0.005(S_{nw} - S_w)]
\end{aligned}$$

where N_w= number of lanes required for weaving

N= number of lanes in the weaving area

VR= volume ratio (weaving volume to total volume)

L= weaving length (feet)

S_w, S_{nw}= weaving and non-weaving speed (mph)

Finally, the fourth edition of The Highway Capacity Manual introduced a set of tabulated values of weaving segment capacity, which is "any combination of flows that causes the density to reach the LOS E/F boundary condition of 43.0 pcpmpl for freeways or 40.0 pcpmpl for multilane highways." [Ref. 5] The value of weaving capacity depends on a number of variables: configuration (weaving type), number of lanes, free-flow speed, weaving length, and the volume ratio.

2.2 Other Weaving Studies for Freeways

The PINY method [Ref. 6] was developed at the Polytechnic Institute of New York using data collected at 17 northeastern sites. Four basic variables used in this model are number of lanes, weaving section length, traffic volumes, and speeds. Basically, this model is for operational analysis, which can be used to estimate the level of service for both weaving and non-weaving vehicles for a given set of geometry and traffic compositions. Level of service in this model is based firmly on the difference between the weaving and non-weaving speed. For design purposes, this model introduced a set of nomographs and equations to iteratively establish the appropriate number of lanes and weaving section length, given traffic volumes and desired level of service.

The Leisch method [Ref. 7] developed by Jack Leisch and Associates was designed to be used with the 1965 HCM or with Leisch's reformatting and expansion of the 1965 HCM [Ref. 8]. The Leisch technique uses a nomograph approach to establish an appropriate section length and number of lanes given traffic volumes and a desired level of service. Weaving volume, level of service, freeway volumes, and weaving ratio are the inputs to the nomographs. Even though the same data was used for both the PINY and the Leisch methods, they yielded substantially different results in many cases [Ref. 9] The Leisch method took into account certain characteristics related to the configuration of a subject weaving section.

The fact that the parameters a, b, c, and d in the HCM method are determined from the minimum number of lane shifts required by a driver in each of weaving streams implies weaving traffic is completely segregated on entering the weaving section traffic. However, field measurements collected indicate that weaving traffic is not fully segregated on entering the weaving section [Ref. 10]. As a result, Fazio [Ref. 10] introduced the lane shift concept as a refinement to the estimation of weaving and non-weaving speeds. A lane shift model was developed based on the number of lane changes for any one vehicle depending on how many lanes away from the crown line the vehicle entered the weaving section. The resulting speed models, which incorporated the lane shift in the speed estimation equation, represent an extension of the JHK and 1985 HCM models. Type A weaves only were used in this work; however, the use of lane shifts should be applicable to any configuration. Fazio also found that, when evaluating the 1985 HCM procedure, nearly half of the 67 cases in this study did not meet the model's specified constraints, indicating the degree to which the model is limited in its application.

Fazio [Ref. 11] also developed a speed model for a Type A weaving section with single lane entrance and exit ramps and a three-lane freeway. This model was based on lane position within the weaving section; then the lane number was directly incorporated into the model as a multiplicative factor (lane 1 was the auxiliary lane and lane 4 was the left lane). A multiple linear regression analysis was performed using average running speed by lane within the weaving section as the dependent variable and the equivalent peak passenger car flow rate, length, and lane with the weaving section as independent variables. This model can be used to determine the level of service (LOS) of traffic operations at existing weaving section if we have the length of weaving section and the lane volumes at the point of entry weaving section.

Cassidy, Skabardonis, and May [Ref. 12] applied regression analysis and classification and regression tree (CART) to identify basic relations between traffic characteristic and weaving section design. Data was collected from 8 different test sites in California. Then, six existing methods for design and analysis of freeway weaving sections were applied to all the data sets. Initial comparison results indicated that all methods typically predicted operating speeds slower than observed speeds, although they were quite variable. The JHK [Ref. 35] and 1985 HCM models were recalibrated for the California data, resulting in R^2 values of 0.09 for the JHK model and 0.25 and 0.44 for Type B and Type C weaving areas, respectively, with the 1985 HCM model.

The reestimated parameters of both models were quite different from the original parameters, particularly for the JHK model.

Cassidy, et al. [Ref. 12], also used the CART technique to test all the data set collected from eight test sites in order to gain deeper insights into factors influencing the operation of freeway weaving sections. Unsatisfactory results were also found from the CART analysis.

Also, speed versus v/c scatter plots for average weaving speed and average non-weaving speed were constructed using 5-min observed data. These plots indicated that speed is insensitive to flow up to v/c values of about 0.8. Moreover, an average weaving speed plot showed a high degree of scatter among data, and separate plots by individual location or by configuration type did little to decrease this variation. From these results and previous results found by Persaud and Hurdle [Ref. 13], Cassidy concluded that average travel speed is not an ideal measure of effectiveness for weaving sections.

Furthermore, Cassidy and May [Ref. 14, 15] proposed a new analytical technique for a freeway weaving section based on the lane distributions of four traffic streams through a weaving area. This new technique predicts the spatial distribution of traffic in each individual lane rather than predicting the average speeds of vehicle traveling in the weaving areas because freeway weaving operation may be mainly influenced by what is occurring in each individual lane. In addition, changes in vehicle distributions (within individual lanes) over some interval of length reflect lane-changing activity. As a result, both lane-changing and lane utilization becomes the two basic components of capacity of a given weaving area.

The INTRAS model was calibrated and used for predicting spatial distributions for each of the four traffic streams through graphs representing empirical and simulated data. The results from the simulation showed that capacity flow values are 2,200 pcph at any point within the weaving area and the maximum rate of lane changing are from 1,100 to 1,200 lane changes per hour over any 250-foot segment of weaving area. The primary limitation of this study was that the calibration test sites did not vary in their number of lanes; thus, this model works well for a weaving section with five or six lanes within the weaving area [Ref. 14].

Ostrom, Leiman, and May [Ref. 16] proposed computer program FREWEV, which can be used to design and analyze major freeway weaving. FREWEV is an implementation of the point flow by movement method, which "predicts the distribution of each movement throughout the section and estimates the total volume at a point as a sum of the individual movements." [Ref. 16] The point flow by movement method appears to work best for major weaves, while the total point flow method appears to work better for ramp weaves [Ref. 16].

Windover and May [Ref. 17] introduced a revised version of the Level D method, originally developed by Caltrans [Ref. 18]. This revised version of Level D is more accurate than the previous Level D method if we modify the Level D estimation of freeway-to-freeway (FF) percentage in the right most through lane. Currently, "the FF percentages used in the Level D methodology were determined to be consistently low during both calibration and validation stages." [Ref. 17] As a result, a computer model, FRELANE, was designed to analyze traffic performance in weaving sections based on the total point flow theory.

Harkey and Robertson [Ref. 19] wanted to find the relationship between curved freeway segment and weaving area by applying the 1985 HCM weaving methodology to two curved freeway sections in North Carolina. They conclude that the geometry influenced the traffic operation in the weaving section, but did not quantify it.

Pietrzyk and Perez [Ref. 20] calculated the weaving section length from the speed estimation equation in the 1985 HCM. From this work, the necessary weaving section length could be directly calculated for given conditions of the weaving section including the LOS. This work can also be applied to collector-distributor roadways, but LOS criteria have not been established off the freeway mainlanes.

Five basic weaving analysis methodologies (Ref. 2, Ref. 3, Ref. 6, Ref. 35, and Ref. 8) are compared in a study in Japan [Ref. 21]. The authors found the 1985 HCM and JHK to be the best methods while the PINY and Leisch methods underestimated the speeds. In addition, the 1965 HCM is inadequate for the particular weaving areas selected.

Alexiadis [Ref. 22] found that the average speed in weaving areas in Boston were higher than the speed predicted by the 1985 HCM weaving method. Data collected from 11 sites in Boston were used to re-calibrate the coefficients of the speed equation introduced in the 1985 HCM weaving method. The new model passed a t-test at a 95% confidence level. The author also collected data from two-sided Type C weaves on the Logan Airport access roads, which was used to estimate weaving speed and non-weaving speed equations. The (1+VR) term in the HCM was replaced with (v_{w2}/v), the crossing-weaving ratio, which leads to values much closer to the observed speeds than those predicted by the 1985 HCM equations.

Vermijs [Ref. 23] used simulation model FOSIM, developed at the Delft University of Technology in the early 1990s, to evaluate capacity for several type A major weaves and ramp weaves. Three basic factors, which have impacts on weaving area, were tested in this research: (1) weaving section length, (2) weaving flow, (3) traffic mix. The results showed that the capacity of the weaving section increased with the increasing of the length, up to a certain length. The length range from 400 to 1000 meters (1310 to 3280 feet) had no significant impact on weaving capacity [Ref. 24]. In addition, capacity decreases when the weaving flow increases. Lastly, higher percentages of truck in weaving section also decrease the capacity of weaving areas.

Stander and Tichauer [Ref. 25] examined a 220-meter (720-foot) Type A weave in South Africa, which consisted of two through lanes and an auxiliary lane. This weave was divided into three longitudinal segments of 73 meters (240 feet) each. Under the slow and moderate traffic conditions, most of the lane changes took place in the first segment. However, when the traffic increased, more drivers waited until the third segment to make lane changes; as a result, roughly the same number of lane changes occurred at the first and the third segment.

Denney and Williams [Ref. 26] proposed new methods to estimate the capacity of Type A weaves in research funded by the National Cooperative Highway Research

Program. In this research, three new techniques were introduced: (1) Individual Weaving Region Model, (2) A Probabilistic Capacity Model, (3) The Merge-Point Capacity Model. The first model was developed based on the assumption that, "at capacity, weaving maneuvers are evenly distributed along the length of the weaving zone." As a result, the weaving zone can be divided into individual weaving regions, each roughly the length of a single lane change maneuver. The second model was proposed based on the assumption that the inner freeway lanes are not impacted significantly by turbulence in the weaving zone; consequently, their individual capacity is closed to that of a comparable basic freeway segment. However, the data analyze does not support either model.

To provide the basis for evaluating and refining the best capacity models, a pilot study was conducted on southbound I-610 in Houston, Texas. The results showed that, the third model, Merge Point Model, provided the best description of the field data in the pilot study. The Merge Point Model was formulated based on the concept that "the traffic volumes affecting the weave are the cars that must change lanes in the weaving lanes and the cars entering the weaving lanes." The capacity of a single lane is then factored up by the inverse of the ratio of cars that must change lanes to the cars in the weaving zone. The model for ramp weaves is as follows:

For total weaving volumes, V_w, less than 700 veh/hour:

$$C_w = 2300 + V_w$$

For $V_w \geq 700$ veh/hour and total entering volume on the weaving lanes, $V_t \leq 3000$ veh/hour:

$$C_w = 3000 \tag{2.9}$$

For $V_t \geq 3000$ veh/hour:

$$C_w = (2 - VR_w)\left(1 + \frac{V_w - 2142}{2370}\right)(2300)$$

where

VR_w= the ratio of weaving volume over all cars in the weaving lanes

C_w= the capacity in the weaving lanes (not the whole freeway section)

V_w= the total weaving volume

V_t= the total entering volume on the weaving lane

A similar model was developed for Type A major weaves in this research.

Even though this model was designed for Type A weaves, by defining VR_w as the ratio of the car that must change lanes to total cars entering the weaving lanes, this model can also be applied for Type B weaves. For Type B weaves, one weaving movement does not have to change lane; therefore, one of the weaving flows is zero.

Roess and Ulerio [Ref. 27] wanted to remove the issue of configuration from the weaving analysis process. The new model was proposed in this paper, as a part of NCHRP Project 3-75, in order to calculate capacity of weaving area based on total lane-changing activities and speeds within the weaving section. It was modified from the equation used for estimating capacity of basic freeway segment by an additional factor, f_{wv}, which reflects the impact of weaving vehicles.

Continuing this work, Vu, et al. [Ref. 28], estimated the capacity of weaving section using VISSIM simulation package. A Type B weave with 6 lanes on I-80 EB, in Emeryville, California, was used as a test bed with data provided by the Next Generation Simulation (NGSIM). As in the previous study, the VISSIM model was calibrated based on the average running speed and total lane changing activities within the weaving section. After speed and density were extracted from the simulation runs, flow was simply found by multiplying speed by density. Afterward the capacity was found from the peak (or maximum value) of flow rate on a speed-flow curve.

In conclusion, speed-based methods have been a framework for weaving analysis. The HCM 2000 weaving analysis is also based on speed estimation. However, previous studies from California [Ref. 30] concluded that any models based on speed estimations cannot be accurate. Denney and Williams [Ref. 26] developed a non-speed based model in the NCHRP 3-55 project; however, this model can only applied to Type A and possibility Type B weaves. Next, Chapter 3 discusses the collection of the field data.

CHAPTER 3

DATA COLLECTION

This chapter describes the data collection process in San Antonio, Texas using the video equipment and voice recorder. The data obtained was used to calibrate and validate the simulation model.

3.1 Introduction

The site selected for this project was southbound I-35/410 between the Rittiman Road entrance on the right and the southbound I-410 exit on the left. This weaving area, whose length is 0.52 mile (2746 feet), is located in San Antonio, Texas, and illustrated in Figure 3.1.

Data was collected at this location on two days: June 29, 2005, and June 30, 2005. Data was collected from 4:00 p.m. to 6:00 p.m. on June 29, 2005, and from 7:30 a.m. to 9:30 a.m. on June 30, 2005. Data needed for this project were the volumes of the mainlanes, exit ramp, entrance ramp, and entrance ramp to the exit ramp. One O-D count is needed and R-R volume is the easiest to count. Due to recent construction, the loop detectors at this location were not operational. However, TxDOT has cameras at this location; therefore, volumes of the first three streams were obtained by counting directly from videotapes, which were recorded at TransGuide, the Traffic Management Center (TMC) for the San Antonio District of TxDOT. The traffic volume from the entrance ramp to the exit ramp was obtained using the license plate matching technique for 5-minute intervals.

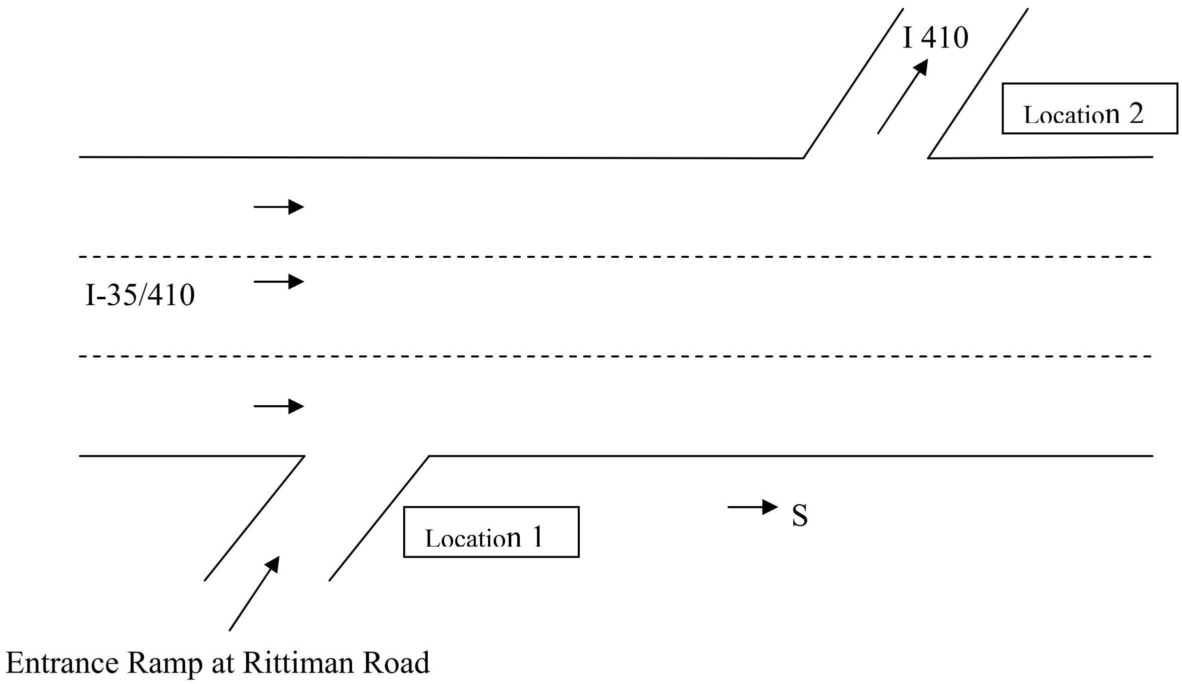

Figure 3.1 I-35/410 Southbound, Two-Sided Weave

3.2 Site Visit and Equipment Check

Before doing data collection in the afternoon, the data collection team went to the site in the morning and selected the most comfortable locations with the best view for license-plate readers. Moreover, this visit helped the license-plate readers became familiar with the site and the traffic behavior at the site.

The data collection team also visited the traffic control room at TransGuide. The purpose of this visit was to set the view angle of the cameras to make sure that the videotapes recorded at TransGuide had the best view for traffic volume counts in each freeway lane, and the entrance ramp and exit ramp.

Afterward, the data collection team checked equipment and reviewed safety issues related to data collection. At this stage, watches and clocks were synchronized and each observer verified that he had backup batteries and audiotapes. Audiotapes were also labeled with the time, date, and specific location of data collection. Finally, water bottles were purchased for the whole team before the scheduled data collection time.

3.3 Begin Data Collection at Scheduled Times

Two students spoke clearly the last three digits of each vehicle's license plate number into voice recorders at the entrance ramp (location 1) and the exit ramp (location 2) when the vehicle passed the checkpoint. At the same time, one student, who sat at TransGuide, also started to record videotapes for two locations. This task will be described in detail in section 3.5. While the license plate was read, the time from the watch was also recorded. These time checks can be used as an additional time check in transcribing the tape at a later date. The plate number was collected while the vehicle was approaching because this technique allowed the observers to record the plate number at the instant the vehicle crosses the designated checkpoints. When changing audiotapes, observers checked to make sure that they picked the tape with correct label. There was an incident on southbound I-35 at 9:15 a.m. on June 30, 2005; thus, the data collected in the last 15 minutes on this day was not used.

3.4 Transcribing the License Plates into a Spreadsheet

Transcription of the license plates from the audiotapes into a spreadsheet was performed in the office. License plate transcription from audiotapes took approximately two to three hours per hour of tape. Then, these license plate numbers were put in Excel spreadsheets, and license plate were matched. The resulting volumes for each of these forty-four 5-minute-intervals are shown in Table 3.1.

Table 3.1 Volume from the entrance ramp to exit ramp (veh/5min)

Day	Time	Volume from the entrance ramp to exit ramp (veh/5min)
June 29, 2005	4:00 p.m. – 4:05 p.m.	16
	4:05 p.m. – 4:10 p.m.	18
	4:10 p.m. – 4:15 p.m.	20
	4:15 p.m. – 4:20 p.m.	20
	4:20 p.m. – 4:25 p.m.	13
	4:25 p.m. – 4:30 p.m.	13
	4:30 p.m. – 4:35 p.m.	14
	4:35 p.m. – 4:40 p.m.	18
	4:40 p.m. – 4:45 p.m.	15
	4:45 p.m. – 4:50 p.m.	11
	4:50 p.m. – 4:55 p.m.	15
	4:55 p.m. – 5:00 p.m.	13
	5:00 p.m. – 5:05 p.m.	16
	5:05 p.m. – 5:10 p.m.	8
	5:10 p.m. – 5:15 p.m.	25
	5:15 p.m. – 5:20 p.m.	12
	5:20 p.m. – 5:25 p.m.	14
	5:25 p.m. – 5:30 p.m.	13
	5:30 p.m. – 5:35 p.m.	19

Table 3.1 –continued

Day	Time	Volume from the entrance ramp to exit ramp (veh/5min)
	5:35 p.m. – 5:40 p.m.	24
	5:40 p.m. – 5:45 p.m.	13
	5:45 p.m. – 5:50 p.m.	11
	5:50 p.m. – 5:55 p.m.	18
	5:55 p.m. – 6:00 p.m.	14
June 30, 2005	7:35 a.m. – 7:40 a.m.	23
	7:40 a.m. – 7:45 a.m.	12
	7:45 a.m. – 7:50 a.m.	8
	7:50 a.m. – 7:55 a.m.	16
	7:55 a.m. – 8:00 a.m.	9
	8:00 a.m. – 8:05 a.m.	16
	8:05 a.m. – 8:10 a.m.	10
	8:10 a.m. – 8:15 a.m.	10
	8:15 a.m. – 8:20 a.m.	10

3.5 Collecting Mainlane Volume, Exit Ramp Volume, Entrance Ramp Volume

TransGuide has two cameras at this weaving location: the first one is at the entrance ramp at Rittiman Road and the second is at the I-410 exit ramp. The first camera was set to record the traffic on the mainlanes and the entrance ramp, while the second camera focuses on the exit ramp. Transcription of traffic volumes from videotapes to computer is also performed in the office with a mechanical counting board. Tables 3.2 and 3.3 are the traffic volumes for the two-day counts. In these tables, traffic volumes on lane 1, lane 2, and lane 3 of the freeway mainlanes are counted at the merge-gore of entrance ramp, where lane 1 is the right lane and lane 3 is the left lane.

Table 3.2 Traffic Volume on June 29th, 2005

Time	Exit Ramp Volume	Entrance Ramp Volume	Lane 1 Volume	Lane 2 Volume	Lane 3 Volume
4:00 p.m. – 4:05 p.m.	168	65	84	99	103
4:05 p.m. – 4:10 p.m.	175	60	105	105	105
4:10 p.m. – 4:15 p.m.	154	73	83	117	95
4:15 p.m. – 4:20 p.m.	157	64	79	109	96
4:20 p.m. – 4:25 p.m.	148	74	87	111	104
4:25 p.m. – 4:30 p.m.	171	75	82	108	110
4:30 p.m. – 4:35 p.m.	151	59	85	106	105
4:35 p.m. – 4:40 p.m.	173	80	92	110	107
4:40 p.m. – 4:45 p.m.	185	56	82	102	94
4:45 p.m. – 4:50 p.m.	165	58	85	98	104
4:50 p.m. – 4:55 p.m.	178	72	91	99	104
4:55 p.m. – 5:00 p.m.	155	59	97	90	102
5:00 p.m. – 5:05 p.m.	179	86	80	92	92
5:05 p.m. – 5:10 p.m.	174	67	92	83	96
5:10 p.m. – 5:15 p.m.	180	91	87	110	106
5:15 p.m. – 5:20 p.m.	188	69	101	121	104
5:20 p.m. – 5:25 p.m.	165	78	101	115	115
5:25 p.m. – 5:30 p.m.	171	54	105	98	103
5:30 p.m. – 5:35 p.m.	170	89	100	110	99
5:35 p.m. – 5:40 p.m.	171	66	82	99	107
5:40 p.m. – 5:45 p.m.	154	56	96	106	102
5:45 p.m. – 5:50 p.m.	165	50	102	116	125
5:50 p.m. – 5:55 p.m.	157	60	98	101	99
5:55 p.m. – 6:00 p.m.	149	55	101	119	128

Table 3.3 Traffic Volumes on June 30th, 2005

Time	Exit Ramp Volume	Entrance Ramp Volume	Lane 1 Volume	Lane 2 Volume	Lane 3 Volume
7:30 a.m. – 7:35 a.m.	133	69	113	156	186
7:35 a.m. – 7:40 a.m.	124	80	114	152	162
7:40 a.m. – 7:45 a.m.	135	68	111	145	169
7:45 a.m. – 7:50 a.m.	126	63	117	140	141
7:50 a.m. – 7:55 a.m.	126	77	104	153	143
7:55 a.m. – 8:00 a.m.	138	60	104	127	146
8:00 a.m. – 8:05 a.m.	143	67	94	130	138
8:05 a.m. – 8:10 a.m.	131	60	97	133	145
8:10 a.m. – 8:15 a.m.	122	61	119	146	148
8:15 a.m. – 8:20 a.m.	128	46	120	132	157
8:20 a.m. – 8:25 a.m.	130	51	109	128	145
8:25 a.m. – 8:30 a.m.	109	64	87	114	108
8:30 a.m. – 8:35 a.m.	118	50	76	110	121
8:35 a.m. – 8:40 a.m.	117	62	81	124	120
8:40 a.m. – 8:45 a.m.	108	52	65	83	107
8:45 a.m. – 8:50 a.m.	122	74	65	113	113
8:50 a.m. – 8:55 a.m.	116	48	75	108	111
8:55 a.m. – 9:00 a.m.	103	63	76	94	91
9:00 a.m. – 9:05 a.m.	76	50	55	72	69
9:05 a.m. – 9:10 a.m.	93	60	54	70	80
9:10 a.m. – 9:15 a.m.	145	78	96	107	133

3.6 Travel Time Samples Collection

The team also collected travel times through this weaving section, which are shown in Tables 3.4 and 3.5.

Table 3.4 Travel time on June 29, 2005

Wednesday June 29, 2005. Time: 4:00 p.m. – 6:00 p.m.

Time		Travel time	Lane	Approximated Speed (mph)
Begin	End			
1:06:55	1:08:15	80s	Left	23
1:30:15	1:31:50	95s	Right	20, queue at ramp:3-4 veh
1:50:03	1:50:58	55s	Middle	34
2:09:15	2:10:05	50s	Right	37
2:29:08	2:30:55	107s	Left	18
2:51:09	2:52:15	66s	Middle	28

Table 3.5 Travel Time on June 30, 2005

Thursday June 30, 2005. Time: 7:30 a.m. - 9:30 a.m.

Time		Travel Time	Lane	Approximated Speed (mph)
Begin	End			
52:07	52:49	42s	Left	44
1:10:37	1:11:12	35s	Middle	53
1:22:06	1:22:58	52s	Right	36
1:33:37	1:34:08	31s	Left	60
1:51:29	1:52:06	37s	Middle	50
2:14:42	2:15:20	38s	Right	49

Volume and speed data from 7:35 a.m. to 8:35 a.m. on Thursday, June 30, 2005, were used to calibrate the simulation model. Volume and speed data from 4:00 p.m. to 6:00 p.m. on Wednesday, June 29, 2005, were used to validate the model. Next, chapter 4 will discuss the simulation of the weaving area using VISSIM.

CHAPTER 4

SIMULATION OF WEAVING AREAS

Simulation is a key element of this research and VISSIM was selected because of its capabilities. The primary objective of this chapter is to introduce the VISSIM parameters that affect the capacity estimation of weaving areas. This chapter consists of three sections. The first section presents an overview of the VISSIM software. VISSIM inputs are discussed in the second section and the third section presents the simulation outputs.

4.1 VISSIM Overview

VISSIM [Ref. 29] is a microscopic, time step and behavior based simulation model developed to model urban traffic. In contrast to less complex simulation software using constant speeds and deterministic car following logic, VISSIM uses a psycho-physical driver behavior model. In addition, VISSIM simulates traffic flow by moving "driver-vehicle-units" through a network. Every driver with his/her specific behavior characteristic is assigned to a specific vehicle. As a consequence, the driver behavior corresponds to the technical capabilities of his/her vehicle. In addition, VISSIM can model public transportation operations such as buses, light rail, heavy rail, pedestrian, and bicycles. The model was developed at the University of Karlsruhe, Germany, during the early 1970s and first commercially distributed in 1992 by PTV

Transworld AG, who continues to distribute and maintain VISSIM today [Ref. 31]. VISSIM version 4.10 was used in this research.

The simulation package VISSIM includes two main components: (1) traffic simulator and (2) signal state generator (SSG). The traffic simulator is a microscopic traffic flow simulation model, which includes car following and lane changing logic. Even though links are used in the simulator, VISSIM does not have a traditional node structure. The lack of nodes provides the user with the flexibility to control traffic operations (e.g., yield conditions) and vehicle paths within an intersection or interchange [Ref. 31].

Four driving modes are also defined in VISSIM: free driving, approaching, following, and braking. In each mode, the acceleration is described as a result of speed, speed difference, distance and the individual characteristics of driver and vehicle. The driver switches from one mode to another as soon as he reaches a certain threshold that can be expressed as a combination of speed difference and distance.

On the other hand, the SSG is a signal control routine which polls detector information from the traffic simulator on a discrete time step basis (as small as one tenth of a second). Then, it determines the signal status for the following second and returns this information to the traffic simulator [Ref. 29]. The signal state generator allows users to model different types of signalized intersections including fixed time, actuated, transit signal priority, and ramp metering. One important feature of the SSG is that it is programmable. With a descriptive language called VAP (Vehicle Actuated

Phasing), the user can access loop-detector measurements, and use them to generate commands for traffic signals.

4.2 VISSIM Inputs

4.2.1 Network Geometry Coding

VISSIM networks are based on links and connectors. Links are used to define the width and number of lanes for a given roadway segment. There are five different link types and each link type is represented by its driving behavior model. Connectors are used to connect the links at intersections and implicitly have the same link types as the link from which they originate. Since this research focuses on freeway weaving, the link type for this model is freeway (type 3). Links and connectors of the weaving area in this project are built on an aerial photograph downloaded from Google Earth.

4.2.2 Vehicle Types and Traffic Compositions

A group of vehicles with similar technical characteristics and physical driving behavior is defined as a vehicle type. Typically, the following vehicles types are available: car, HGV (truck), bus, tram, pedestrian, and bike. Each vehicle type is characterized by minimum and maximum acceleration, minimum and maximum deceleration, weight, power, and length. Traffic mix is the proportion of each vehicle type present in the source flows. In this research, the traffic mix is: (1) 98% car, (2) 2% HGV (truck). Both vehicle types are set to enter the network with speed distribution number 80, which has a minimum speed of 46.6 mph and maximum speed of 68.4 mph.

4.2.3 Vehicle Inputs and Routing

Vehicle inputs are defined for a specific link and time period in vehicles per hour even if the time period is different from one hour. In this research, five different time periods were established for each run; one five-minute time period followed by 4 fifteen-minute time periods. The first period is the warm up time and the next four 15 minute-intervals are the real running time where the simulation outputs are collected. The 5-minute warm up time is included because the network starts empty. Routing is defined based on the origin-destination matrix and affects only vehicles of a class that is included in the routing decision.

4.2.4 Lane Change Model

The lane change logic is one of the most important elements in a weaving simulation because lane-changing activities are the major characteristics of weaving segments. There are basically two kinds of lane changes in VISSIM: necessary and free lane changes. A necessary lane change occurs when a vehicle must make a lane change in order to reach the next connector of a route while a free lane change occurs when drivers have more room or higher speed in the adjacent lane.

The driving behavior parameter for necessary lane changes contains the maximum acceptable deceleration for the vehicle and the trailing vehicle on the next lane, which depends on the distance to the emergency stop position of the next connector of the route. Other parameters included in the necessary lane change model include look-back distance, emergency stop distance, waiting time before diffusion, and minimum headway.

For the case of a free lane change, VISSIM checks for the desired safety distance of the trailing vehicles on the new lane, which depends on its speed and the speed of the vehicle that wants to change to that lane. Even though the user cannot change the "aggressiveness" for these lane changes, changing the parameters for the desired safety distance will affect the free lane changes as well. Those parameters that are important for the weaving simulation are described below:

The look-back distance defines the distance where a vehicle will begin to maneuver towards the desired lane; the default value is 656.2 ft. The look-back distance works only if the direction decisions value is different than "All." Direction decisions are the direction, which the drivers have to follow in order to stay on their route. For example, if drivers who want to exit a freeway start to maneuver to the right lane 1000 ft before the exit ramp, a look-back distance value of 1000 ft and Direction Decisions "Right" have to be set in the Connector Lane Change.

The emergency stopping distance is the distance where a vehicle will stop to wait for an opportunity to change lanes in order to stay on its route if the vehicles cannot change lanes because of heavy traffic; default value is 16.4 ft.

The waiting time before diffusion is the maximum amount of time a vehicle can wait at the emergency stop position for a gap to change lanes in order to stay on its route. When the time is reached, the vehicle is taken out of the network and a message will be written to the error file; default value is 60 seconds.

Minimum headway (front/rear) defines the minimum distance to the vehicle in from that must be available for lane change in standstill condition, default value 1.64 ft.

4.2.5 Car-Following Model

Car-following models provide quantitative values of the acceleration/deceleration for one vehicle following another when the leading vehicle changes its speed over time. VISSIM includes two versions of the Wiedemann model: Wiedemann 74 for arterial streets and Wiedemann 99 for freeway. Since this project is about freeway weaves, only the Wiedemann 99 model is used. The Wiedemann 99 model consists of ten tunable parameters: CC0 to CC9. Those CC-parameters that are modified from the default values were described below:

- CCO (standstill distance) is the desired distance between stopped cars. The default value of CCO is 1.5m.
- CC1 (headway time) is the time (in s) that a driver wants to keep. The higher the value, the more cautious the driver is. The default value of CC1 is 0.9s. CCO and CC1 are used to calculate the safety distance defined as a minimum distance a driver will keep while following another car: dx_safe=CCO+v*CC1, where v (m/s) is the speed of the trailing vehicle.
- CC4 and CC5: These control the speed differences during the "car following" state. Smaller values result in a more sensitive reaction of drivers to accelerations or decelerations of the preceding car. It is recommended in [Ref. 29] that these parameters have opposite signs and equal absolute values. The default values are CC4=-0.35 and CC5=0.35.

4.3 VISSIM Outputs

Outputs from VISSIM included in the calibration process are link evaluation and data collection because the results can be used for simulation calibration and a capacity estimation model for the weaving area. The link evaluation feature in VISSIM allows the user to gather simulation results (link number, lane number, speed, density, volume, etc.) based on an active link in a user-defined time interval. All the simulation results are written in a link evaluation file (*.STR). The location of each link in the network is shown in Figure 4.1. Entrance ramp at Rittiman Road is link 6 and exit ramp to the southbound I-410 is link 9.

On the other hand, the data collection feature allows the user to collect simulation results (minimum speed, mean speed, maximum speed, number of vehicle, acceleration, etc.) on single cross sections or at a point. All the simulation results extracted from the data collection feature are written to a text file (*.MES). The location of each data collection point in the network is shown in Figure 4.2. The number 1-7 and 14-17 refer to specific points where the data was collected. Output files from VISSIM are formatted as a text file, which can be read into Excel. The user-defined time interval in this model is from 300 seconds to 3900 seconds (1 hour) with an interval of 900 seconds (15 minutes). This means that VISSIM will collect information for a one-hour run and reports it into four 15-minutes data sets. Table 4.1 and Table 4.2 are examples of link evaluation outputs and data collection output for the Simulation Model with a look back distance of 2500 ft and a160 ft acceleration lane for the morning peak from 7:35 a.m. to 8:35 a.m.

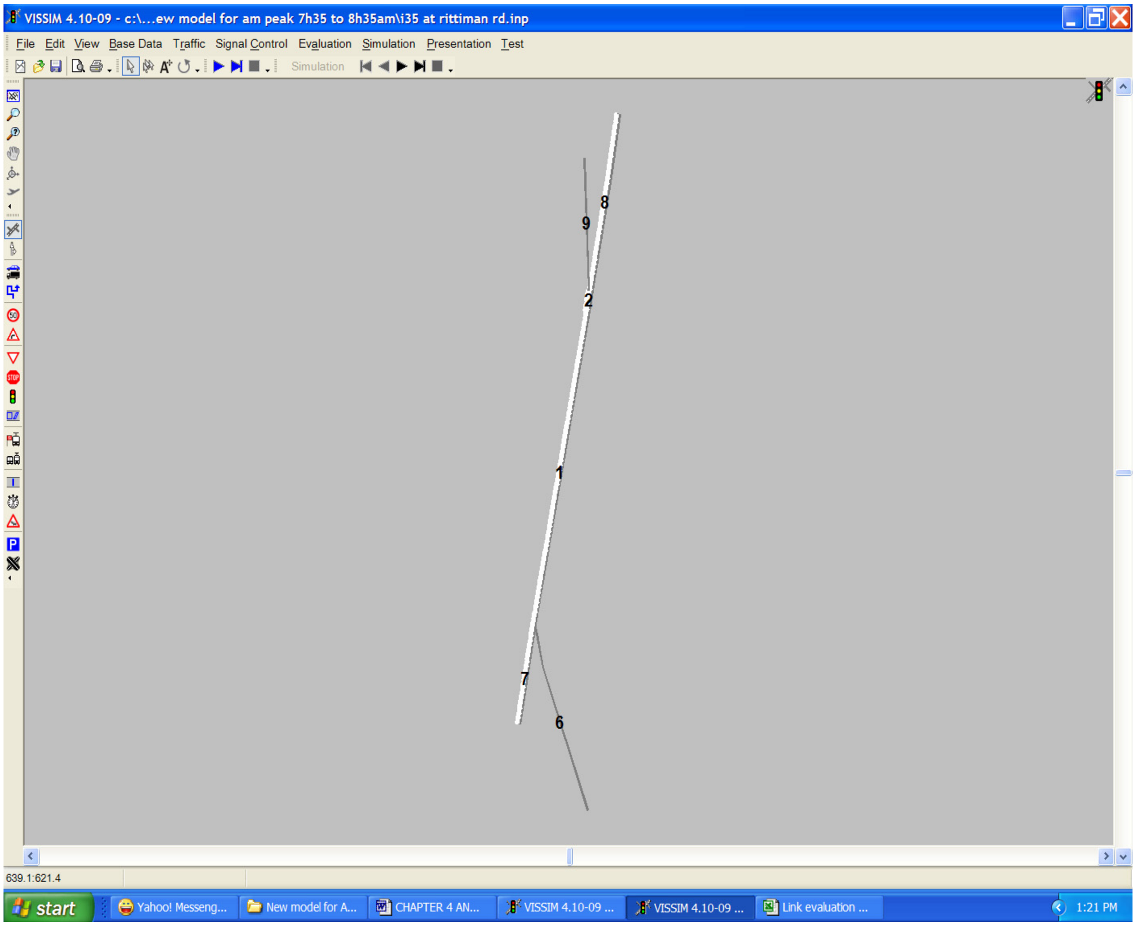

Figure 4.1 Location of Each Link in The Network (North is up).

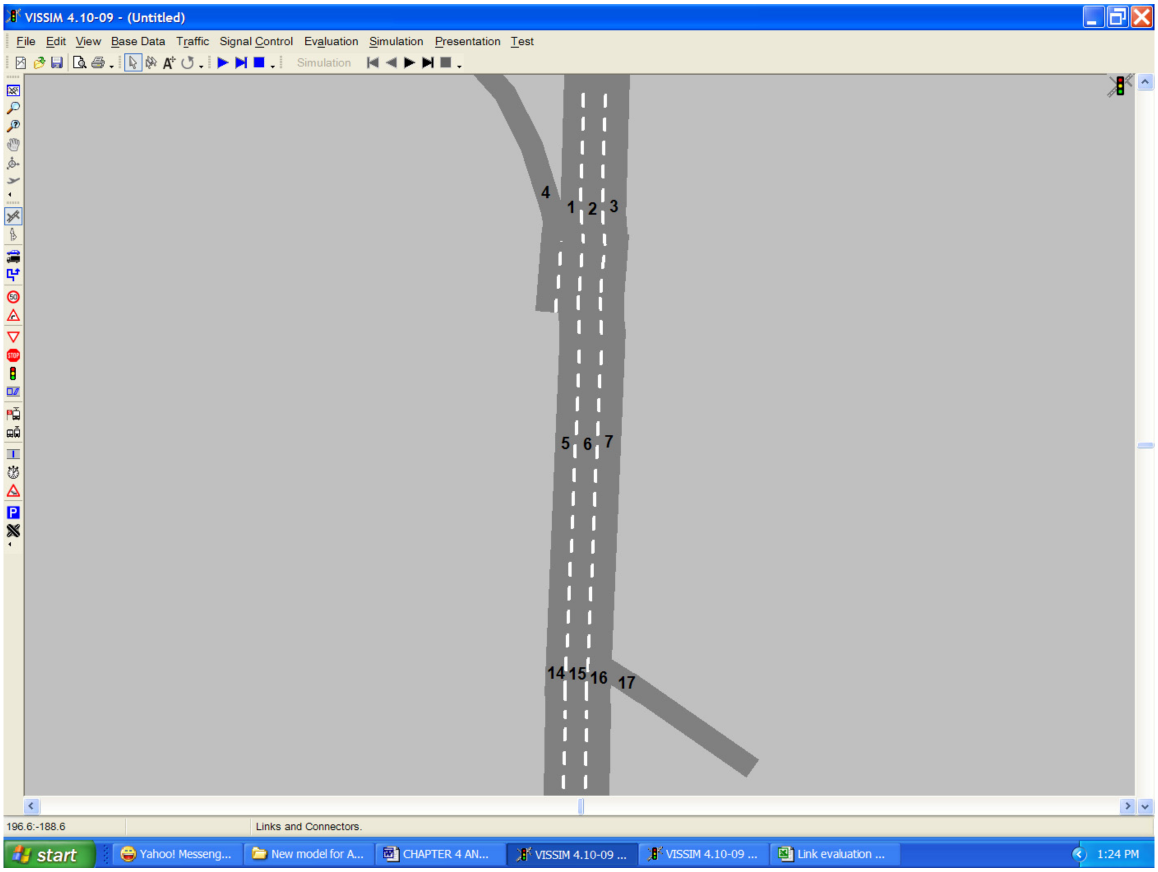

Figure 4.2 Location of Each Data Collection Point (North is up)

.

Table 4.1 An Example of Link Evaluation File

Link	Lane	Density (veh/mi)	Speed (mph)	Volume (vph)
1	1	33.25	48.72	1620.16
1	2	36.42	48.4	1762.62
1	3	49.76	46.02	2289.64
2	1	42.42	13.08	554.79
2	2	74.15	17.38	1288.52
2	3	47.71	37.4	1784.28
2	4	41.32	41.45	1712.7
6	1	31.9	51.25	1635.06
7	1	32	51.83	1658.64
7	2	29.69	52.17	1548.89
7	3	16.28	53.66	873.4
8	1	34.95	46.61	1629.41
8	2	33.79	49.8	1682.78
8	3	32.89	50.92	1674.44
1	1	31.59	48.54	1533.55
1	2	35.06	48.5	1700.39
1	3	48.1	46.03	2213.63
2	1	37.08	13.79	511.31
2	2	62.11	21.44	1331.97
2	3	40.15	40.53	1627.39
2	4	34.75	43.62	1515.8
6	1	31.24	51.52	1609.27
7	1	29.01	52	1508.29
7	2	29.26	52.16	1526.25
7	3	15.79	53.77	849
8	1	29.41	49.66	1460.6
8	2	30.45	51.58	1570.82
8	3	28.91	52.05	1504.72
1	1	31.15	48.04	1496.37

Table 4.1-continued

	Lane	Link	v(speed) (mph)	Volume (vph)
1	2	34.7	48.46	1681.24
1	3	48	45.83	2199.66
2	1	21.15	18.6	393.44
2	2	50.01	29.52	1476.25
2	3	35.82	46.37	1661.07
2	4	30.48	49.66	1513.84
6	1	29.76	51.56	1534.3
7	1	28.33	51.62	1462.36
7	2	28.79	52.07	1498.91
7	3	17	53.24	905.32
8	1	30.51	51.01	1556.49
8	2	31.7	51.88	1644.57
8	3	30.28	52.42	1587.56
1	1	24.68	50.74	1252.29
1	2	28.01	50.92	1426.13
1	3	40.1	49.98	2004.1
2	1	20.11	18.98	381.51
2	2	36.66	35.94	1317.38
2	3	27.48	50.04	1375.02
2	4	24.32	51.92	1262.88
6	1	25.33	52.46	1328.74
7	1	24.1	52.06	1254.58
7	2	24.98	52.06	1300.72
7	3	16.38	53.23	871.99
8	1	25.08	52.49	1316.51
8	2	25.37	52.54	1332.89
8	3	25.25	52.86	1334.62

Table 4.2 An Example of Data Collection File

Data Collection Point	From	To	Speed Mean (mph)	Speed Minimum (mph)	Speed Maximum (mph)	Number of Vehicles	Accel. Maximum (ft/s^2)	Accel. Minimum (ft/s^2)
1	300	1200	36	1.1	59.1	345	10.5	-24.8
2	300	1200	44.4	8.1	60.2	454	9.3	-23.9
3	300	1200	47	13.1	59.6	450	7.6	-21.9
4	300	1200	32	2.4	40.3	211	-1.5	-17.3
5	300	1200	46.4	22.7	63.8	349	7.2	-22.5
6	300	1200	43.8	22.5	59.2	500	8.2	-17.8
7	300	1200	41.5	14.9	58.4	584	8.3	-23.5
14	300	1200	52	44.9	63.6	420	3.8	-1.5
15	300	1200	52.2	46.4	66.5	432	5.3	-1.6
16	300	1200	52.3	47.3	67.3	171	5.2	-1.4
17	300	1200	51.1	42.7	57	411	5.9	-1.3
1	1200	2100	41.7	5.7	62.1	332	8.9	-22.4
2	1200	2100	47.8	10.2	60.7	414	8.4	-23.6
3	1200	2100	49.3	14.2	60.9	391	4.9	-22.8
4	1200	2100	32.6	2.3	40.6	205	8.8	-16.4
5	1200	2100	46.4	23.7	60.5	350	6.9	-4.4
6	1200	2100	43.9	19.9	66.2	463	7.8	-20.4
7	1200	2100	42.1	13.3	58.5	556	8.1	-22
14	1200	2100	51.9	44.9	61.6	387	5.5	-1.9
15	1200	2100	52.1	46.6	66.7	420	5.8	-1.8
16	1200	2100	52.2	42.4	60.7	169	5.2	-0.8
17	1200	2100	51.1	42.4	59.7	403	5.8	-1.4
1	2100	3000	46.9	7.1	65.1	360	5.6	-21
2	2100	3000	50	13.6	64.3	441	7	-23.1
3	2100	3000	51.6	22.1	60.9	400	5.6	-22.4
4	2100	3000	35.5	22.9	40.2	164	-1.3	-11.7
5	2100	3000	45.7	20	64.8	353	6.7	-9.9

Table 4.2- continued

Data Collection Point	From	To	Speed Mean (mph)	Speed Minimum (mph)	Speed Maximum (mph)	Number of Veh	Accel. Maximum (ft/s^2)	Accel. Minimum (ft/s^2)
7	3000	3900	49.3	28.3	62.7	480	7.2	-11.7
14	3000	3900	52.3	44.1	61.9	319	4.1	-2.7
15	3000	3900	52.2	45.7	61.4	350	5.6	-2.2
16	3000	3900	52.5	45.3	65	188	5.3	-1
17	3000	3900	52	44.1	58.5	335	5.2	-1.2

This chapter introduced VISSIM and presented the simulation inputs and outputs needed for analyzing a weaving section. The next chapter will discuss in detail the selected inputs for calibrating the simulation. In addition, chapter 5 includes the validation process for simulation model in order to receive the best-calibrated simulation model.

CHAPTER 5

MODEL CALIBRATION AND VALIDATION

This chapter discusses the calibration and validation processes. In this chapter, lanes 1, 2, and 3 stand for right, middle, and left lane respectively for a three-lane weaving section. This chapter includes seven sections: (1) Calibration Process Overview, (2) Selection of Calibration Parameters, (3) Simulation Running Time and Simulation Resolution, (4) Tolerance and Number of Runs, (5) Calibration Process, (6) Model Selection, and (7) Validation.

5.1 Calibration Process Overview

Model calibration is the procedure where model parameters are adjusted so that the model represents the local driver behavior and traffic performance characteristics. In other words, model calibration is the process to make sure that the model behaves the same as the observed traffic in the field. This task is performed after all input data and model coding have been thoroughly checked.

Calibration is important because no single model is expected to have the ability to equally represent all possible traffic conditions. Even the most detailed microscopic simulation model still has a portion of variables determined by real-world traffic conditions [Ref. 32]. Moreover, every microscopic simulation software package includes a set of user-defined parameters for the purpose of calibrating the model to local conditions. Even though the software developers suggest default values for these user-defined parameters, models that use these default values can rarely produce accurate results. The objective of calibration is to find the set of parameter values for the model that best duplicates local traffic conditions and behavior.

5.2 Selection of Calibration Parameters

Only parameters that affect the performance of traffic in a freeway weaving section are selected at this stage. These parameters are:

- Length of acceleration lane: Even though an acceleration lane does not exist at this site, it is still included in the simulation in order to obtain similar traffic behavior with field conditions. Without an acceleration lane very large gaps are required in the freeway lanes to allow the entry vehicle to enter and accelerate. If the length of an acceleration lane is too short, traffic will queue on the entrance ramp and spill back on the frontage road. If the length of an acceleration lane is too long, the travel speed is too high when compared to travel speed in the field. The range of acceleration lane length was from 100 ft to 250 ft.
- The look-back distance, as mentioned in chapter 4, defines the distance at which vehicles will begin to maneuver towards the desired lane. The default value is 656.2 ft. The range of look back distance used for calibration was from 2000 ft to 3000 ft.
- Emergency stopping distance: the default value of 16.4 ft was used.

- Waiting time before diffusion: the default value of 60 seconds was used. If the elimination on of ramp blockages is desired, the value of this parameter has to be decreased.
- Minimum headway: was 3ft (default values 1.6ft)
- Car following model parameters: The ranges of the car following model parameters in this work are shown in table 5.1.

Table 5.1 Calibration Car Following Model Parameters

Parameters	Value
CC0	1.5 to 1.7
CC1	1.0 to 1.3
CC4	-0.35 to –2
CC5	0.35 to 2

5.3 Simulation Running Time and Simulation Resolution

The simulation running time is 3900 seconds, which includes 300 seconds for warm-up. The simulation resolution, which is the number of times each vehicle's position is calculated within one simulation second, is set to 5 (range 1 to 10).

5.4 Tolerance and Number of Runs

The number of simulation runs required for each observation point can be computed using the following equation:

$$n = \left(\frac{Z \times \sigma}{e} \right)^2 \tag{5.1}$$

where n = number of simulation runs required for each observation point

Z = normal score

σ = standard deviation

e = tolerance

The standard deviation is obtained from multiple runs. Normally, four runs are considered to be the minimum number of preceding repetitions required for calculating the sample standard deviation. However, this number must be larger than the minimum number of run required for simulation in equation 5.1 [Ref. 36]. For the purpose of calibration, seven runs of model five (which is described in detail in the next section) were conducted. Two major parameters, which are the volumes for each lane from 7:35 to 7:50 a.m. and speed, were selected to calculate the required number of runs. The number of required runs based on these two parameters is shown in Table 5.2 and Table 5.3

The tolerance for volumes and speed in tables 5.2 and 5.3 were 20 vph and 2 mph, respectively. The normal score for 95% confidence is 1.96. Applying these values into equation 5.1, the number of runs needed for calibration was found. The result showed that one simulation run is needed for calibration in most of the cases; however, three runs were conducted and the mean value of these three runs was used to compare among models. The number of runs for validation may be different from the number for calibration.

Table 5.2 Volumes on Each Lane for Different Random Seed for Model 7 from 7:35 to 7:50 a.m.

Random Seed	Exit Ramp Volume	Lane 1 Volume	Lane 2 Volume	Lane 3 Volume	Entrance Ramp Volume
RS 30	422	325	455	445	209
RS 20	426	330	470	438	209
RS 25	408	346	434	436	211
RS 35	401	340	454	446	205
RS 40	414	331	453	449	205
RS 45	428	365	452	425	206
RS 50	423	373	455	424	210
Standard Deviation	10.06	18.38	10.48	10.01	2.48
Tolerance e	20	20	20	20	20
95% Confidence	1.96	1.96	1.96	1.96	1.96
Number of runs	1	3	1	1	1

Table 5.3 Speeds for Different Random Seed for Model 7 from 7:35 to 8:35 a.m.

Random Seed	Lane 3	Lane 2	Lane 1
RS 20	43.79	49.77	51.07
RS 30	41.94	50.03	50.49
RS 25	42.55	49.73	49.19
RS 35	44.92	49.24	51.4
RS 40	43.85	49.99	51.41
RS 45	43.12	47.95	50.76
RS 50	45.61	47.94	50.63
Standard Deviation	1.29	0.92	0.76
Tolerance e	2	2	2
95% Confidence	1.96	1.96	1.96
Number of runs	1	1	1

5.5 Calibration Process

The model was calibrated using the data collected in the field from 7:35 a.m. to 8:35 a.m. At this stage, values of selected parameters were changed in different runs until the best-fit model was found. The parameter selection methodology consisted of iterated runs, visual evaluation, and volume and speed comparisons. Volumes of each freeway mainlane, entrance ramp, and exit ramp were extracted from the videotapes, and these values were compared with the output values of VISSIM model. The locations where volumes were collected and compared are indicated as location numbers 1, 2, 3, 4 and 17 in Figure 4.2. Similarly, travel speeds (space mean speed) from merge gore to diverge gore on each freeway lane are also compared with the field values. Since only six travel speed samples were collected using the probe-vehicle technique, the speed comparison is used to make sure that the travel speeds from the simulation output are reasonable when compared with the field values. The level of accuracy for the volume output is considered more important than speed.

5.5.1 Models with no changes in the Car Following Model

In these models, all parameters related to the Widemann 99 car following model are unchanged. The values of acceleration lane length and look back distance are adjusted while the default values are applied to emergency stopping distance and waiting time before diffusion.

5.5.1.1 Model with look back distance 3000 ft from exit ramp and 100 ft acceleration lane length

During the simulation, there were many of vehicles waiting on the entrance ramp due to the short length of the acceleration lane, which did not match the observed condition on the field. Moreover, the discharge rate at the entrance ramp is smaller than the input flow and 15 vehicles are removed from the network after waiting for a lane change for more than 60 second. The length of the acceleration lane is too short.

5.5.1.2 Model 1: Look back distance 3000 ft and 150 ft acceleration lane length

In order to avoid a long queue formed at the entrance ramp, the length of acceleration lane was increased from 100 ft to 150 ft in this model. The look back distance was unchanged, which allows seeing how the simulation behaves when the length of acceleration was changed. No error file was written after the simulation run because no car was removed from the network. The number of waiting vehicles at the entrance ramp also decreased. However, the queue is still longer than observed in the field. The volume and speed comparison between simulation output and field data for this model is presented in Figure 5.1 to Figure 5.5. Volumes collected from the simulation are very similar to the field data while the speed is quite different. In Figures 5.1 to 5.4, lane 1 volume is always smaller than the field value, but lane 2 and 3 volumes are larger than the field values. This means exiting vehicles maneuver to lane 3 earlier than vehicles in the field did. The look back distance was too long. In addition, it can be seen clearly that the speed differences on the left and middle lanes are less than five mph whereas the speed difference in the right lane is about 15 mph.

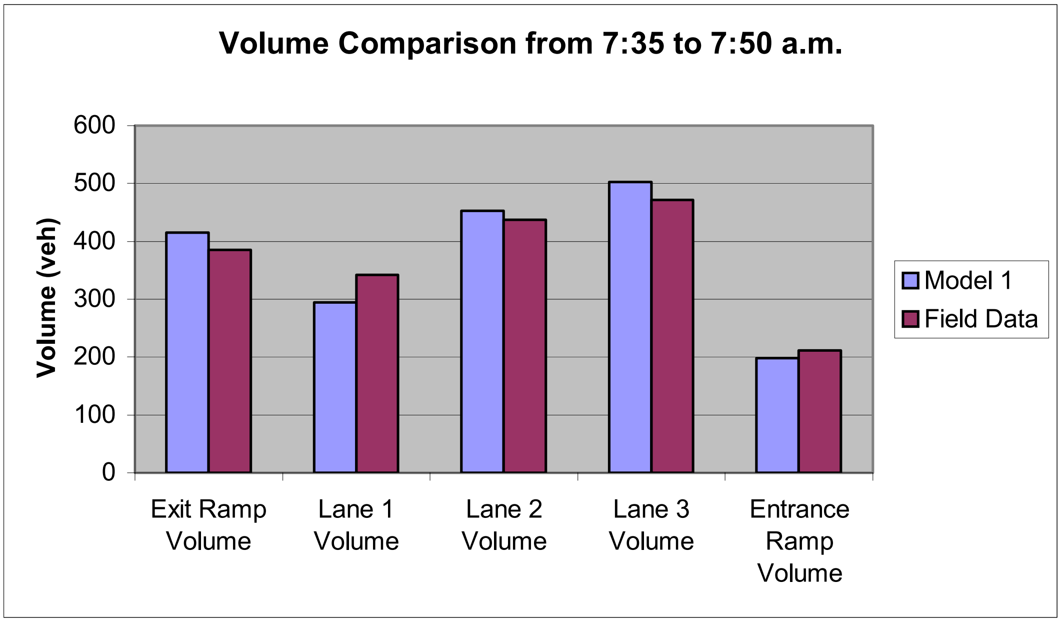

Figure 5.1 Volume Comparison from 7:35 to 7:50 a.m. for Model 1

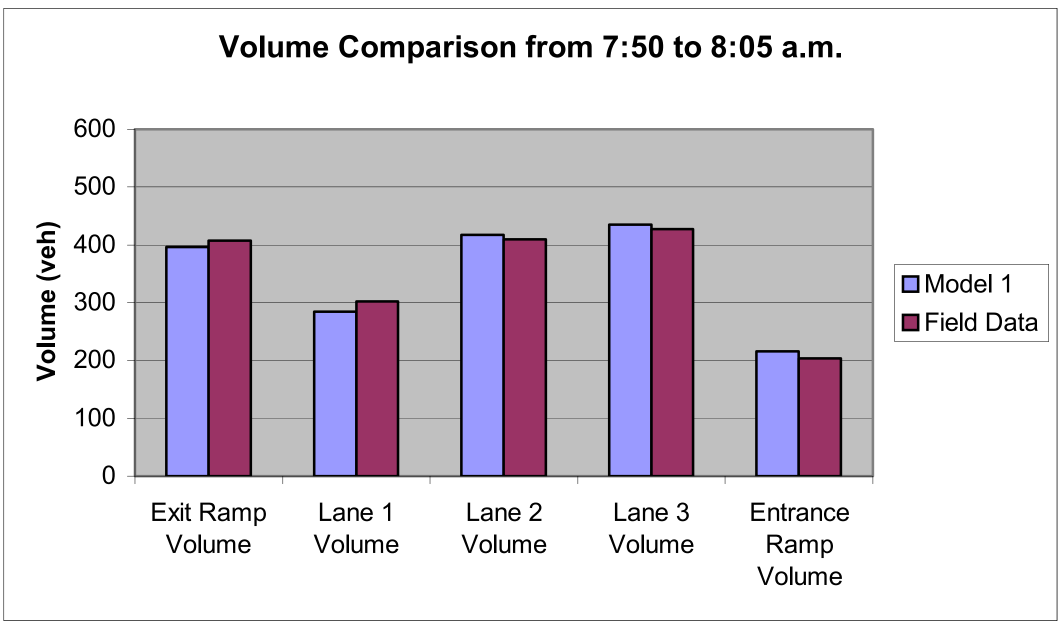

Figure 5.2 Volume Comparison from 7:50 to 8:05 a.m. for Model 1

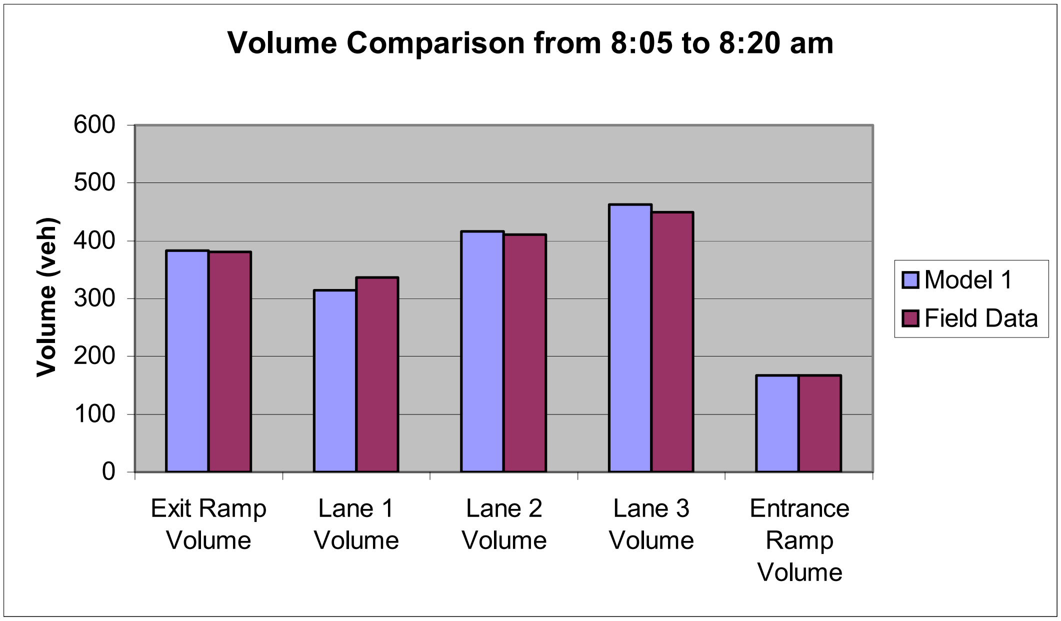

Figure 5.3 Volume Comparison from 8:05 to 8:20 a.m. for Model 1

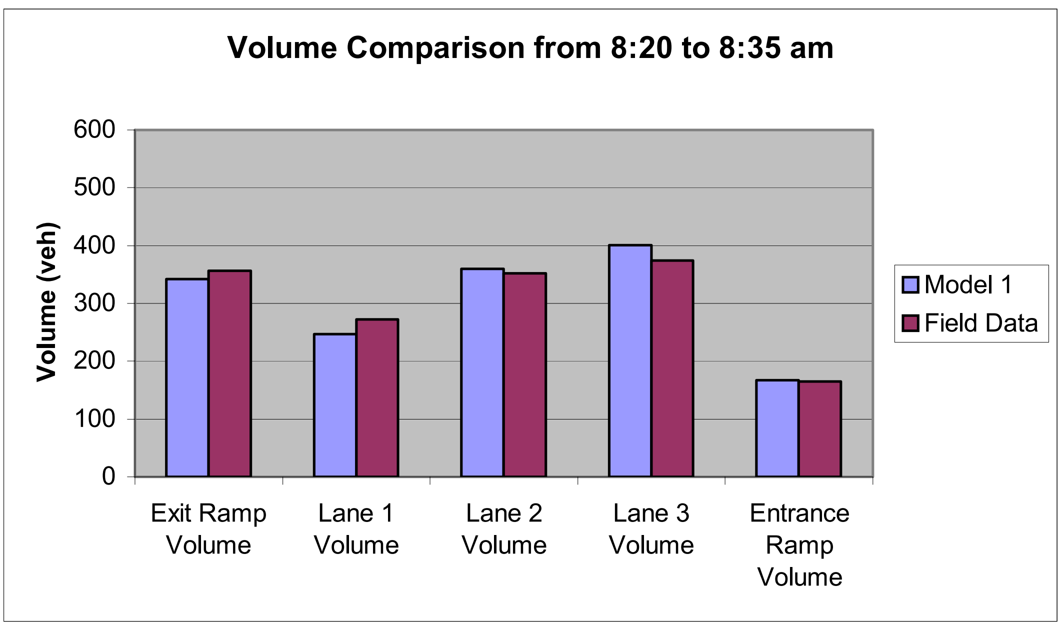

Figure 5.4 Volume Comparison from 8:20 to 8:35 a.m. for Model 1

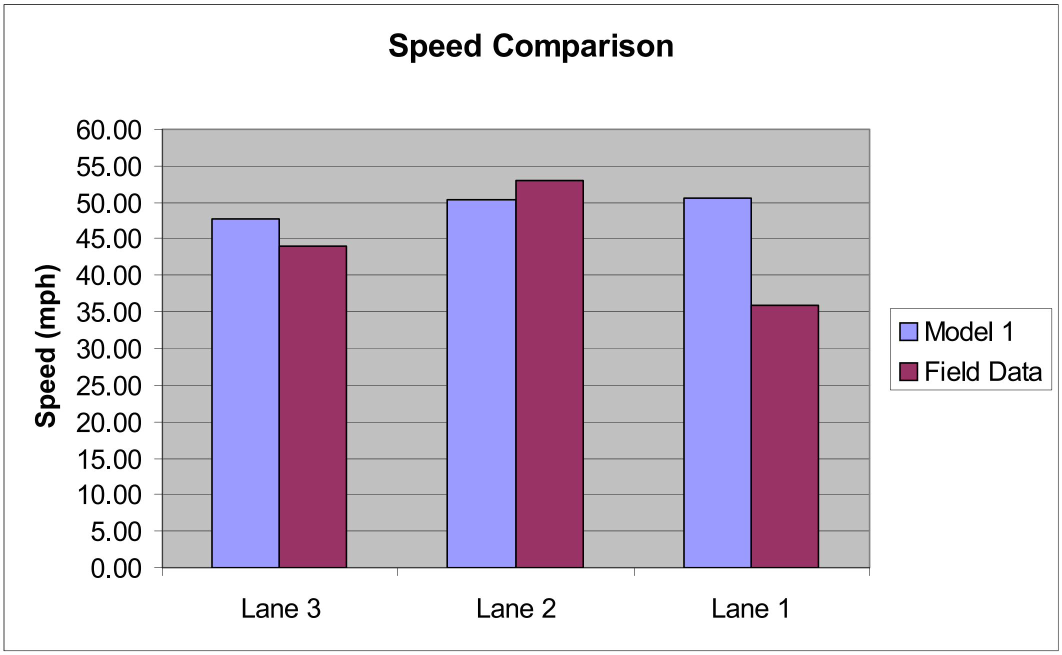

Figure 5.5 Speed Comparison for Model 1

5.5.1.3 Model 2: Look back distance 2500 ft from exit ramp and 200 ft acceleration lane length

In this model, the look back distance was decreased to 2500 ft from the exit ramp while the acceleration lane length was increased to 200 ft. The purpose of increasing the length of acceleration lane was to reduce the queue length at entrance ramp. As a result, model 2 appeared to behave in a similar fashion to the field conditions. Volumes on lanes 1, 2, and 3 in this model are closer to the field data comparing to model 1 as a result of decreasing look back distance. The volume and speed comparisons between simulation output and field data for this model are presented in Figure 5.6 to Figure 5.10. Even though the volumes slightly change on lane 1 and 3, the speed remains the same for both model 1 and 2.

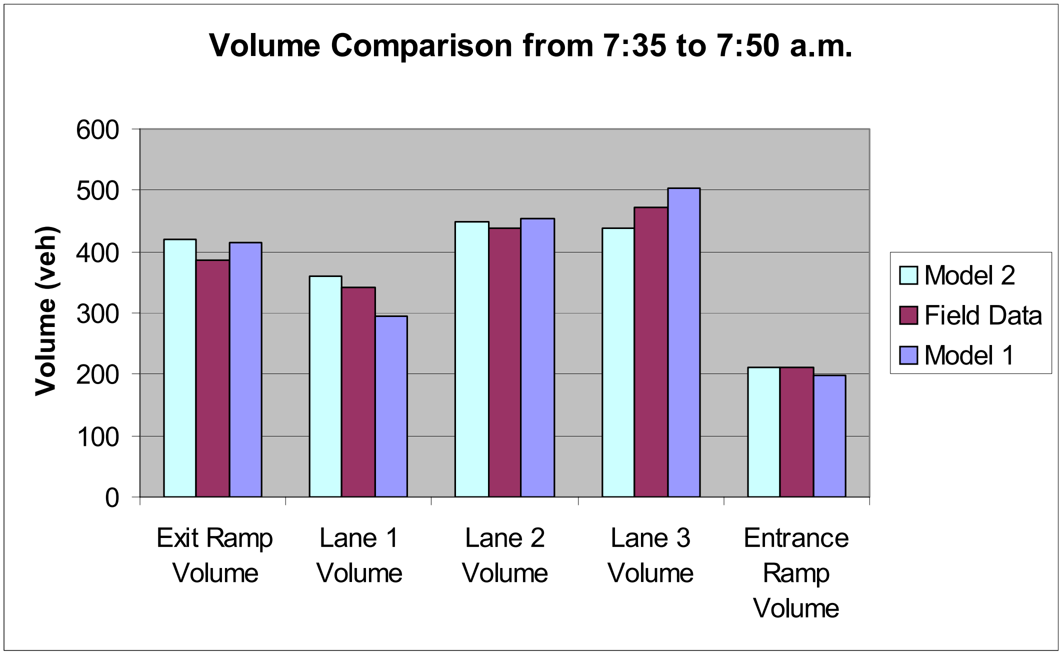

Figure 5.6 Volume Comparison from 7:35 to 7:50 a.m. for Model 2

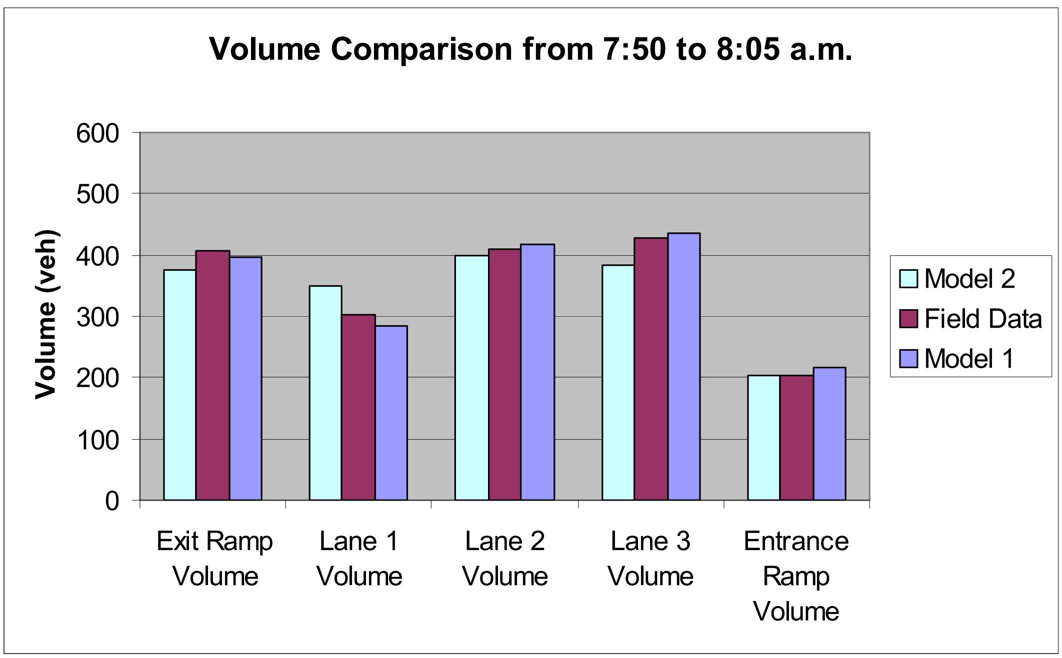

Figure 5.7 Volume Comparison from 7:50 to 8:05 a.m. for Model 2

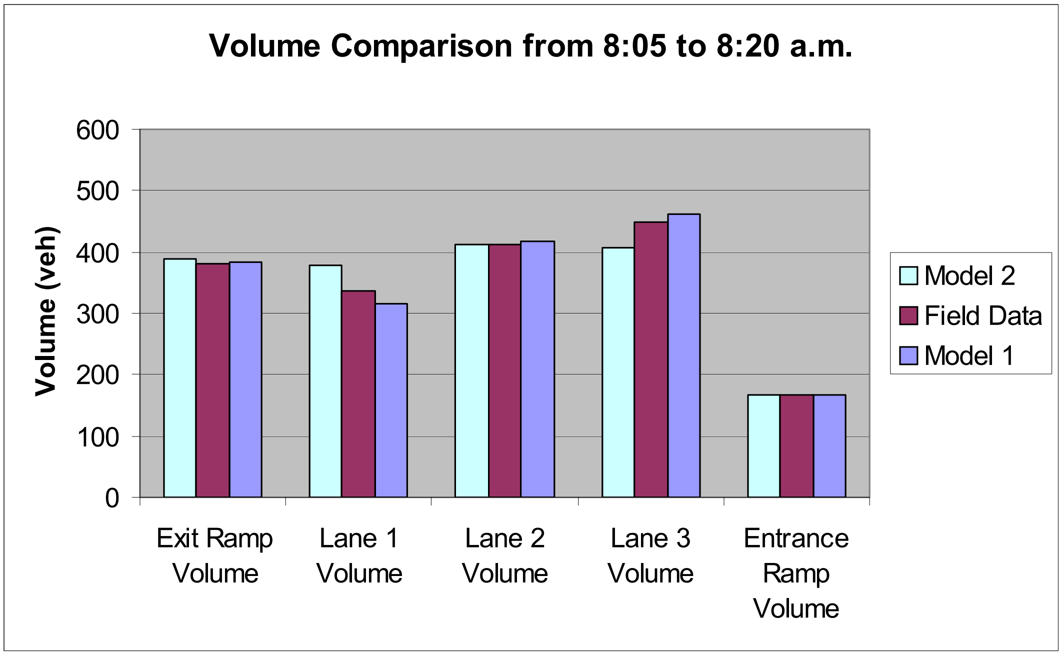

Figure 5.8 Volume Comparison from 8:05 to 8:20 a.m. for Model 2

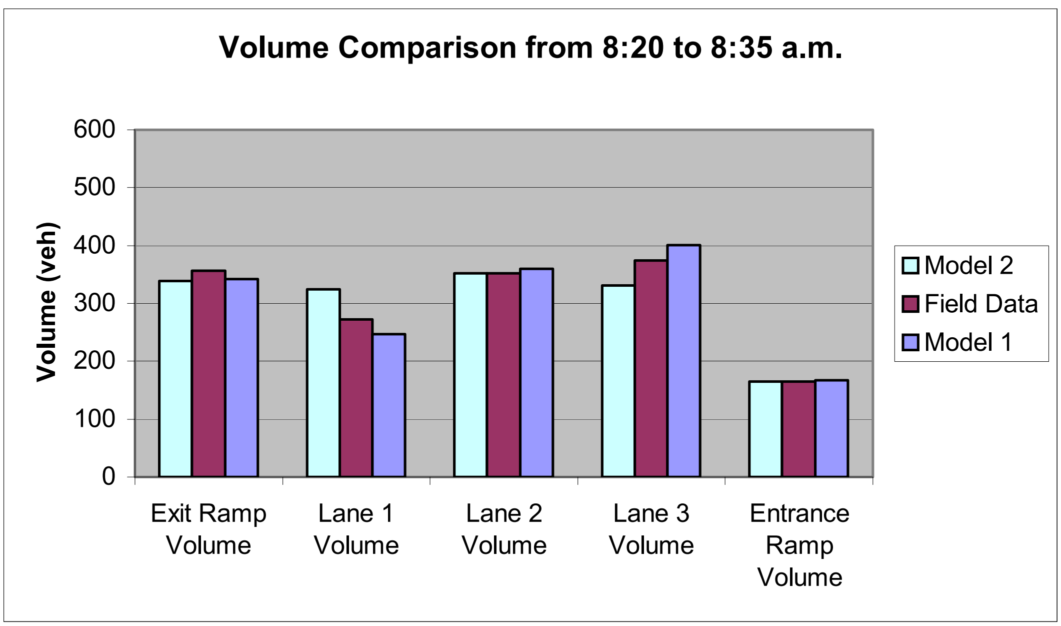

Figure 5.9 Volume Comparison from 8:20 to 8:35 a.m. for Model 2

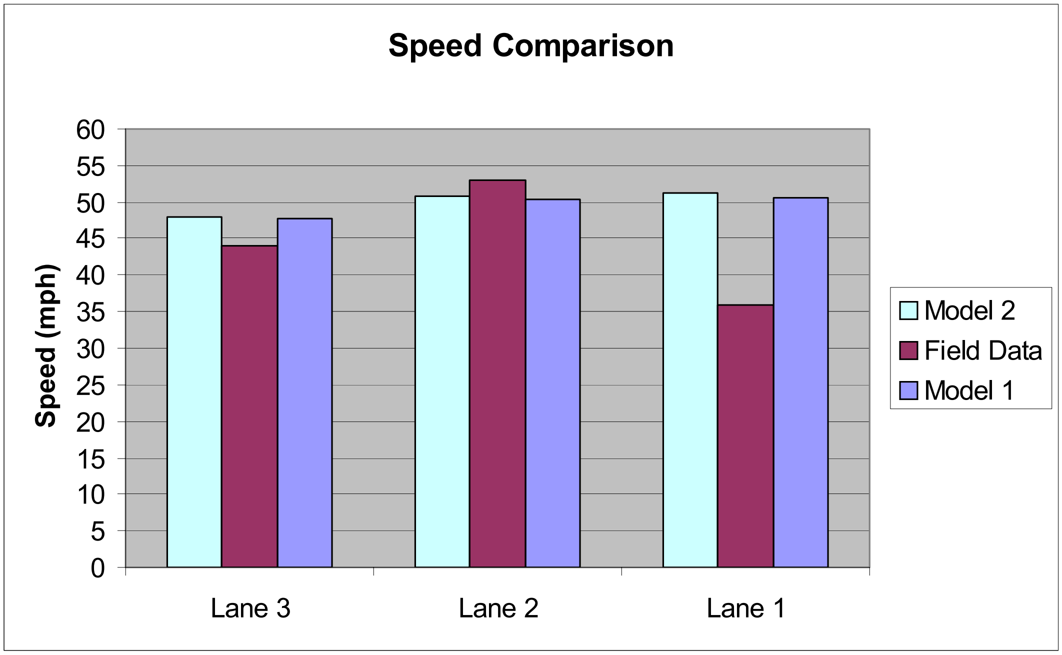

Figure 5.10 Speed Comparison for Model 2

5.5.1.4 Model 3: Look back distance 2100 ft from exit ramp and 250 ft acceleration lane length

The look back distance in this model is 2100 ft from the exit ramp and the acceleration lane length was increased to 250 ft. The volume and speed comparison between simulation output and field data for this model is presented in Figures 5.11 to 5.15. Lane 1 volumes in model 3 are larger than the field values and the volume of lane 3 in model 3 is smaller than the field volume because the point where vehicles start to make their lane changes was pushed 400 ft downstream compared to model 2. The speed does not change between the three models.

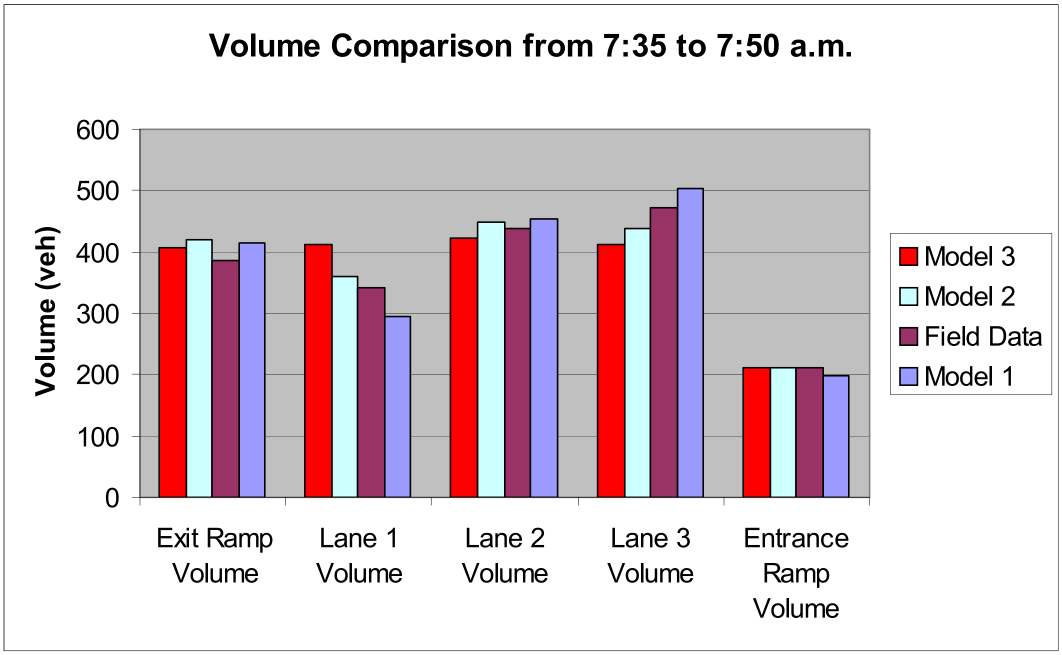

Figure 5.11 Volume Comparison from 7:35 to 7:50 a.m. for Model 3

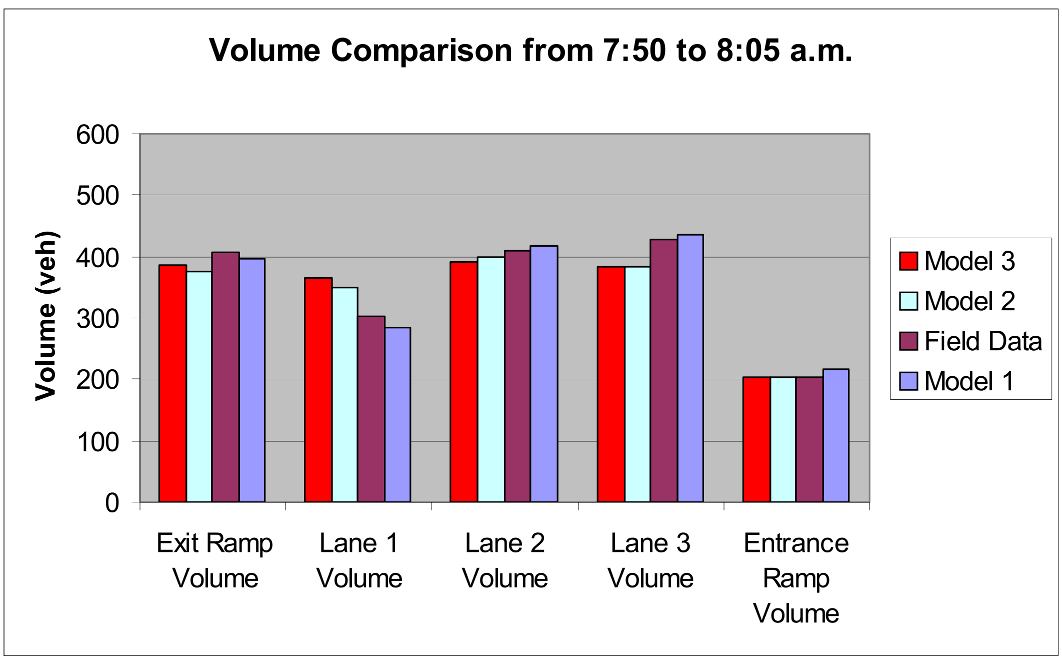

Figure 5.12 Volume Comparison from 7:50 to 8:05 am for Model 3

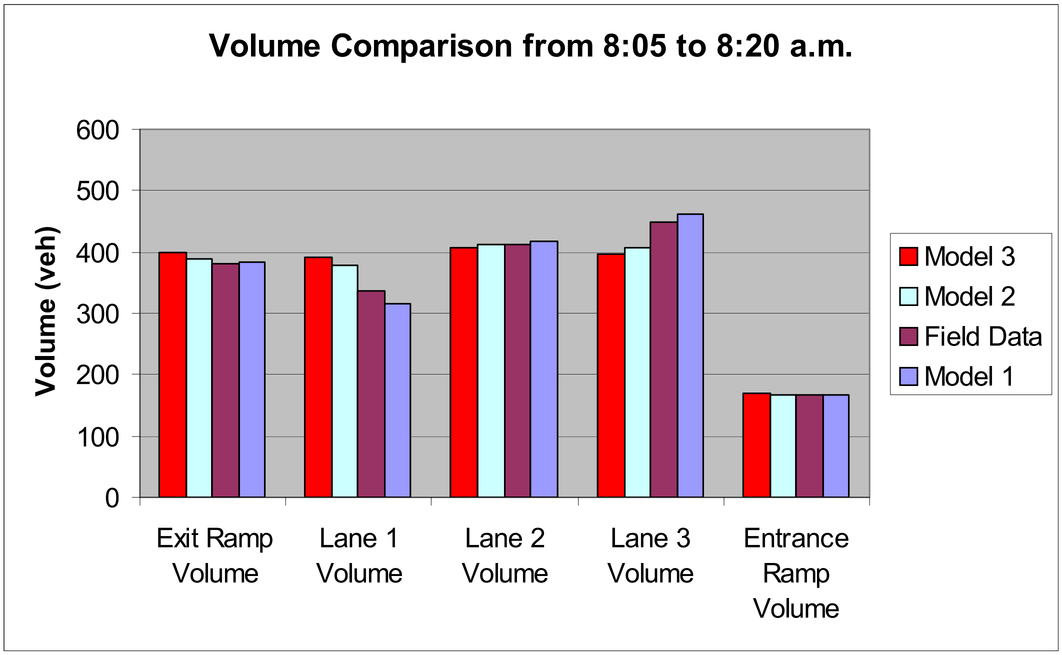

Figure 5.13 Volume Comparison from 8:05 to 8:20 a.m. for Model 3

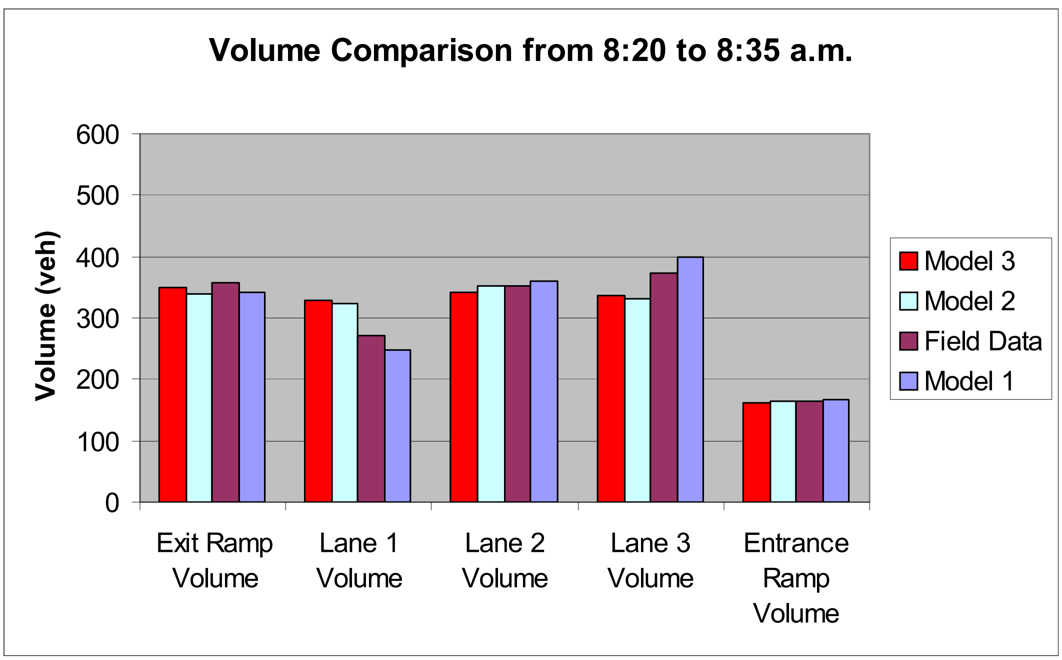

Figure 5.14 Volume Comparison from 8:20 to 8:35 a.m. for Model 3

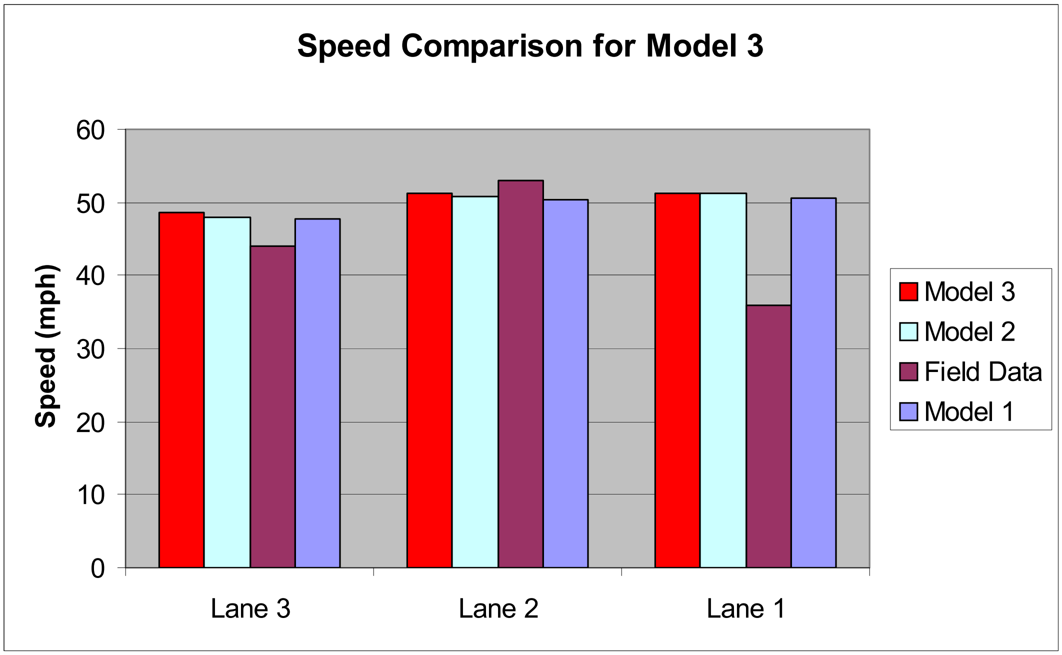

Figure 5.15 Speed Comparison for Model 3

5.5.2 Models with Changes in the Car Following Model

In this section, in addition to the changes in look back distance and acceleration lane length, the values of CC0, CC1, and CC4/CC5 in the car following model were adjusted in order to find the best model that can duplicate the field conditions. The car following model was adjusted to calibrate travel speeds, which were far higher than the field values in the previous models. The range of CCO, CC1, and CC4/CC5 values used to calibrate was shown in Table 5.1.

5.5.2.1 Model 4: Look back distance 2500 ft from exit ramp, 200 ft acceleration lane length, CC0= 1.7, CC1= 0.9, CC4=-2.0, and CC5=2.0

The volume and speed comparison between simulation outputs and field data for this model are presented in Figures 5.16 to 5.20. Volume and speed on lane 3 in this model are closer to the field values compared to the previous models. Even though the speed on the right lane is still higher than the field speed, it is the closest to field data when compared to previous models. As in other models, lane 1 volume in this model is still higher than field volume during most of the simulation time.

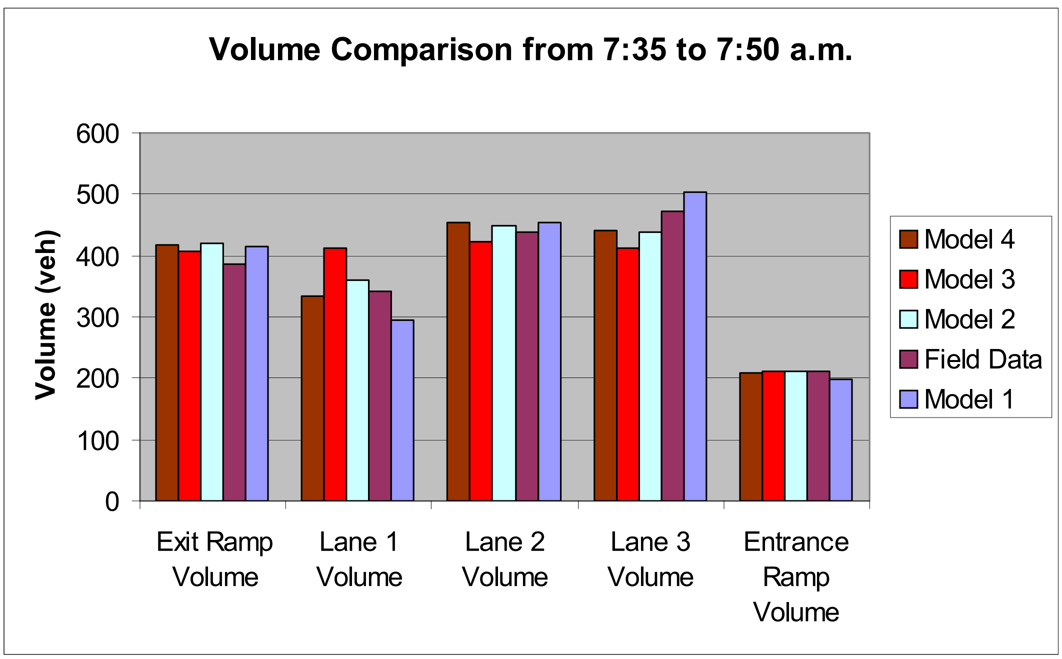

Figure 5.16 Volume Comparison from 7:35 to 7:50 a.m. for Model 4

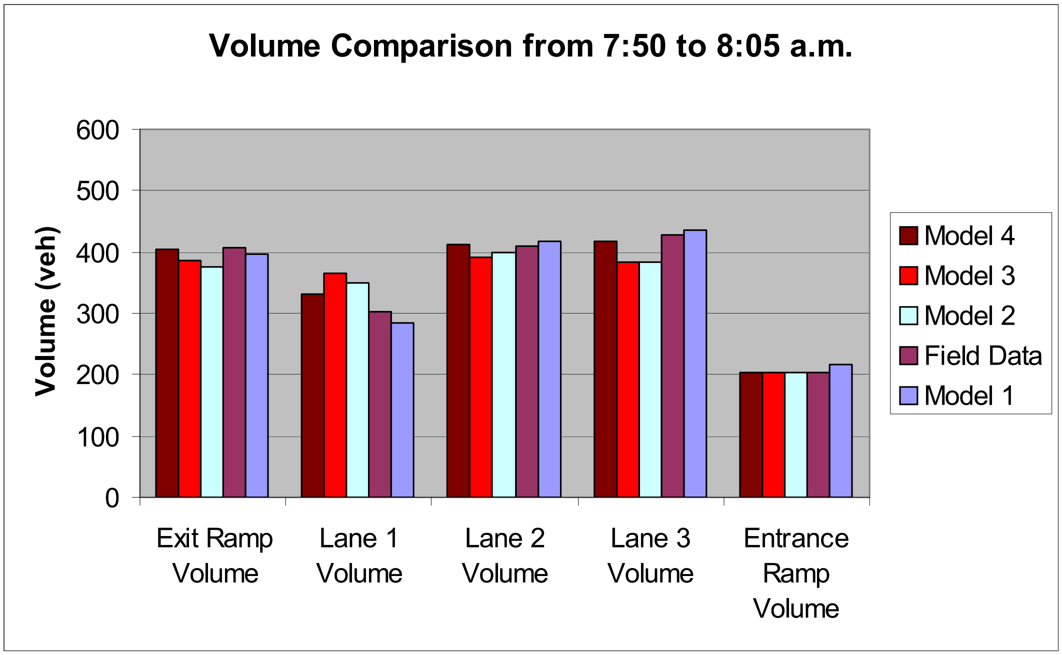

Figure 5.17 Volume Comparison from 7:50 to 8:05 a.m. for Model 4

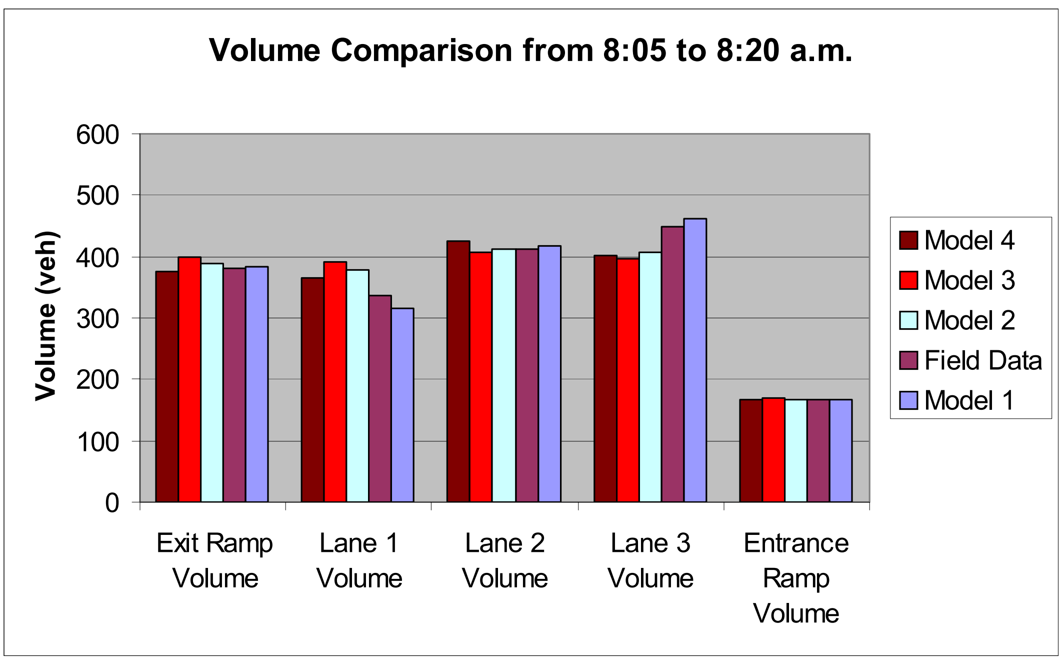

Figure 5.18 Volume Comparison from 8:05 to 8:20 a.m. for Model 4

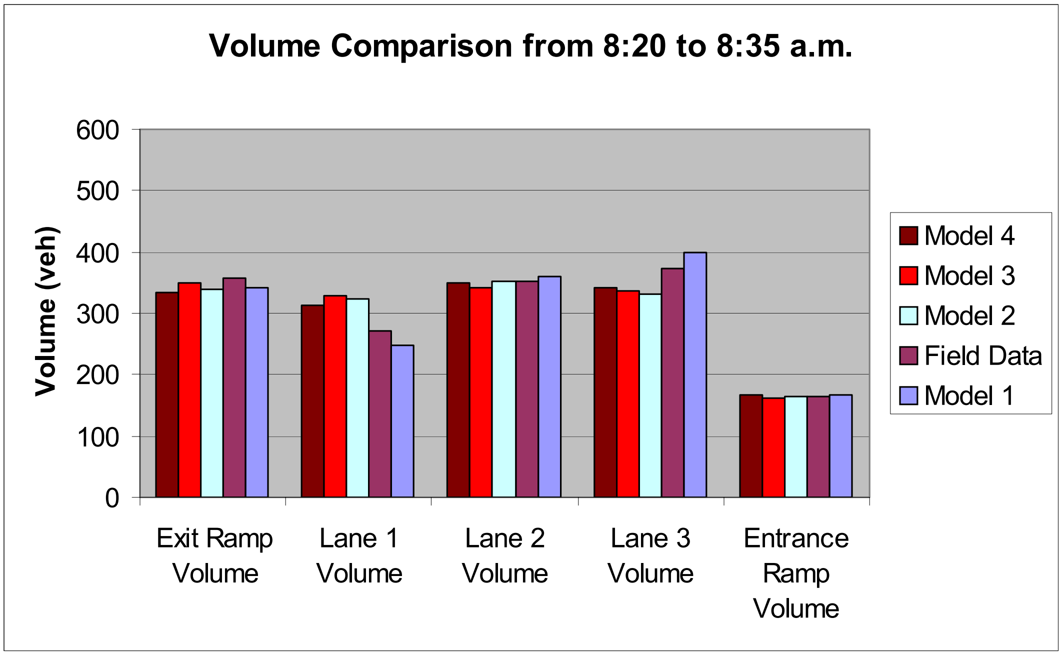

Figure 5.19 Volume Comparison from 8:20 to 8:35 a.m. for Model 4

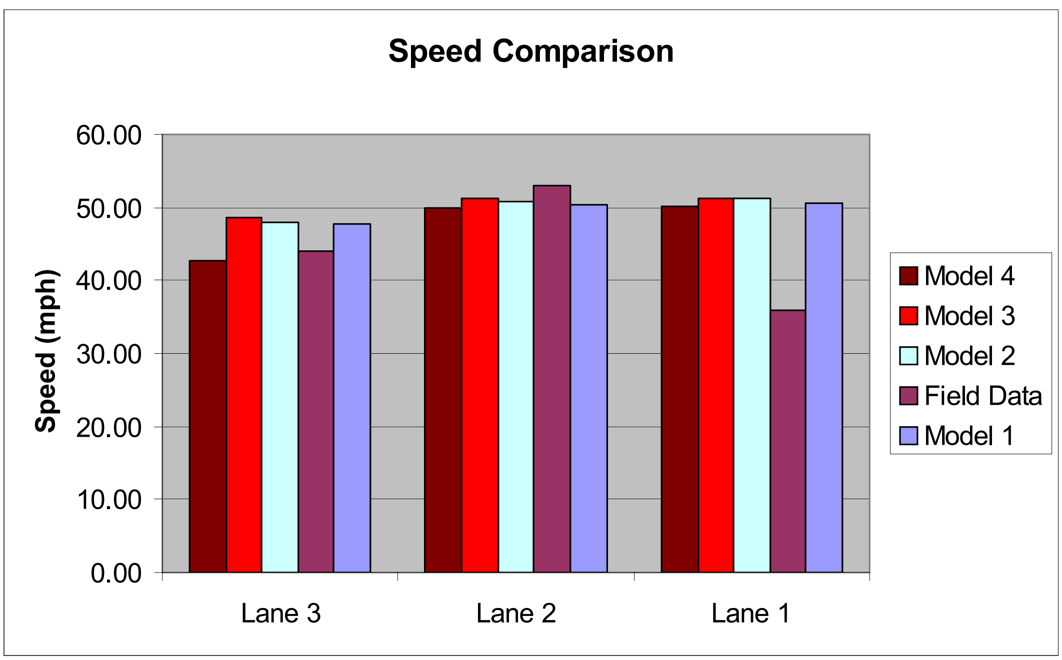

Figure 5.20 Speed Comparison for Model 4

5.5.2.2 Model 5: Look back distance 2200 ft from exit ramp, 200 ft acceleration lane length, CC0= 1.7, CC1= 1.1, CC4=-2.0, and CC5=2.0

CC1 is the minimum time (in seconds) that the driver wants to keep behind the leading vehicle. The higher CC1 is, the more cautious the driver. Thus, CC1 was increased in order to decrease the travel speed in a weaving section. The look back distance was also slightly decreased compared to model 4. The volume and speed comparison between the simulation output and field data for this model is presented in Figures 5.21 to 5.25. The output of this model is similar to model 4. After this model, it can be concluded that slight changes in CC1 do not change travel speed.

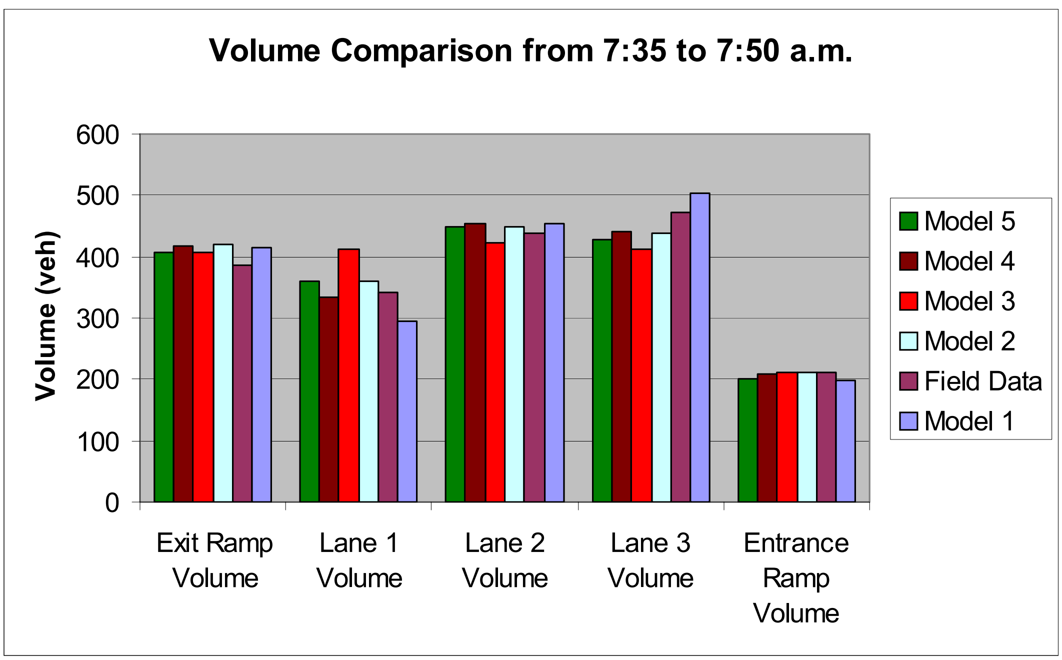

Figure 5.21 Volume Comparison from 7:35 to 7:50 a.m. for Model 5

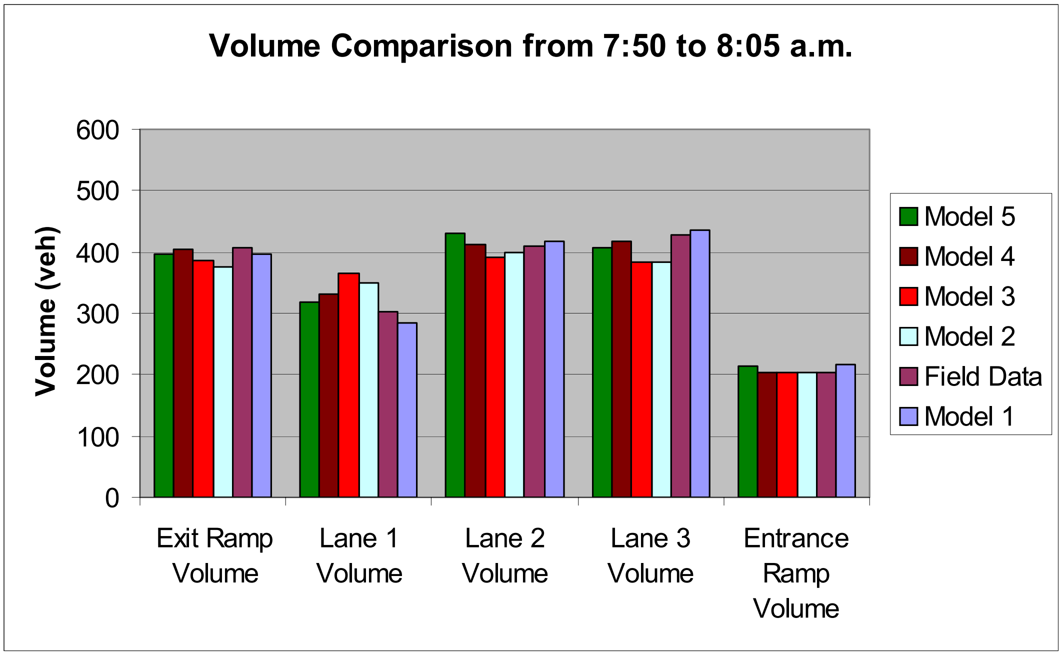

Figure 5.22 Volume Comparison from 7:50 to 8:05 a.m. for Model 5

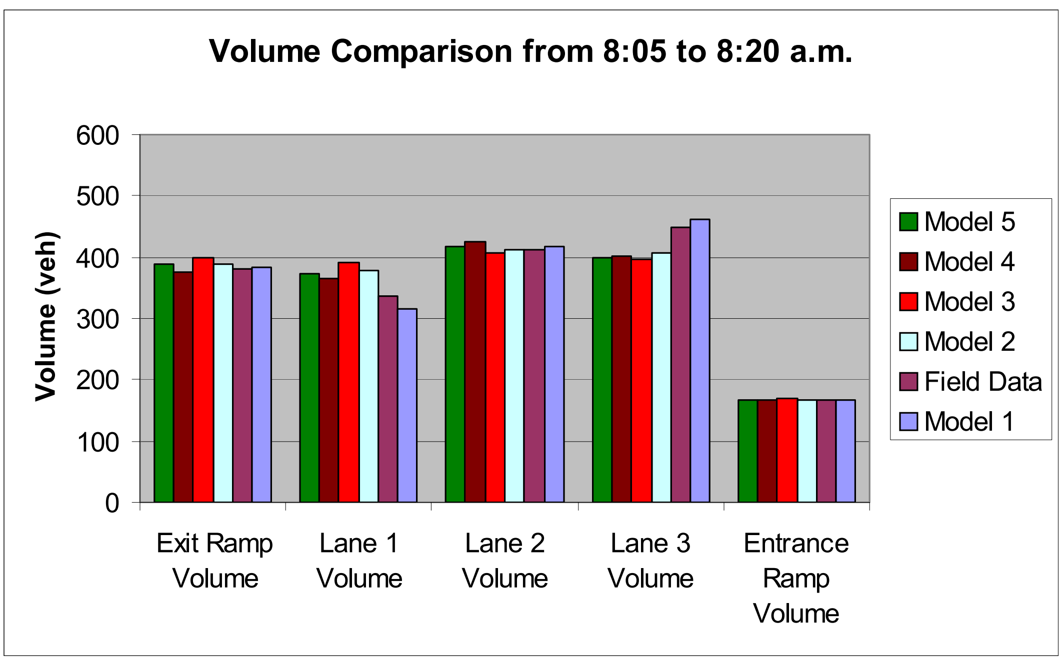

Figure 5.23 Volume Comparison from 8:05 to 8:20 a.m. for Model 5

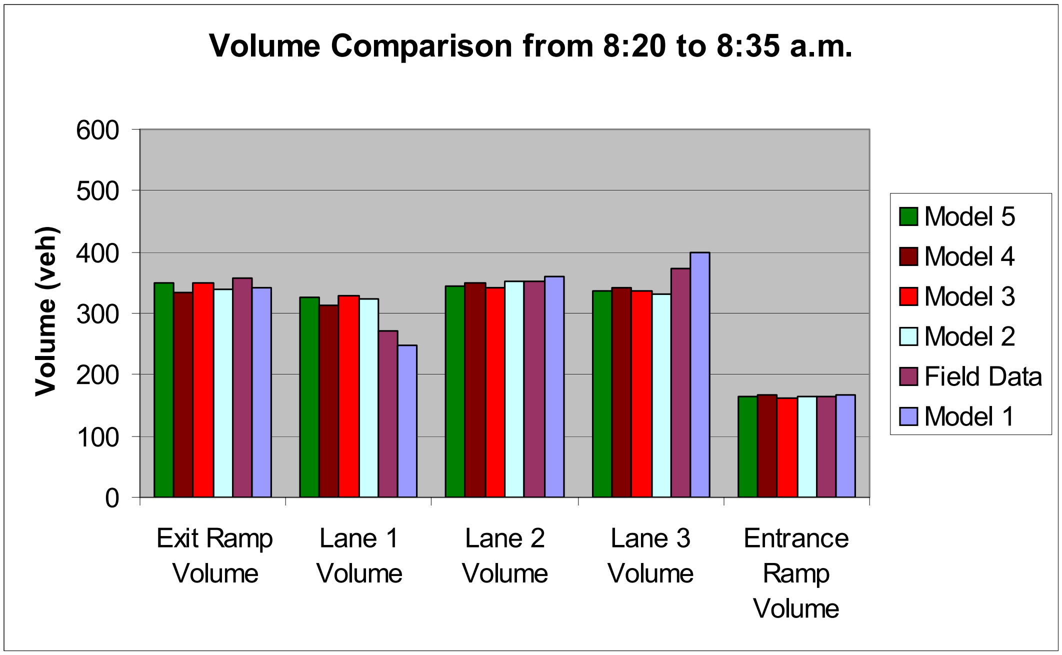

Figure 5.24 Volume Comparison from 8:20 to 8:35 a.m. for Model 5

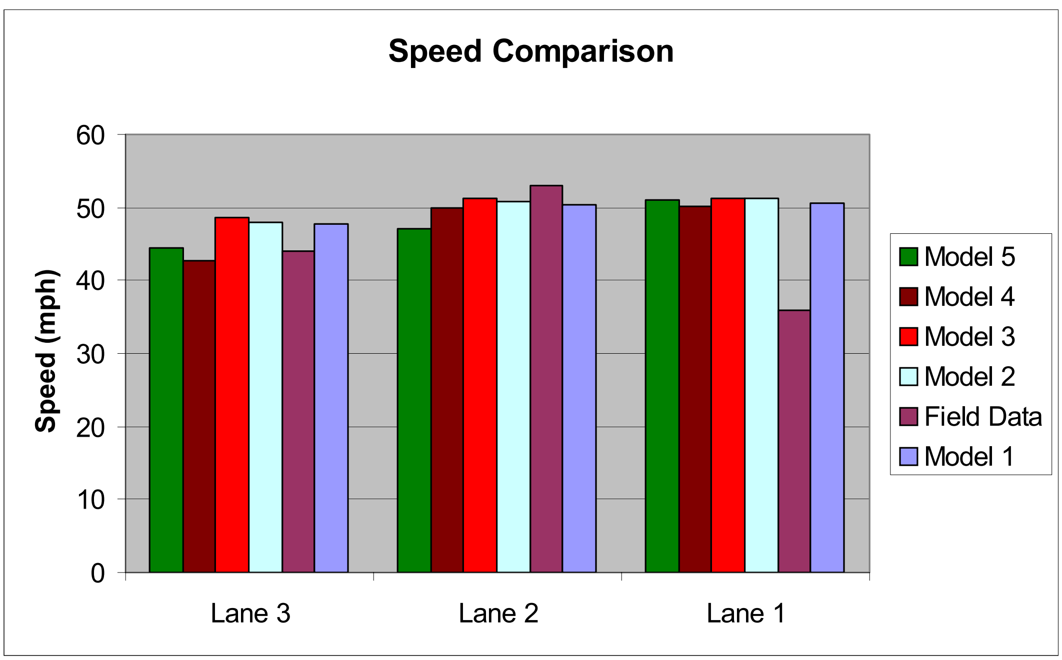

Figure 5.25 Speed Comparison for Model 5

5.5.3 Models with Changes in the Car Following Model and Lane Change Model

In this section, the minimum headway from the front bumper of the trailing vehicle to rear bumper of lead vehicle was changed. The purpose of this adjustment was to see how the system responds to various values of minimum headway. The default value for minimum headway is 1.64 ft.

5.5.3.1 Model 6: Look back distance 2200 ft from exit ramp, 200 ft acceleration lane length, CC0=1.7, CC1=1.1, CC4=-2.0, and CC5=2.0, minimum headway (front/rear) =3 ft

In this model, minimum headway was changed from 1.64 to 3 ft. Look back distance was unchanged. The volume and speed comparison between the simulation output and field data for this model is presented in Figures 5.26 to 5.30. Lane 3 volume and speed in the middle lane are closer to the field values in this model compared to previous models. Thus, compared to model 5, this model more closely reflected the field data.

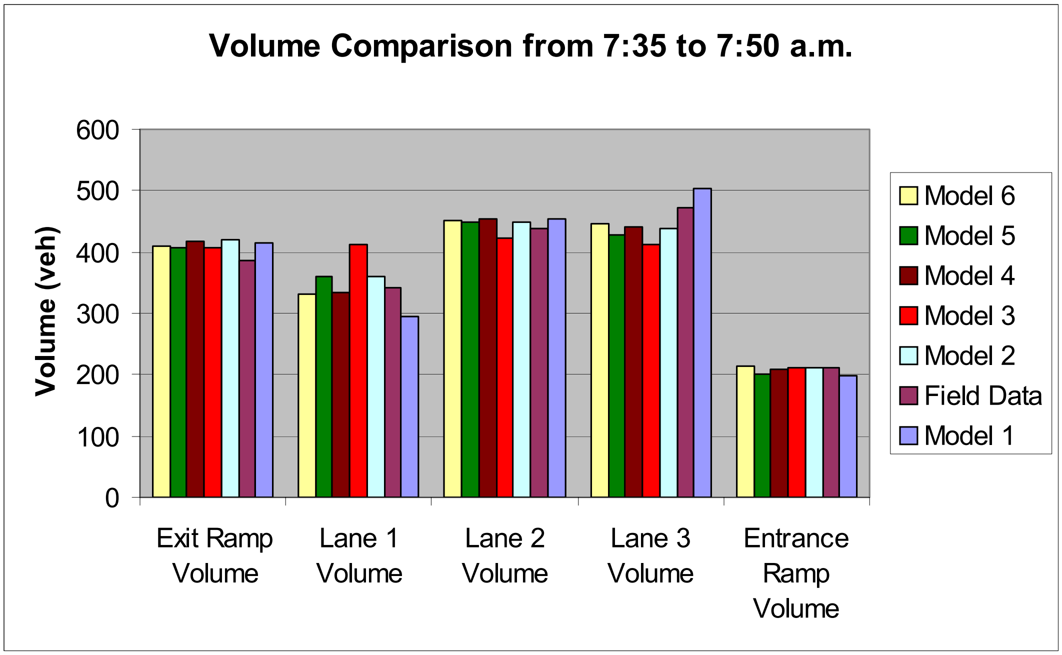

Figure 5.26 Volume Comparison from 7:35 to 7:50 a.m. for Model 6

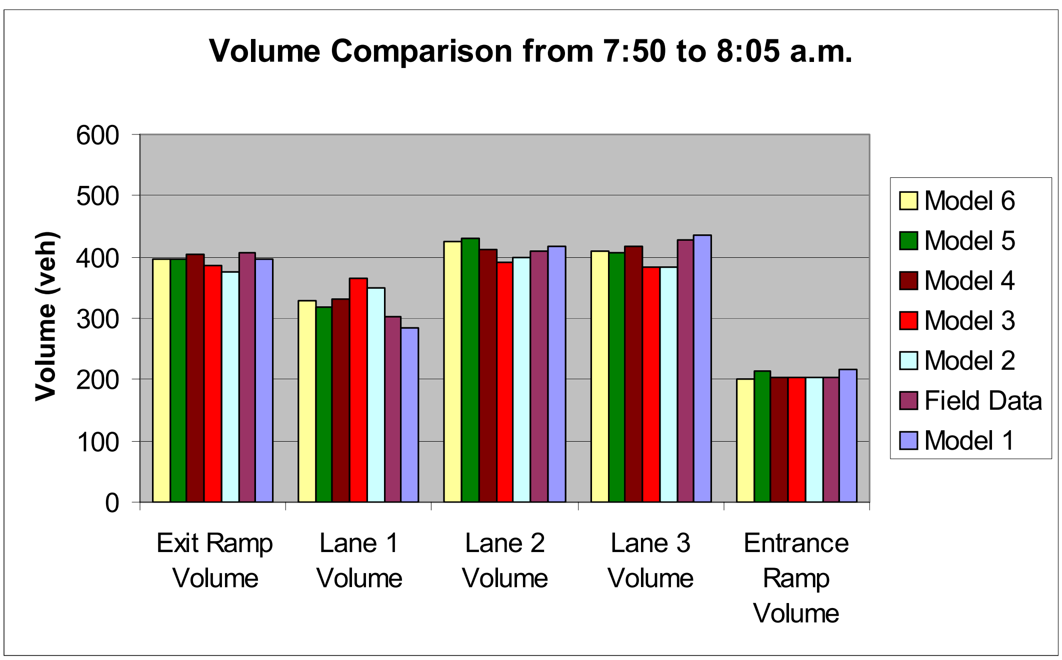

Figure 5.27 Volume Comparison from 7:50 to 8:05 a.m. for Model 6

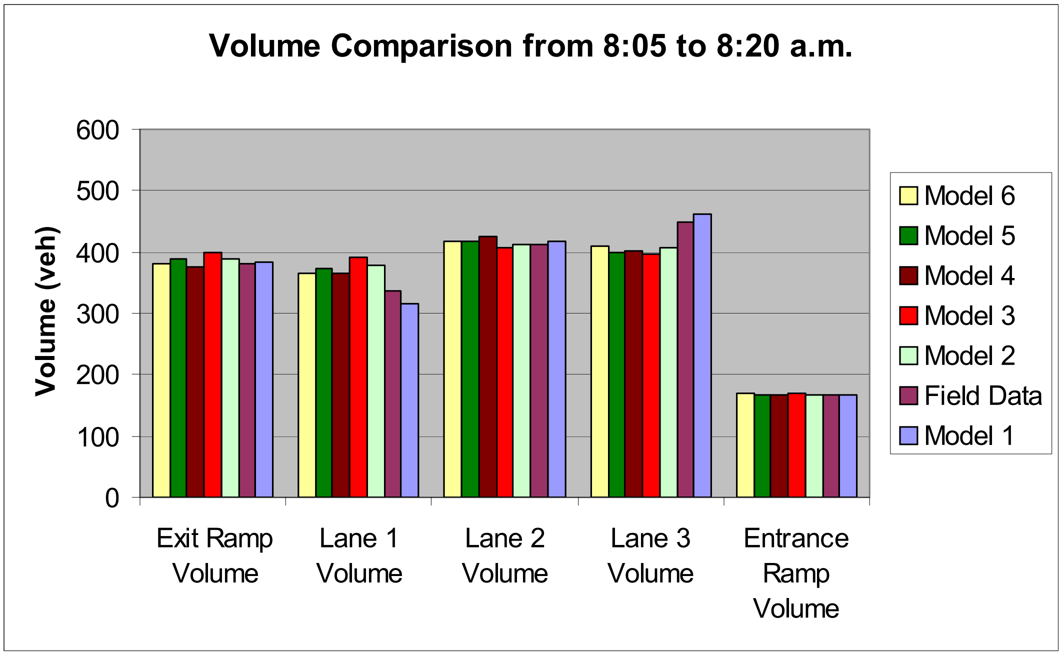

Figure 5.28 Volume Comparison from 8:05 to 8:20 a.m. for Model 6

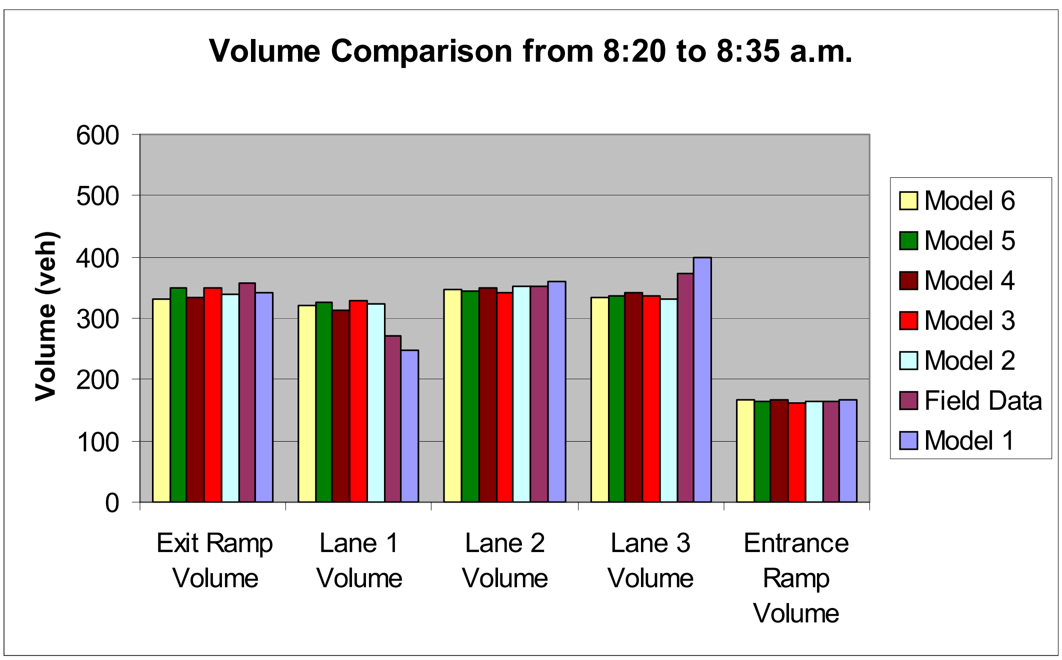

Figure 5.29 Volume Comparison from 8:20 to 8:35 a.m. for Model 6

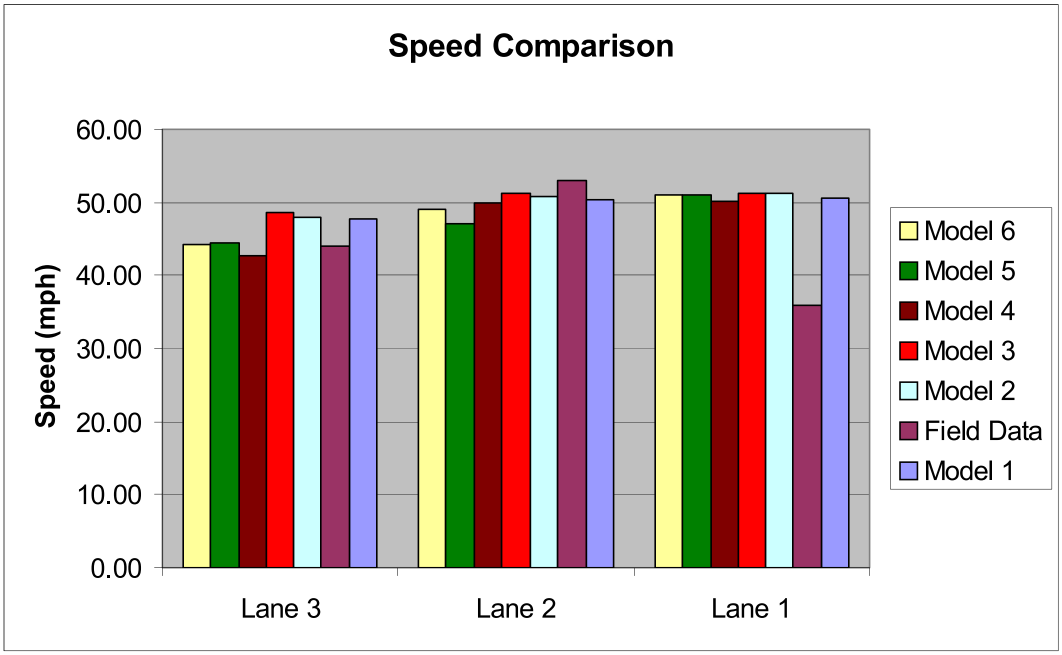

Figure 5.30 Speed Comparison for Model 6

5.5.3.2 Model 7: Look back distance 2500 ft from exit ramp, 200 ft acceleration length, CC0=1.7, CC1=1.1, CC4=-2.0, and CC5=2.0, minimum headway (front/rear)=3 ft

Only the look back distance was changed in this model. Other parameters remained unchanged from model 6. The increase of the look-back distance was intended to increase the lane 3 volume and to decrease lane 1 volume in this model. The volume and speed comparison between the simulation output and field data for this model is presented in Figures 5.31 to 5.35. As seen in Figures 5.31 to 5.34, the volumes in lane 1 and lane 3 are closer to the field values compared with all previous models; moreover, speed in the left lane is also closer to the observed speed.

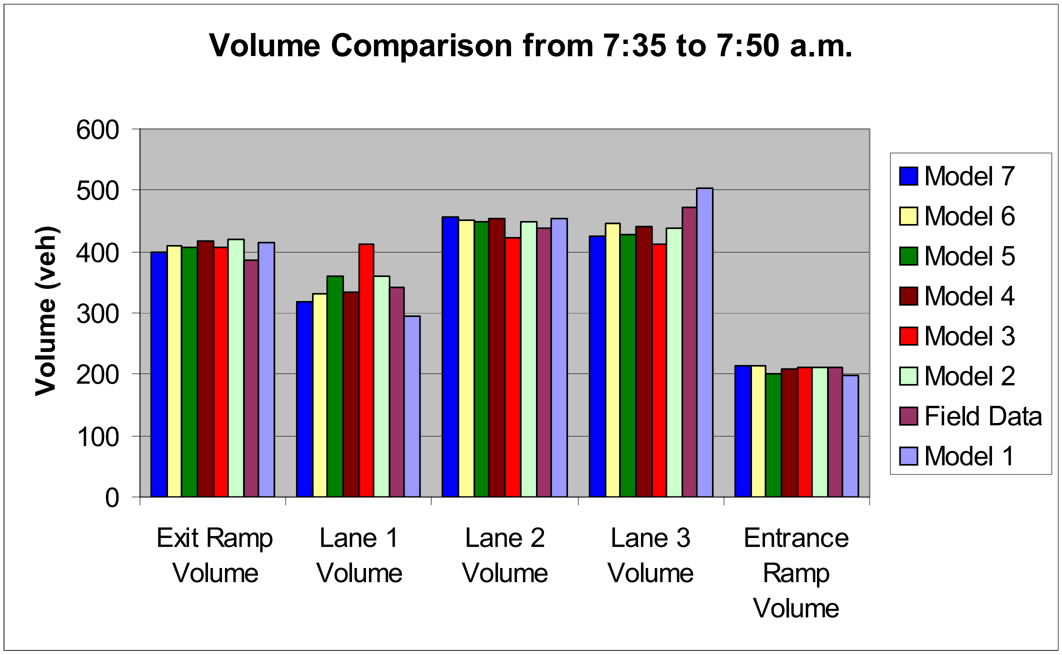

Figure 5.31 Volume Comparison from 7:35 to 7:50 a.m. for Model 7

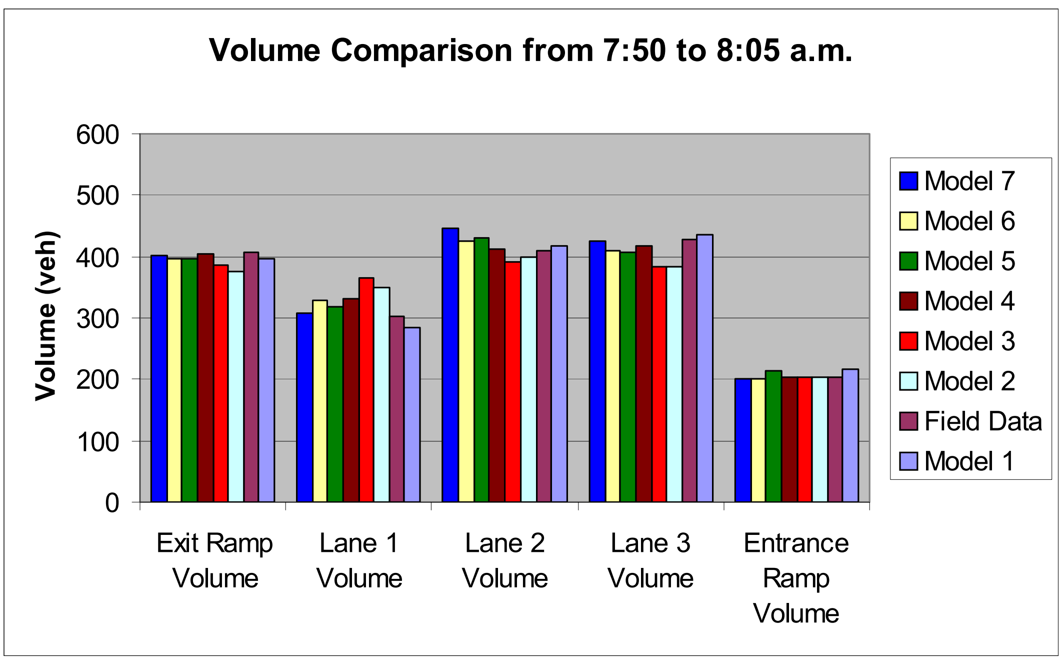

Figure 5.32 Volume Comparison from 7:50 to 8:05 a.m. for Model 7

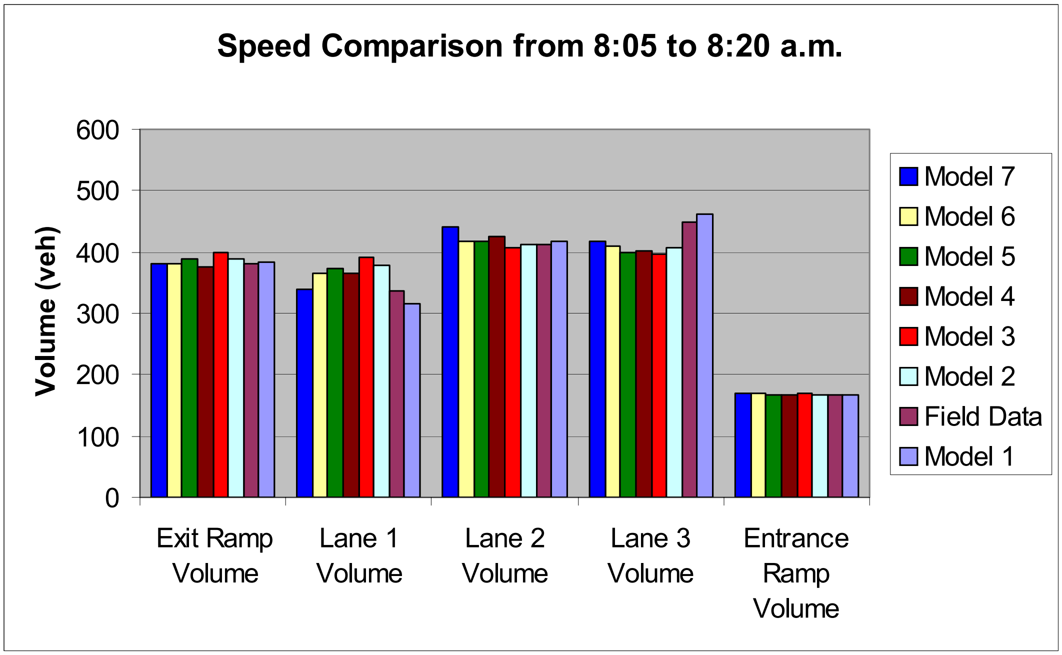

Figure 5.33 Volume Comparison from 8:05 to 8:20 a.m. for Model 7

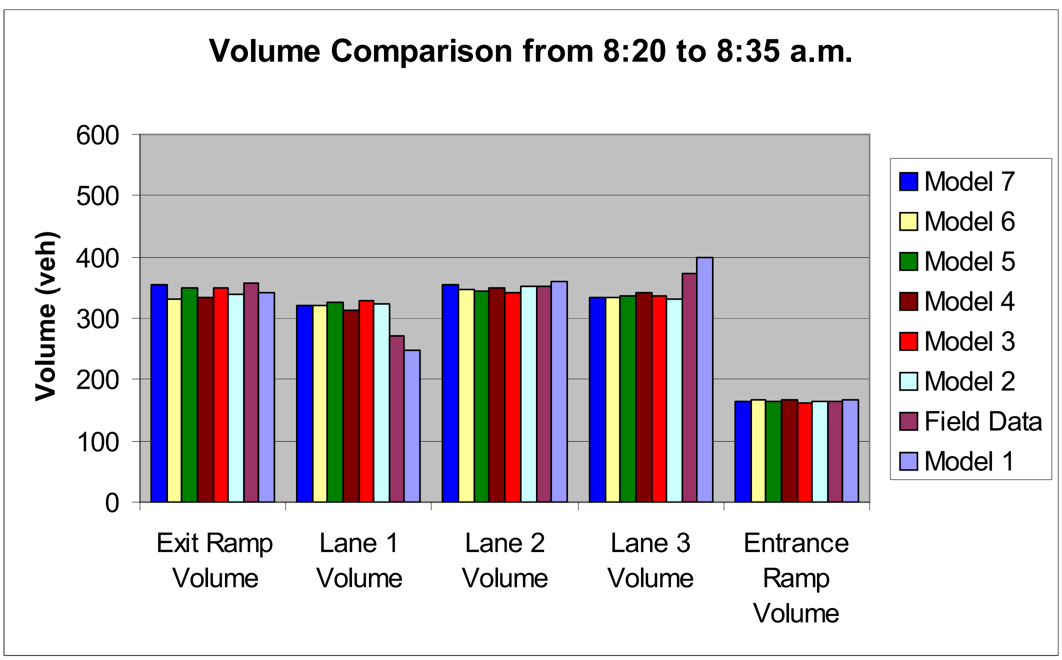

Figure 5.34 Volume Comparison from 8:20 to 8:35 a.m. for Model 7

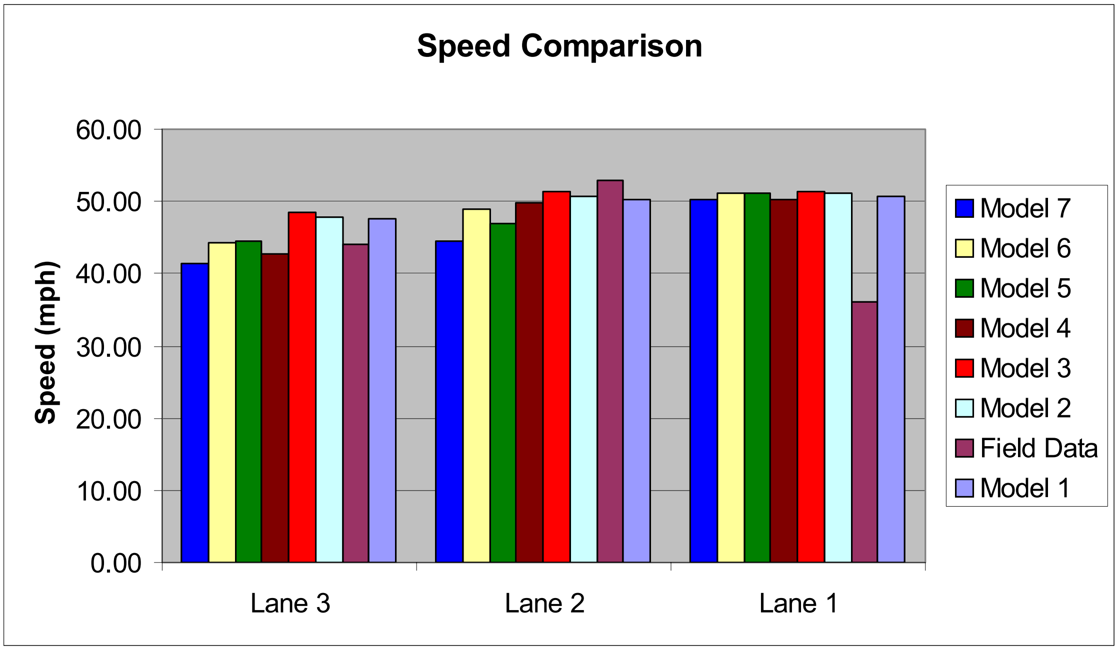

Figure 5.35 Speed Comparison for Model 7

5.6 Model Selection

Seven models were introduced in the previous section. Each model performs well with some measure of effectiveness (MOEs) but performs poorly with others. In order to select the best model from those introduced in section 5.5, a multi-criteria analysis process was conducted using the Simple Additive Weighting (SAW) method, which is widely used in multi-criteria methods [Ref. 33]. In this method, a score is obtained by adding the contributions from each of the chosen criteria. The overall weight score, V_a, for model i, using this method can be written as follows:

$$V_a = \sum_{j=1}^{j=n} w_j r_{aj} \tag{5.2}$$

where

w_j = weight for criterion j

r_{aj} = rating for model a on criterion j

The model with the largest value of V_a is selected.

Twenty-three criteria are used to rate each model; there are the volume for the exit ramp, lane 1, lane 2, lane 3, and entrance ramp for each 15-minute period, and the speed in the lane 3, lane 2, and lane 1. Each of the 23 criteria in this SAW model has the same weight score. The criterion with the largest difference to the field value has the rating of 0 and the field data has the rating of 1. The largest possible score for each model is 23. The raw criterion scores on each criteria option of each model are determined by the difference between the criteria value on each model and the field value. Table 5.4 explains how the raw scores on the first row of Table 5.5 were determined.

Table 5.4 Example of Raw Criterion Score Calculation for the Exit Ramp Volume between 7:35 to 7:50 a.m.

	Exit Ramp Volume	Difference	Raw Score
Model 7	399	14	0.611
Model 6	410	25	0.306
Model 5	407	22	0.389
Model 4	419	34	0.056
Model 3	408	23	0.361
Model 2	421	36	0.00
Field Data	385	0	1.00
Model 1	415	30	0.167

Since the field value has the highest score (1) and Model 2 with the largest difference to the field value has the lowest score (0) (table 5.4), the raw score of other models are calculated using equation 5.3:

$$Raw\ Score = 1 - \frac{difference}{\max\ difference} \tag{5.3}$$

The technique described above can quantify the performance of each model compared to the field data. The model with the highest score out of 23 MOEs will be the best candidate for validation.

Table 5.5 indicates the raw scores for each model on 23 criteria.

Table 5.5 Raw Criterion Scores

		Model						
Time	Criteria	1	2	3	4	5	6	7
7:35 to 7:50	Exit Ramp	0.157	0.000	0.370	0.065	0.389	0.315	0.611
	Lane 1	0.314	0.734	0.000	0.879	0.754	0.826	0.647
	Lane 2	0.158	0.316	0.228	0.158	0.333	0.281	0.000
	Lane 3	0.494	0.450	0.000	0.461	0.267	0.578	0.200
	Entrance Ramp	0.000	0.974	0.897	0.897	0.179	0.821	0.821
7:50 to 8:05	Exit Ramp	0.677	0.000	0.323	0.938	0.708	0.635	0.844
	Lane 1	0.714	0.249	0.000	0.524	0.735	0.587	0.894
	Lane 2	0.806	0.722	0.500	0.907	0.463	0.574	0.000
	Lane 3	0.822	0.016	0.000	0.798	0.527	0.597	0.961
	Entrance Ramp	0.000	0.917	0.861	0.889	0.111	0.750	0.722
8:05 to 8:20	Exit Ramp	0.863	0.510	0.000	0.627	0.569	0.941	1.000
	Lane 1	0.612	0.212	0.000	0.455	0.303	0.485	0.958
	Lane 2	0.817	0.946	0.849	0.570	0.796	0.753	0.000
	Lane 3	0.764	0.224	0.000	0.109	0.079	0.261	0.424
	Entrance Ramp	1.000	0.750	0.000	0.625	0.875	0.126	0.501
8:20 to 8:35	Exit Ramp	0.413	0.280	0.720	0.080	0.667	0.000	0.960
	Lane 1	0.560	0.060	0.000	0.250	0.054	0.149	0.143

Table 5.5-continued

		Model						
Time	Criteria	1	2	3	4	5	6	7
8:20 to 8:35	Lane 2	0.185	1.000	0.000	0.815	0.111	0.407	0.778
	Lane 3	0.388	0.000	0.124	0.233	0.124	0.054	0.093
	Entrance Ramp	0.002	0.800	0.000	0.002	1.000	0.601	0.800
Speed	Lane 3	0.199	0.149	0.000	0.726	0.898	0.962	0.420
	Lane 2	0.680	0.729	0.795	0.627	0.296	0.523	0.000
	Lane 1	0.045	0.006	0.000	0.071	0.014	0.019	0.064
Total Scores		11.040	10.995	8.070	14.605	13.775	15.822	17.582

Since the weight for each criterion is the same, the models with the highest total criterion score is the selected. The top two models, model 4 and model 7, are selected for validation.

5.7 Validation

Models 4 and 7 had the highest criterion scores and were selected to be validated using different data sets (4:00 to 5:00 p.m., and 5:00 to 6:00 p.m.). The number of runs is also calculated for model 4 using the same equation found in section 5.4. Two major parameters, the volumes for each lane from 4:00 to 4:15 p.m. and speed, are selected to calculate the number of runs.

As in the previous section, the tolerance for volume and speed were 20 vph and 2 mph respectively (tables 5.6 and 5.7). The normal score for 95% confidence is 1.96. Although only two run were needed for validation, three runs were conducted and the mean of the three runs was used to compare models 4 and 7 with field data.

Table 5.6 Volume on Each Lane for Different Random Seed for Model 4
from 4:00 to 4:15 p.m.

	Exit Ramp Volume	Lane 1	Lane 2	Lane 3	Entrance Ramp
Random Seed 12	509	295	302	301	199
Random Seed 15	477	291	308	297	196
Random Seed 18	478	294	298	299	198
Random Seed 20	480	299	304	299	198
Standard Deviation	15	3	4	2	1
Tolerance e	20	20	20	20	20
95% confidence	1.96	1.96	1.96	1.96	1.96
n	2	1	1	1	1

Table 5.7 Speeds for Different Random Seed for Model 4 from 4:00 to 5:00 p.m.

	Lan 3	Lane 2	Lane 1
Random Seed 12	43.43	51.82	51.22
Random Seed 15	47.3	49.95	51.22
Random Seed 18	44.76	51.57	51.2
Random Seed 20	45.06	52.78	51.5
Standard deviation	1.61	1.18	0.14
Tolerance e	2	2	2
95% confidence	1.96	1.96	1.96
n	2	1	1

Figures 5.36 to 5.45 show the comparison of lane volumes and speed between models 4 and 7 for different traffic data from 4:00 to 6:00 p.m. From these figures, one can see that the volumes from models 4 and 7 are very similar; however, the speed outputs from model 7 are better than the speed outputs of model 4. As a result, model 7 seems to be the best candidate for the calibrated simulation model.

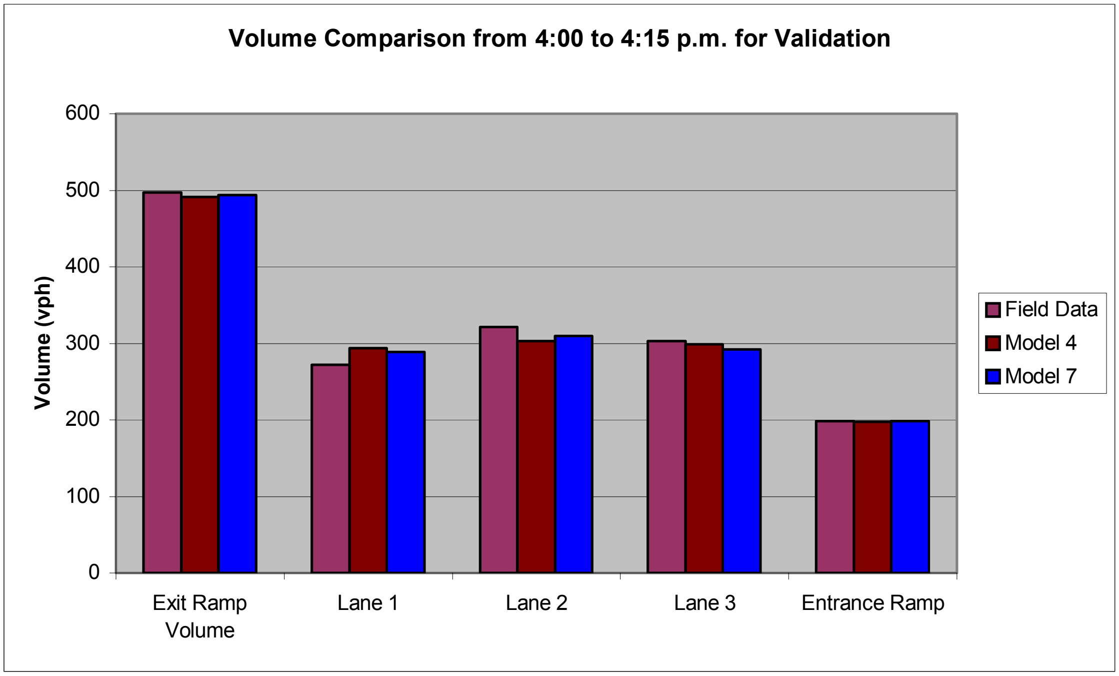

Figure 5.36 Volume Comparison from 4:00 to 4:15 p.m. for Validation

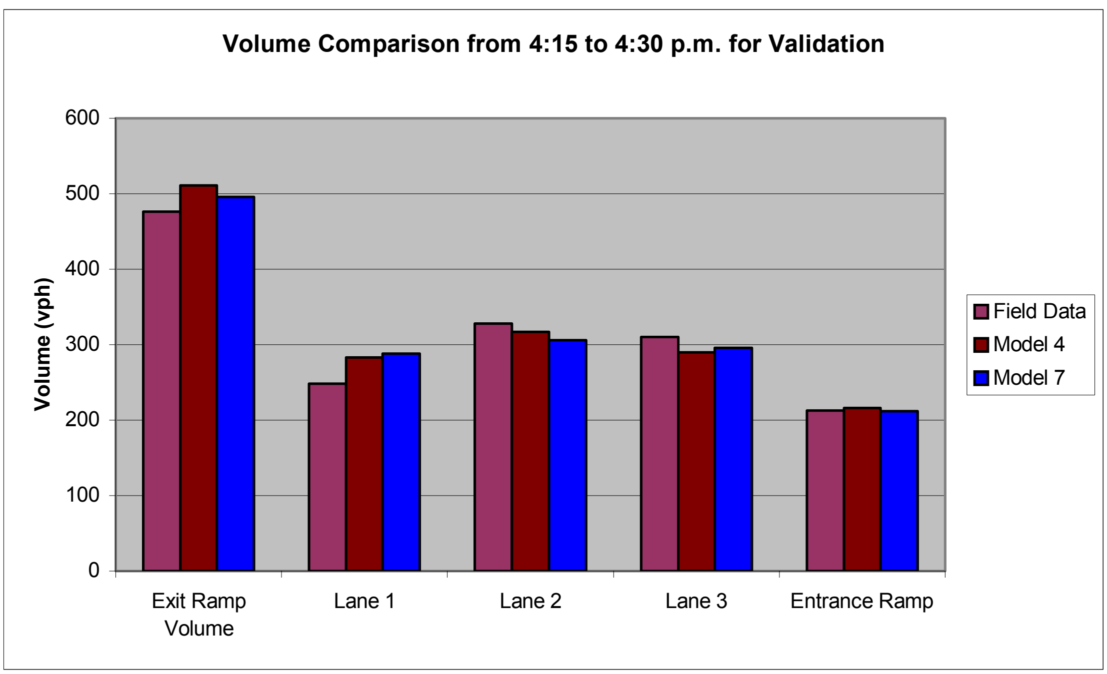

Figure 5.37 Volume Comparison from 4:15 to 4:30 p.m. for Validation

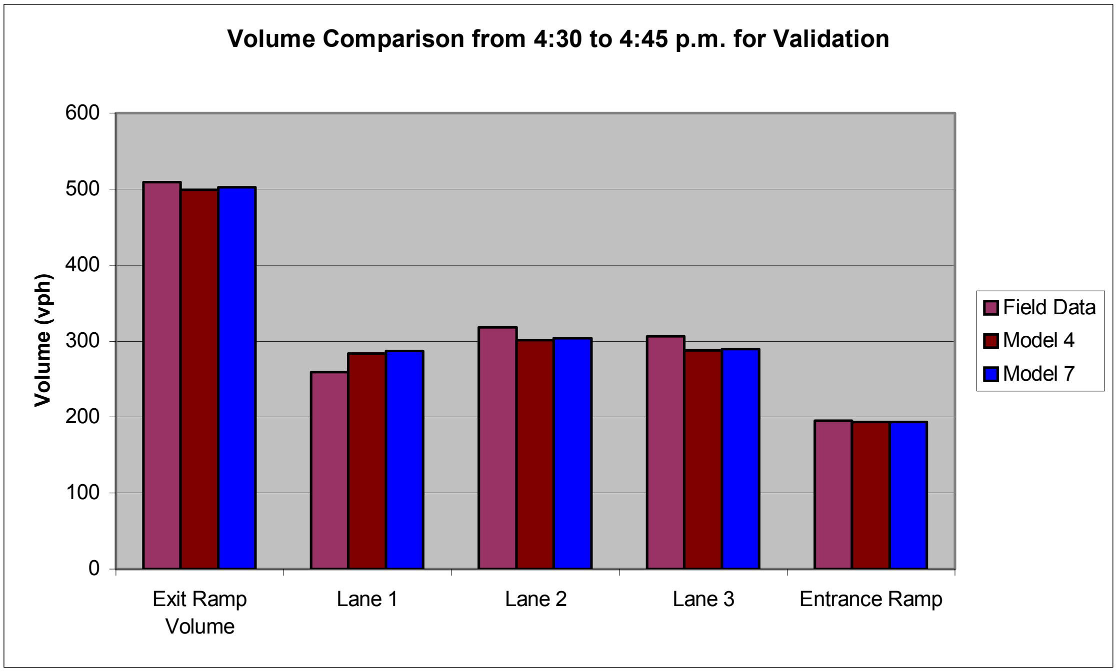

Figure 5.38 Volume Comparison from 4:30 to 4:45 p.m. for Validation

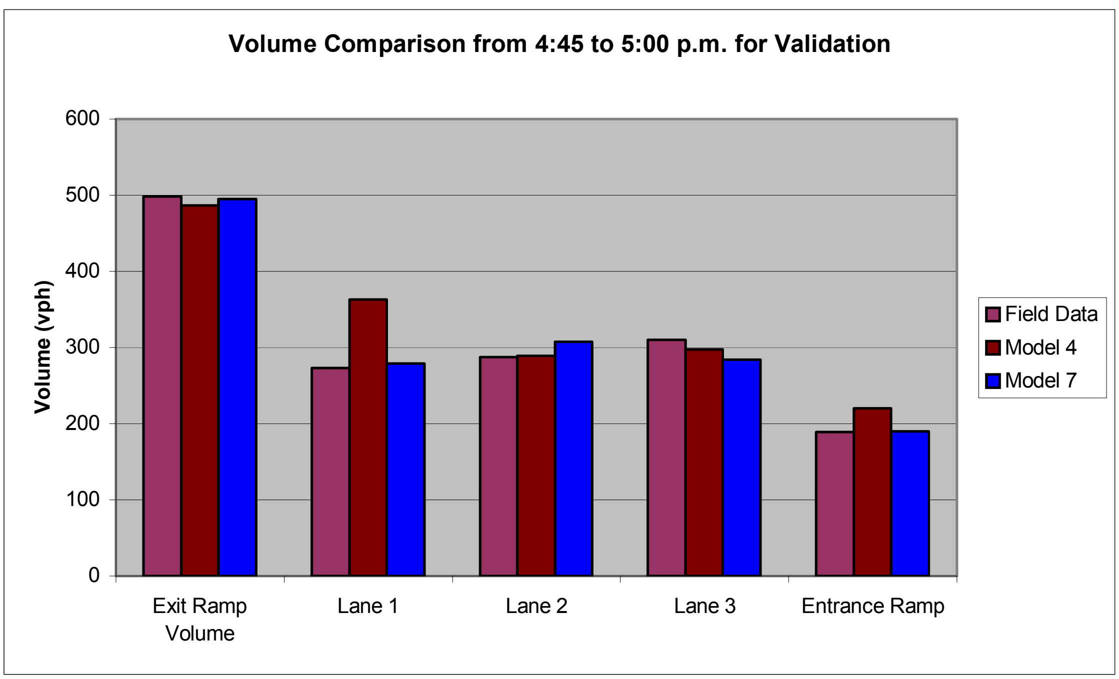

Figure 5.39 Volume Comparison from 4:45 to 5:00 p.m. for Validation

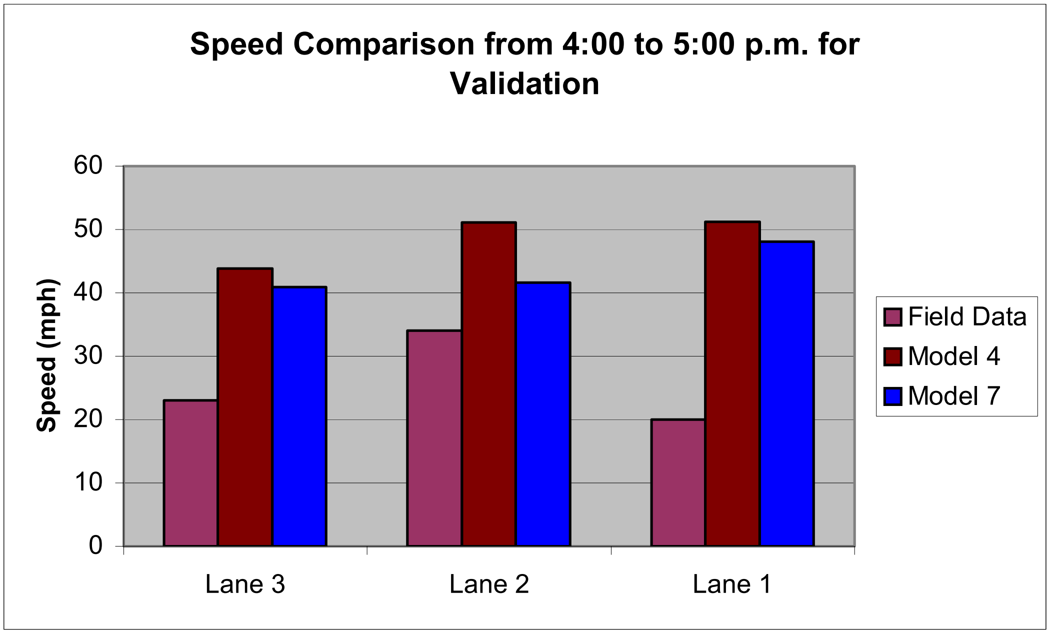

Figure 5.40 Speed Comparison from 4h to 5 p.m. for Validation

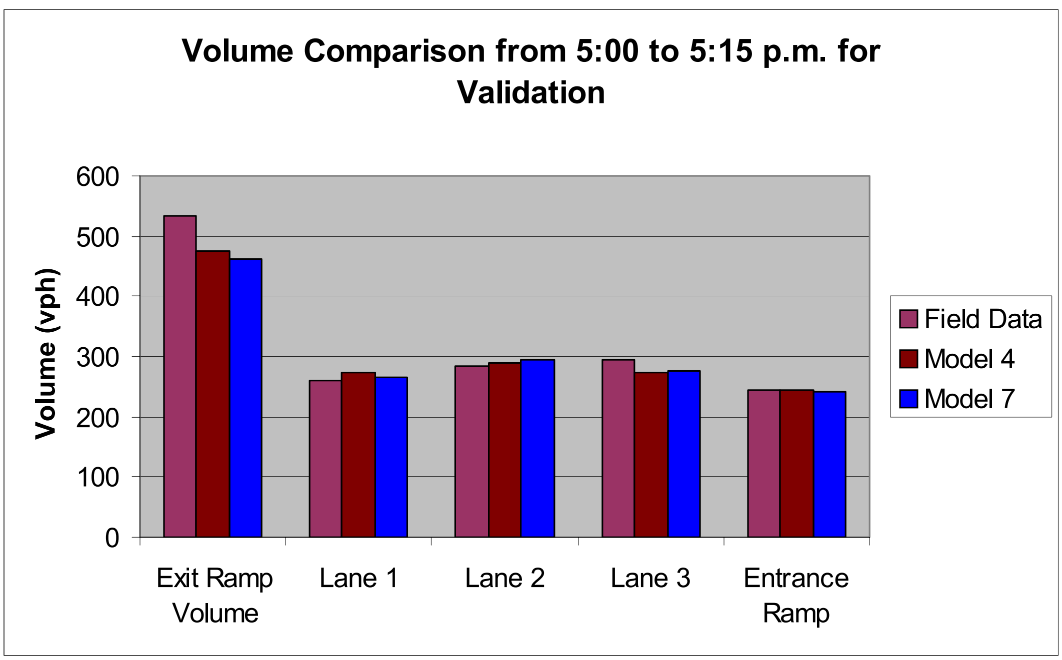

Figure 5.41 Volume Comparison from 5:00 to 5:15 p.m. for Validation

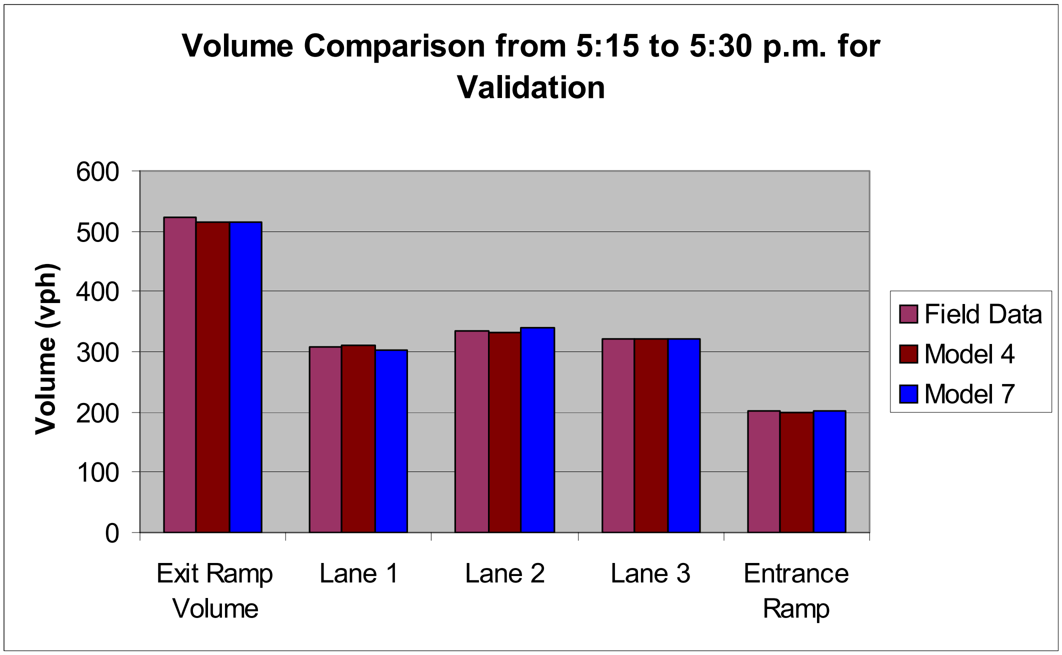

Figure 5.42 Volume Comparison from 5:15 to 5:30 p.m. for Validation

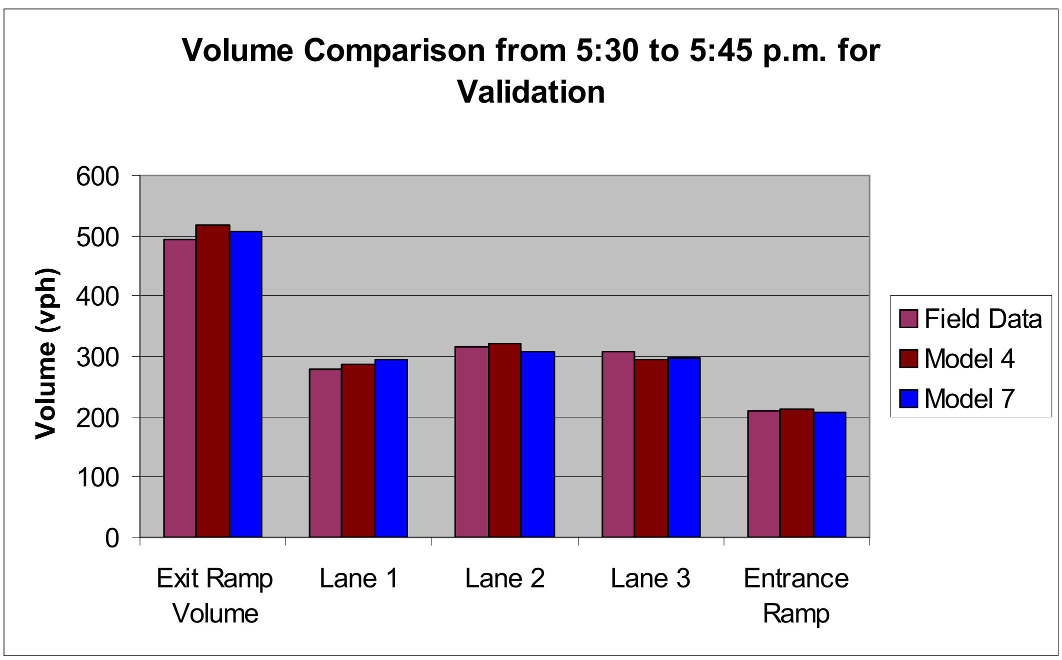

Figure 5.43 Volume Comparison from 5:30 to 5:45 p.m. for Validation

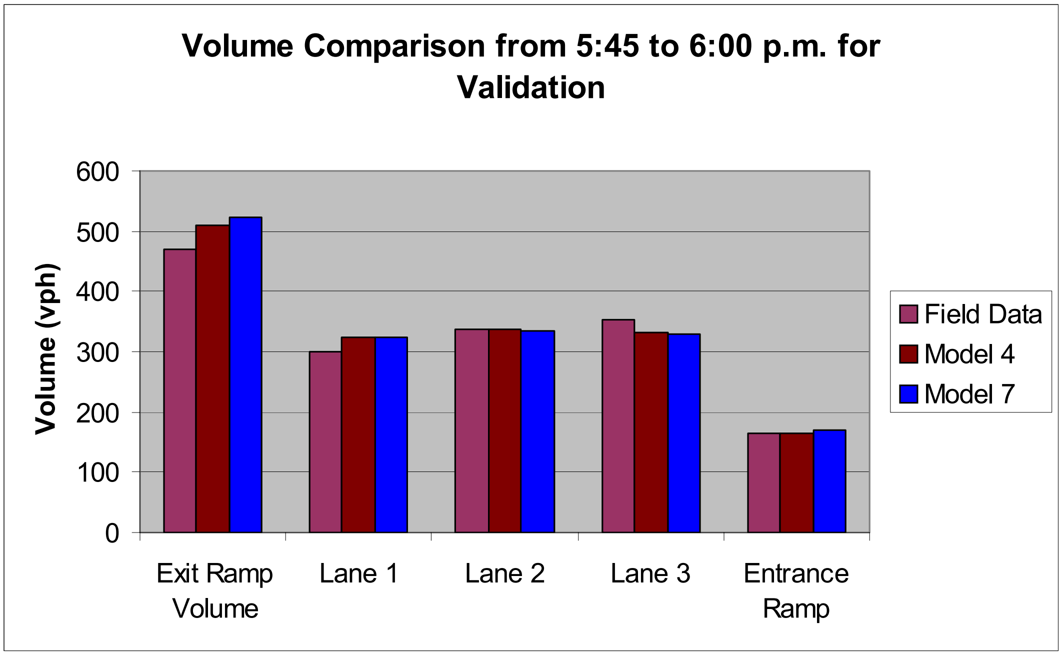

Figure 5.44 Volume Comparison from 5:45 to 6:00 p.m. for Validation

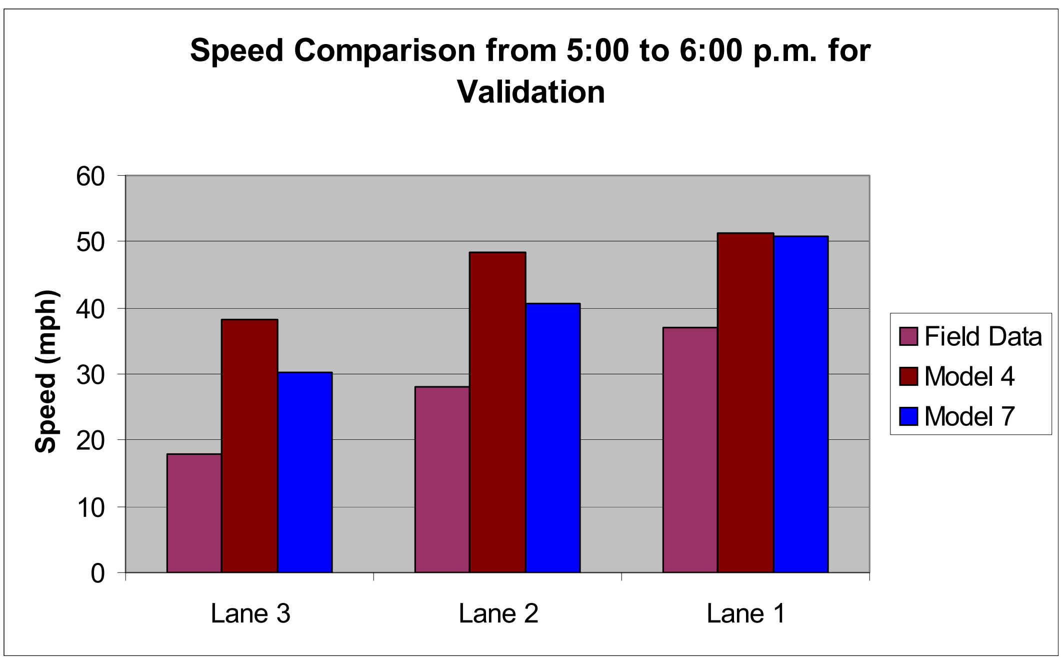

Figure 5.45 Speed Comparison from 5:00 to 6:00 p.m. for Validation

The calibration and validation processes were described in detail in this chapter. The calibration process was conducted using the data from 7:35 to 8:35 a.m., while data collected from 4:00 to 6:00 p.m. was used to validate the calibrated simulation model. Models 4 and 7 appeared to be the best and were used for the validation. Since the traffic from 7:35 to 8:35 a.m. during the a.m. peak was not as heavy as traffic during p.m. peak at the field data, the effectiveness of changing CC1 and minimum headway is not clearly seen during the calibration process. However, as seen in Figures 5.40 and 5.45, the speed reduced significantly between model 4 and model 7. Next, chapter 6 discusses simulation runs and their results.

CHAPTER 6

SIMULATION RUNS AND RESULTS

This chapter includes five sections. Section 6.1 presents stage 1 of the simulation runs, which investigates the correlation between capacity of a weaving section and entrance ramp flow. Section 6.2 discusses stage 2 of the simulation. In contrast to stage 1, stage 2 examines the relationship between capacity and ramp to ramp flow (R-R). Section 6.3 presents the regression model for estimating the capacity of the two-sided Type C weave. In this model, the dependent variable is the weaving capacity and potential independent variables include mainlane flow, exit ramp flow, entrance ramp flow, and R-R flow. The last section compares the densities obtained from the simulation with the values recommended in the 2000 Highway Capacity Manual (HCM).

In order to determine the capacity of the weaving area, repeated simulations are performed under varying traffic flow rates. The range for traffic flow mentioned above is shown in table 6.1 and was suggested by the field data. In the simulation runs, capacity was determined to be the point where the demand started to exceed the throughput flow as the flows were increased. Here, the demand refers to the flows specified in the simulation run while the throughput flow reflects actual number of vehicles that were processed.

Table 6.1 Range of Input Traffic Flow Rate for Simulation

Mainlane (vph)	Exit Ramp (vph)	Entrance Ramp (vph)	Ramp to Ramp (vph)
2000	800	100→2000	100→2000
3500	1000		
4000	1200		
4500	1500		
4800	1750		
5000	2000		
5500			
6000			
6500			

After a warm-up time of five minutes, required to fill the network with vehicles, the simulation was executed for a period of 3600 seconds (1 hour). The average output value from three runs corresponding to different random seed numbers was used for analysis. At this stage, VISSIM outputs include link evaluation (*.STR), data collection (*.MES), and travel time (*.RSZ) files, which allow extracting all parameters, such as R-R flow, exit ramp flow, entrance ramp flow, mainlane flow, density, and etc. The mainlane volume is counted at the point immediately upstream of the entrance ramp gore (Figure 4.2). The two stages of simulation are described in the next two sections:

<u>6.1 Stage 1</u>

The objective of this stage is to investigate the relationship between capacity and entrance ramp flow with different values of mainlane volume and exit ramp volume. The value of the entrance ramp volume will change from 50 to 2000 vehicles per hour (vph) while the volumes of other movements are held constant. At this stage, 10% and

25% of the entrance ramp volume will be directed to the exit ramp (representing the R-R demand). Each movement's volume, which will cause the network to reach capacity, is also clarified at this stage. For example, when the mainlane volume is at 3500 vph or below, the network will not reach capacity. A few hundred runs were conducted at this stage; however, some meaningful results are shown in the Appendix A and following three scenarios are introduced in this section.

Scenario 1: In this scenario, the total mainlane volume is 3500 vph, exit ramp volume is 800 vph, 10% of entrance ramp volume is the R-R demand, and the entrance ramp volume varies from 50 to 2000 vph. Figure 6.1 show the relationship between the entrance ramp demand and entrance ramp flow, which illustrates the capacity of the entrance ramp. The entrance ramp demand is the total number of vehicles that is generated at the ramp while entrance ramp flow is the throughput flow. Also, the volume changes in each lane of the mainlane freeway are shown in Figure 6.2. Figure 6.3 shows the relationship between total volume (mainlane + entrance ramp) and entrance ramp demand.

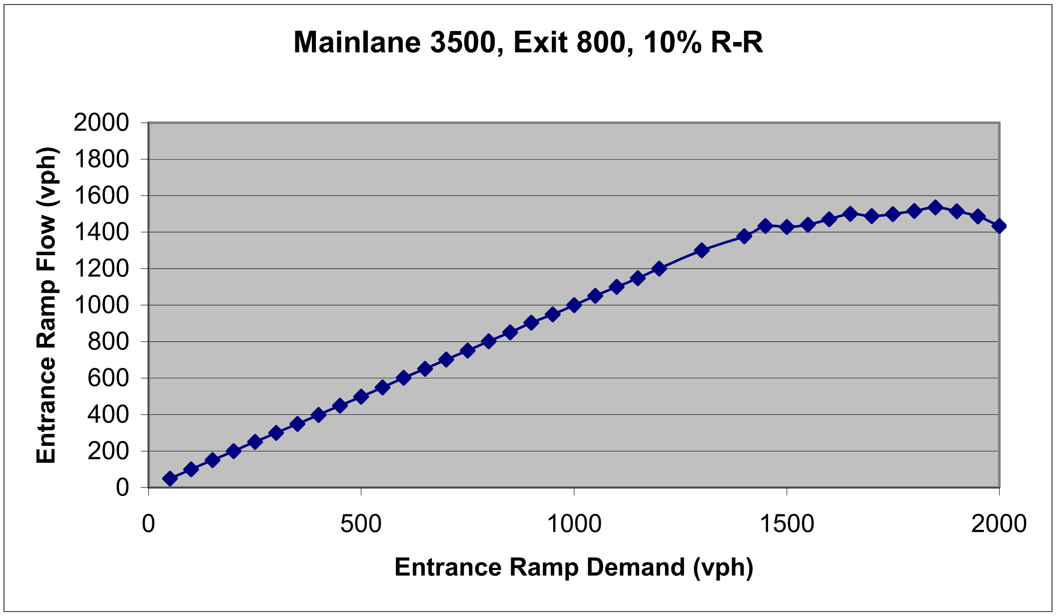

Figure 6.1 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 1

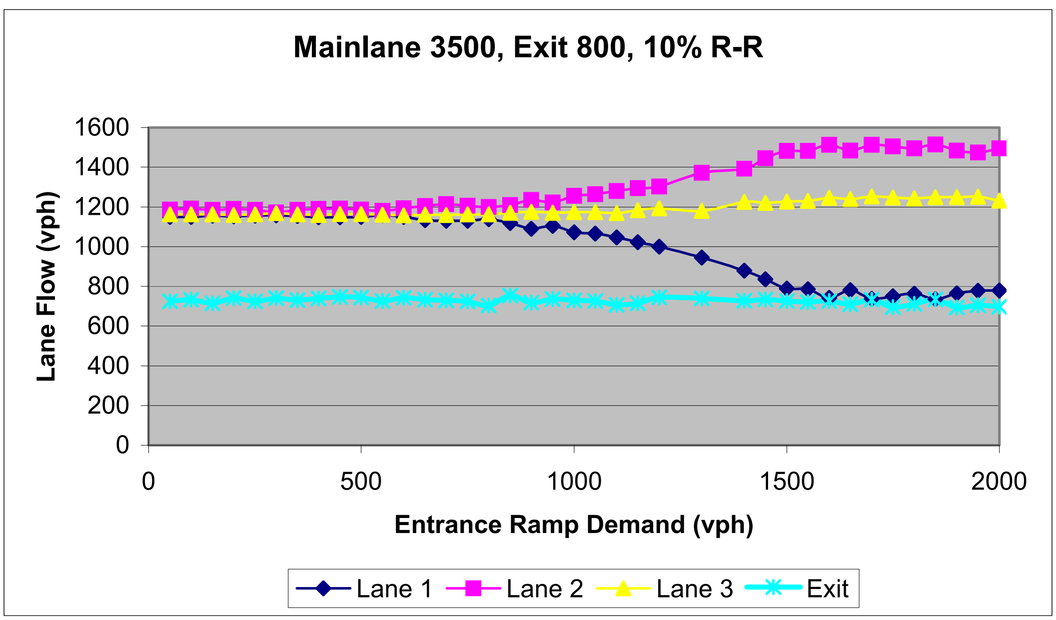

Figure 6.2 Entrance Ramp Demand vs Lane Flow for Scenario 1

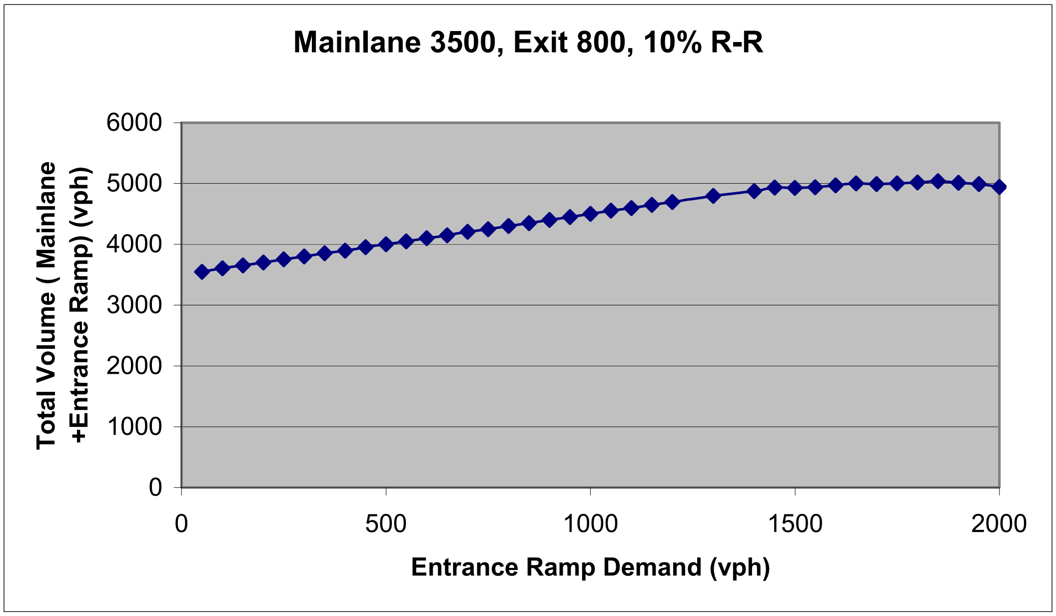

Figure 6.3 Entrance Ramp Demand vs Total Volume for Scenario 1

When mainlane volume is 3500 vph, the value of the entrance ramp flow remains unchanged after its demand reaches 1500 vph (Figure 6.1). This means that the capacity of entrance ramp is about 1500 vph. It is interesting to see how the capacity of entrance ramp changes when the mainlane volume increases (scenario 2). In Figure 6.2, increasing the entrance demand causes the lane 1 volume to decrease significantly because of traffic entering into that lane. Lane 2 volume is higher because vehicles begin to maneuver to the left to avoid the entering traffic in the first lane. Lane 3 and the exit ramp flows remain unchanged because the demands are too low. Figure 6.3 shows the relationship between total volume and entrance ramp demand; however, mainlane and exit ramp throughput volumes are always equal to the demand, which shows that this network is not at capacity yet.

Scenario 2: Mainlane volume and R-R volume increases in this scenario: the total mainlane volume is 5000 vph, exit ramp volume is 800 vph, 25% of entrance ramp volume is the R-R demand, and entrance ramp demand varies from 50 to 2000 vph.

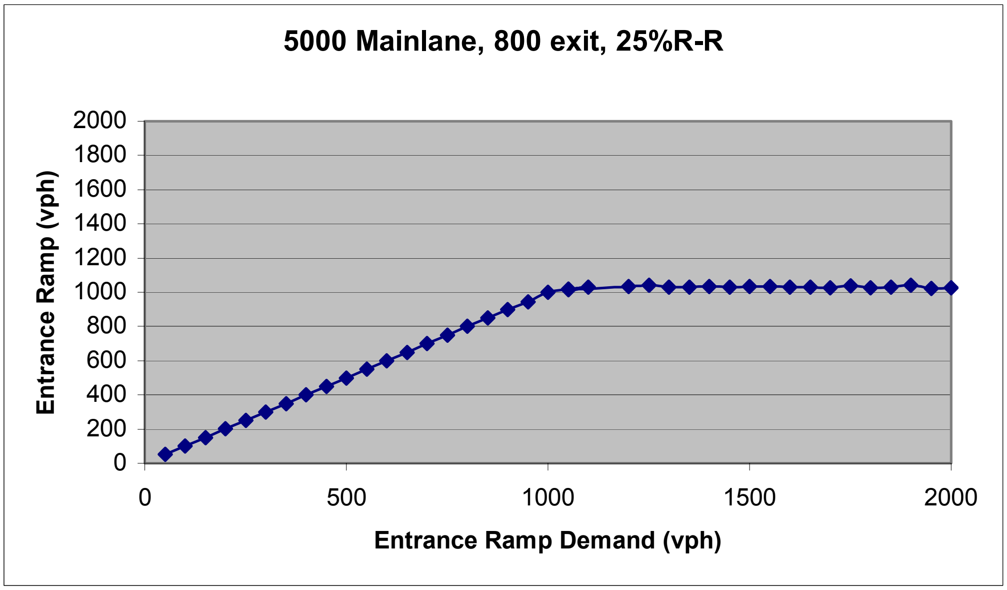

Figure 6.4 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 2

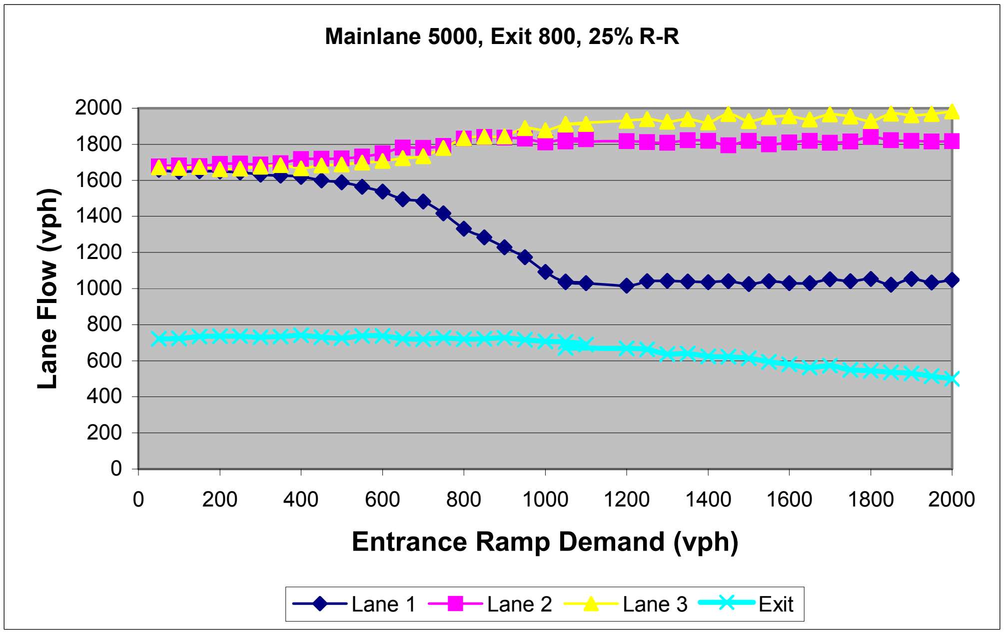

Figure 6.5 Entrance Ramp Demand vs Lane Flow for Scenario 2

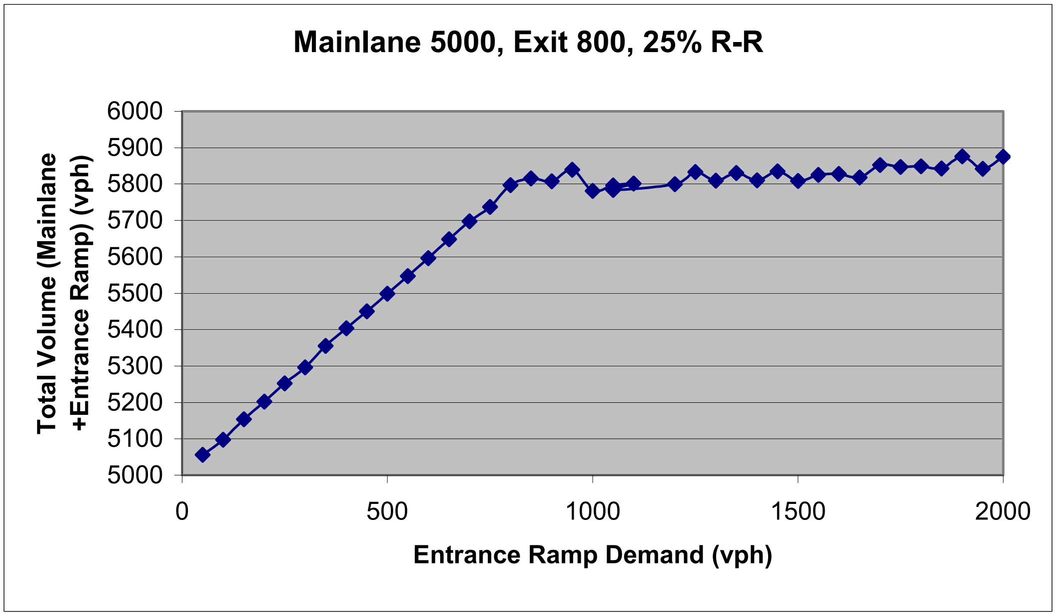

Figure 6.6 Entrance Ramp Demand vs Capacity for Scenario 2

The entrance ramp capacity decreases from 1500 vph (Figure 6.1) to 1000 vph (Figure 6.4) as mainlane volume increases from 3000 to 5000 vph. The volumes on lane 2 and lane 3 begin increasing from 1700 vph and 1800 vph. However, at 1800 vph, the lane 3 volume continues to increase, while lane 2 decreases, implying that lane 2 has reached its capacity (Figure 6.5). The fact that throughput flows are slightly smaller than the demand leads to the conclusion that the mainlane reaches capacity at a point somewhere just below 5000 vph with an exit ramp demand of 800 vph (the minimum value of exit ramp demand in the proposed range).

Scenario 3: The total mainlane volume is 6000 vph, exit ramp volume is 1200 vph, 25% of entrance ramp volume is the R-R demand, and entrance ramp volume varies from 50 to 2000 vph.

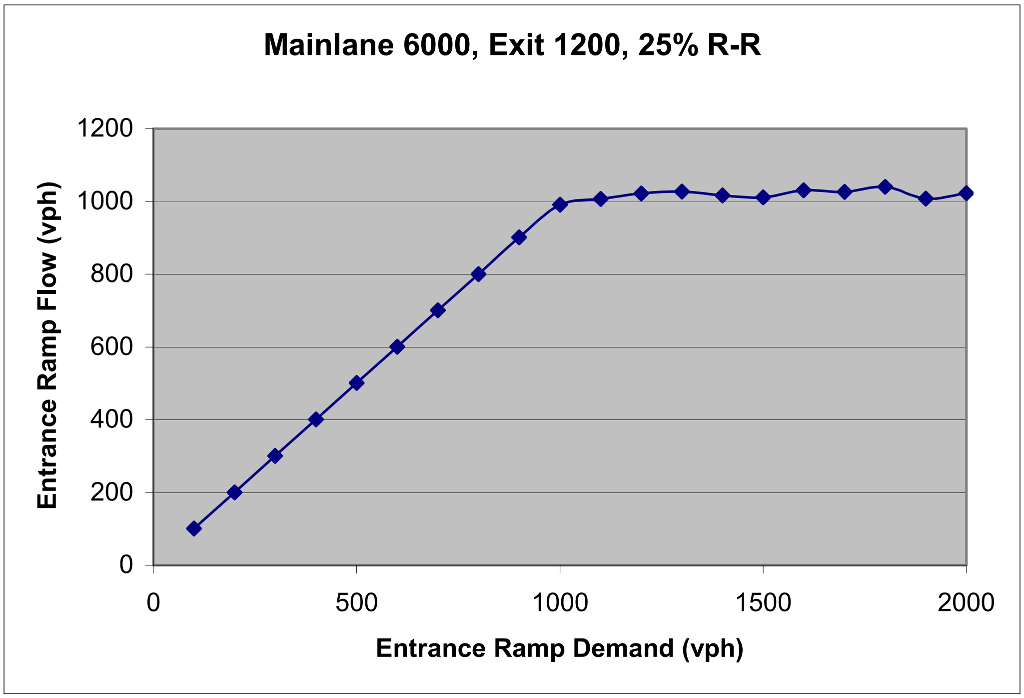

Figure 6.7 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 3

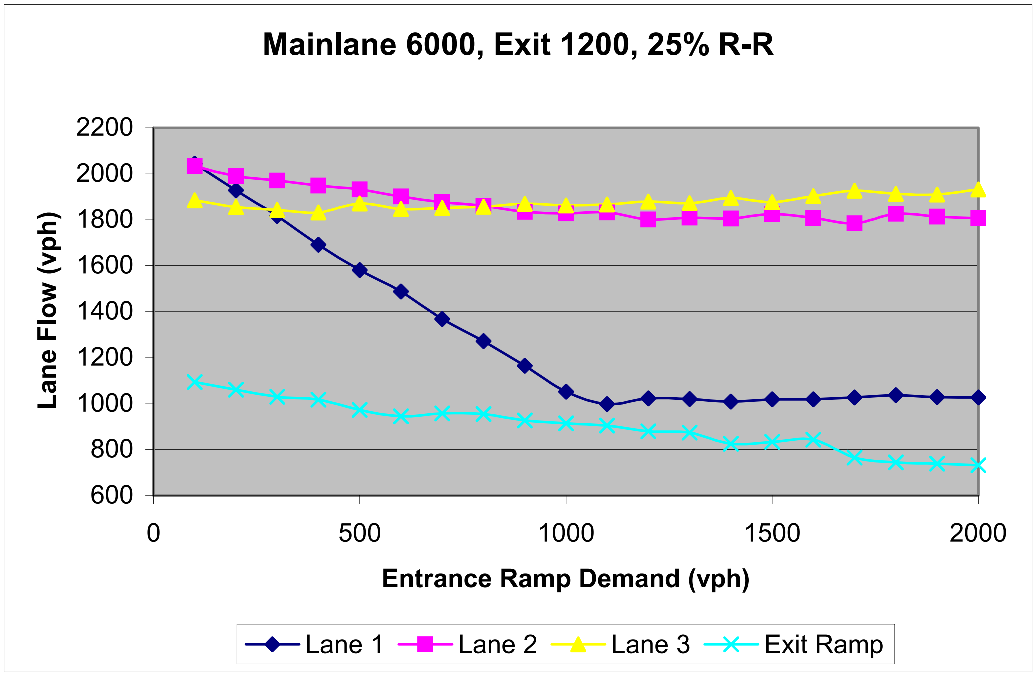

Figure 6.8 Entrance Ramp Demand vs Lane Flow for Scenario 3

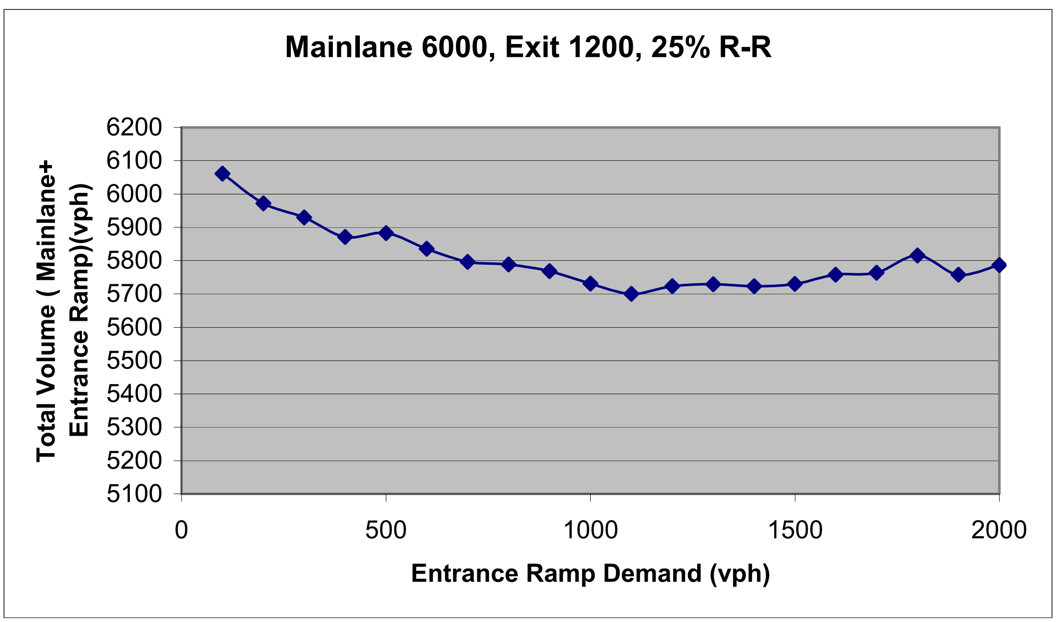

Figure 6.9 Entrance Ramp Demand vs Capacity for Scenario 3

In this scenario, the system is at capacity because the throughput volumes are significantly smaller than the demand. As seen in Figure 6.9, the capacity value is roughly 5750 vph for this scenario. Furthermore, the capacity of the entrance ramp is still around 1000 vph (figure 6.7). Lane 2 capacity also increases from 1700 to 1800 vph as seen in scenario 2. Lane 3 capacity is also the same value as previous scenarios (figure 6.8). One should note that lane 1 capacity is equal to entrance ramp capacity (1000 vph) in all scenarios in stage 1 (including those in Appendix A) when mainlane volume is at capacity.

After this stage, the following conclusions are found:

- Both entrance ramp capacity and lane 1 capacity are 1000 vph if the mainlane demand exceeds 4500 vph.
- When mainlane demand is 5000 vph, the throughput volume is slightly smaller than the demand, which means that this network reaches capacity at the point somewhere just below 5000 vph on the mainlane. Therefore, next stage should be investigated when the mainlane volume is smaller than 5000.
- Regression analysis was conducted for stage 1. However, the result shows that capacity has no relationship with any potential explanatory variables (mainlane flow, exit ramp flow, entrance ramp flow, lane 1, lane 2, and lane 3). Even though the best R^2 obtained from stepwise regression is 0.66, the standard error of the estimate for this model is very high, about 1000 veh. In addition, the coefficients do not explain the relationship between capacity

and independent variables logically. Therefore, stage 2 will focus on R-R flow and how it affects the weaving capacity.

6.2 Stage 2

For a two-sided weave, the R-R flow is a major factor that affects the capacity of the weaving section because the R-R traffic must cross all of the freeway mainlanes. Therefore, in contrast to stage 1, the goal of this stage is to investigate the relationship between weaving capacity and R-R flows. There are two models for this stage: (1) Entrance ramp volume is at capacity 1000 vph, and (2) Entrance ramp volume is smaller than capacity (500 vph).

Model 1: Entrance ramp is maintained at capacity (1000 vph) while the volumes of R-R, mainlane, and exit ramp are changed. Different runs corresponding to different combinations of mainlane volume, exit ramp volume, and R-R volume are conducted in this model. The summaries of all scenarios for this model are shown in Table 6.2.

Table 6.2 Summary of All Scenarios in Model 1

Scenario	Entrance Ramp (vph)	Mainlanes (vph)	Exit Ramp (vph)	R-R (vph)
1	1000	3500	1000	200-1000
2			1500	
3			2000	
4	1000	4000	1000	200-1000
5			1500	
6			2000	
7	1000	4500	1000	200-1000
8			1500	
9			2000	
10	1000	4800	1500	200-1000
11			2000	
12	1000	5000	800	200-1000
13			1500	
14			2000	
15	1000	5500	1000	200-1000
16			1200	
17			1500	
18			2000	
19	1000	6000	800	200-1000
20			1000	
21			1200	
22			1500	
23			1750	
24			2000	
25	1000	6500	1000	200-1000
26			1500	

The selected simulation results are described in Table 6.3, where V is the total throughput volume, V_w is weaving volume, and V_r is the weaving ratio.

Where:

V= Mainlane Flow + Entrance Ramp Flow

V_w= R-R Flow + Freeway to Freeway (F-F) Flow

$f_w = V_w/V$

The system is not reaching capacity until mainlane and exit ramp volumes reach 4500 and 1500 vph, respectively (scenario 8, table 6.2), where the throughput volumes are significantly smaller than the demands. Figures 6.10 to 6.15 show the relationship between R-R demand to lane volumes, exit ramp, and capacity when the system is under capacity and is at capacity. The completed results for all scenarios are in Appendix B.

Table 6.3 presents the selected simulation results from scenarios 2, 8, and 25. Scenario 2 represents an under-capacity state, while scenario 8 is a transition state from under capacity to capacity. Scenario 25 represents a state where demand exceeds capacity.

Table 6.3 Selected Simulation Results for Model 1

Scenario	Input Volume (Demand) (vph)				Flow (vph)						
	Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	V_w	f_w
2	3500	1500	1000	200	3466	1356	986	201	4452	2512	0.564
2	3500	1500	1000	1000	3459	1407	986	914	4445	3880	0.873
8	4500	1500	1000	200	4440	1366	984	197	5424	3468	0.639
8	4500	1500	1000	1000	4356	1324	978	888	5334	4808	0.901
25	6500	1500	1000	200	4649	1036	984	197	5634	4007	0.711
25	6500	1500	1000	1000	4317	1208	985	920	5302	4950	0.934

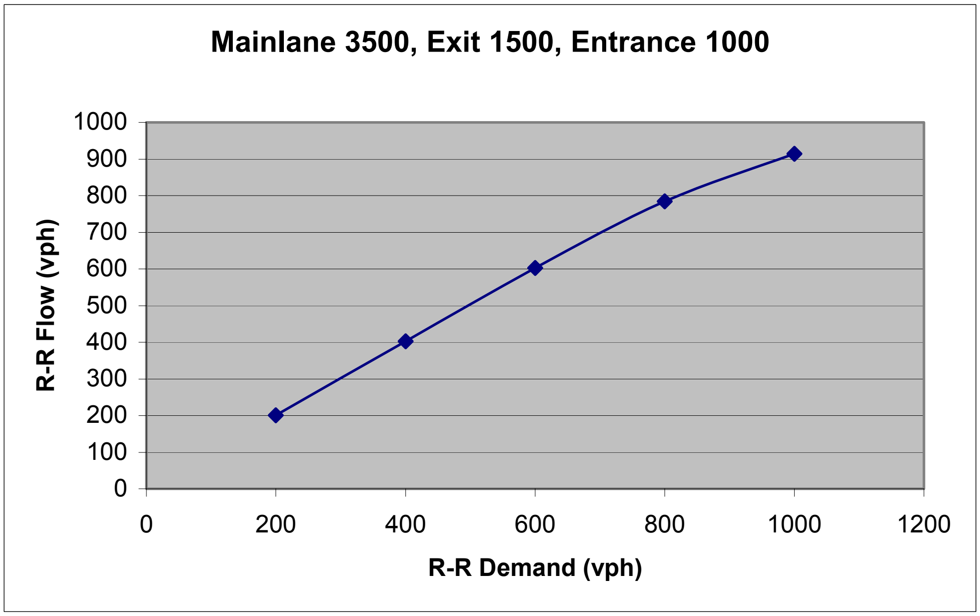

Figure 6.10 R-R Demand vs R-R Flow for Scenario 2

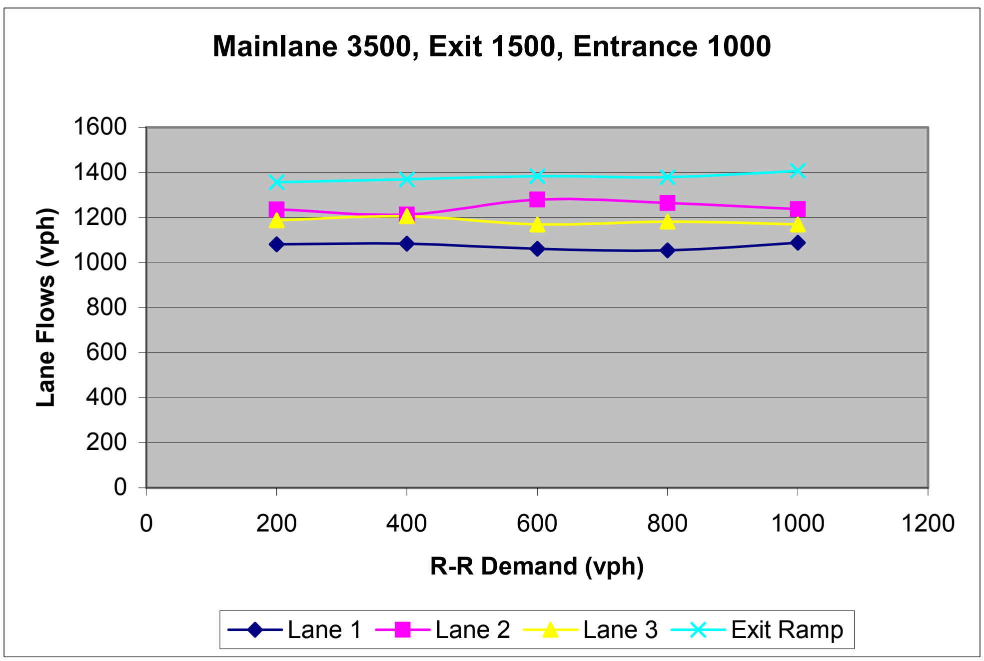

Figure 6.11 R-R Demand vs Lane Flow for Scenario 2

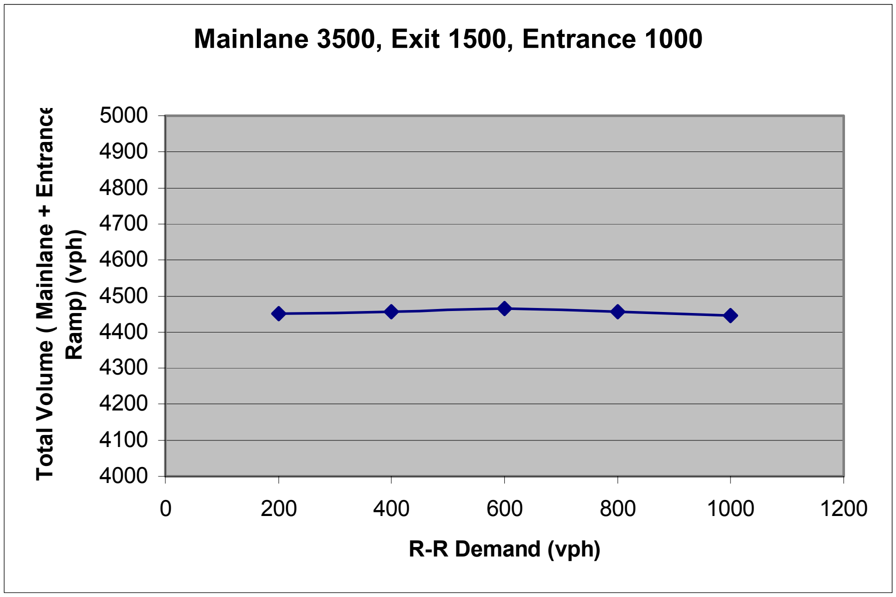

Figure 6.12 R-R Demand vs Total Volume (V) For Scenario 2

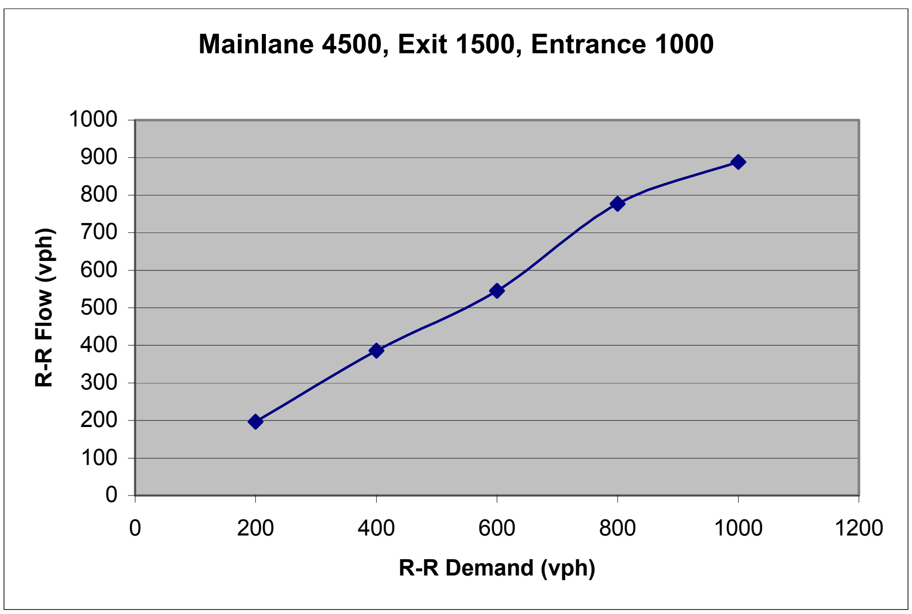

Figure 6.13 R-R Demand vs R-R Flow For Scenario 8

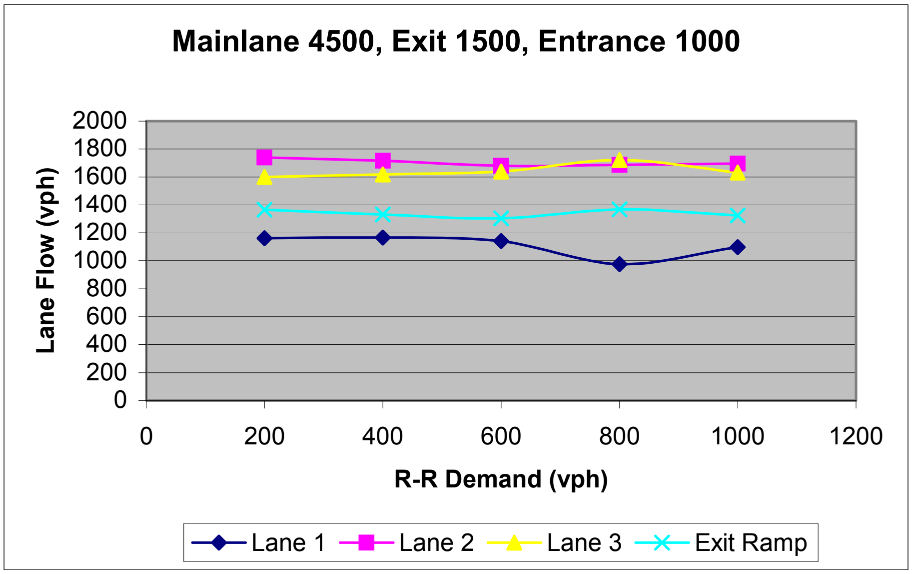

Figure 6.14 R-R Demand vs Lane Flow For Scenario 8

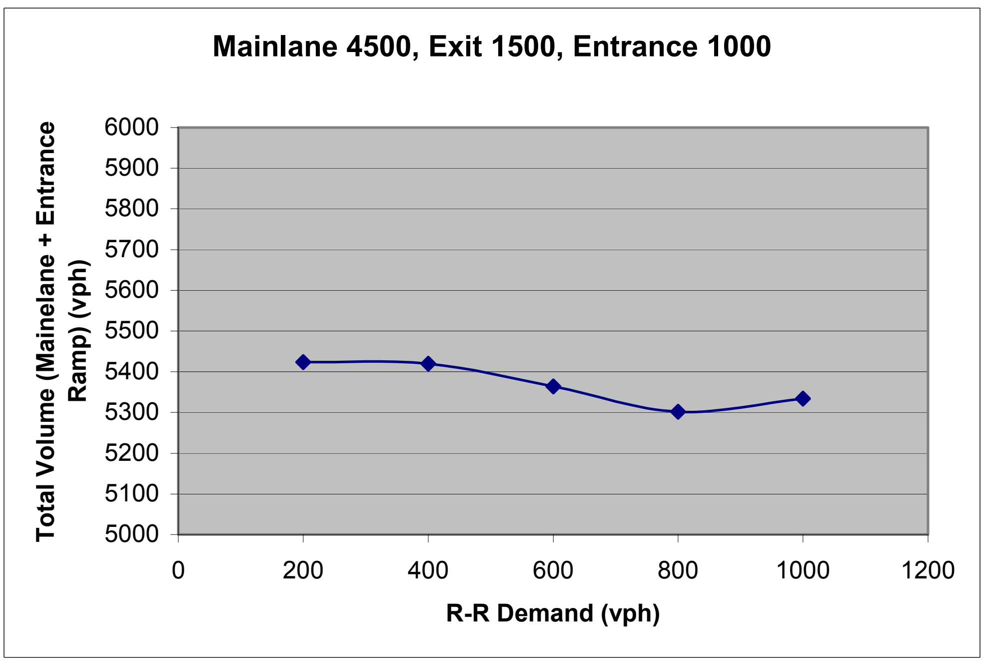

Figure 6.15 R-R Demand vs Total Volume (V) For Scenario 8

In scenario 2, where the system is not at capacity, lane 1, lane 3, and the total volumes remain the same despite the increase in R-R traffic. However, lane 2 flow increases because traffic begins to shift to lane 2 to avoid the entering vehicles in lane 1 (Figure 6.11).

In contrast, in scenario 8, when R-R demand is 600 vph, volumes of lane 1 and lane 2 begin to decrease while lane 3 flow starts to increase, which seems to indicate lane changes to the left because lane 1 and lane 2 have already reached capacity.

When R-R demand reaches 800 vph, lane 3 also decreases because the whole system is at capacity (figure 6.14). Also, as R-R flow increases, the F-F flow also increases because freeway to ramp (F-R) traffic decreases since exit ramp demand is a constant. As a result, weaving volume increases resulting in a reduction in capacity.

Regression analysis is performed later to examine the relationship between capacity and R-R flow. From this model, these following conclusions are found:

- At capacity, as R-R volume increases, total volume (V) decreases.
- As exit ramp demand increases, V also decreases because more vehicles from the mainlane and/or entrance ramp will weave to the left lane to exit, which creates more weaving activities in the system.
- Lane 1 capacity is 1000 vph if the system is at capacity (i.e. the mainlane demand exceeds 4500 vph).
- The sum of lane 1 and entrance ramp flow is roughly 2000 vph, which is approximately the capacity of one lane in freeway recommended by the 2000 HCM.

Model 2: Entrance ramp is 500 vph (smaller than capacity condition), while the volumes of R-R, mainlane, and exit ramp are changed. The objective of this model is to examine how the capacity of a weave changes when the entrance ramp flow is less than capacity and whether the percentage of R-R or total entrance ramp traffic will affect the capacity. The summaries of all scenarios for this model are shown in Table 6.4.

Table 6.4 Summary of Model 2 Scenarios

Scenario	Entrance Ramp (vph)	Mainlanes (vph)	Exit Ramp (vph)	R-R (vph)
1	500	5000	800	Variable
2	500	5000	1200	
3	500	5000	1600	
4	500	5000	2000	
5	500	5500	800	Variable
6	500	5500	1200	
7	500	5500	1600	
8	500	5500	2000	
9	500	6000	800	Variable
10	500	6000	1200	
11	500	6000	1600	
12	500	6000	2000	
13	500	6500	800	Variable
14	500	6500	1200	
15	500	6500	1600	
16	500	6500	2000	

Selected simulation results are shown in Table 6.5, where V is the total throughput volume, V_w is the weaving volume, and f_w is a weaving ratio. The complete simulation results for this model are in Appendix C. All scenarios before scenario 4

Table 6.5 Selected Simulation Results for Model 2

Scenario	Input Volume (Demand) (vph)				Flow (vph)						
	Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	V_w	f_w
1	5500	800	500	100	4951	729	494	101	5445	4424	0.812
1	5000	800	500	300	4950	720	496	302	5446	4835	0.888
1	5000	800	500	500	4935	724	492	492	5427	5195	0.957
4	5500	2000	500	100	4836	1787	491	98	5327	3245	0.609
4	5000	2000	500	300	4840	1818	497	307	5337	3635	0.681
4	5000	2000	500	500	4781	1702	494	494	5276	4068	0.771
9	6000	800	500	100	5407	670	495	102	5902	4941	0.837
9	6000	800	500	300	5343	678	491	295	5834	5256	0.901
9	6000	800	500	500	5262	686	491	491	5753	5558	0.966

are not at capacity because the throughputs are nearly the same as the demands. However, for scenario 4 and beyond, the throughputs are significantly smaller than demand; therefore, those scenarios are at capacity. Figures 6.16 to 6.21 show the relationship between R-R demand to lane volumes, exit ramp, and capacity when the system is at a transition state from under capacity to capacity.

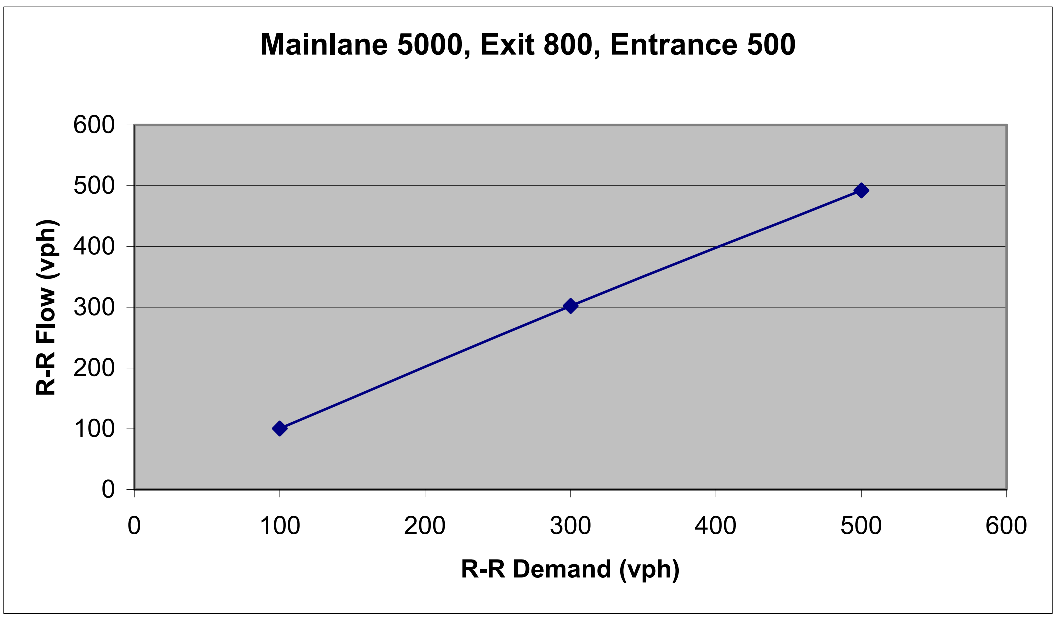

Figure 6.16 R-R Demand vs R-R Flow for Scenario 1

When the exit ramp demand is low, the throughputs of lane 2 and lane 3 are nearly the same (Figure 6.17). On the other the hand, under high exit ramp demand, lane 2 flow is higher than lane 3 because traffic appears to stay in lane 2 to avoid the weaving activities on lanes 1 and 3 (Figure 6.20).

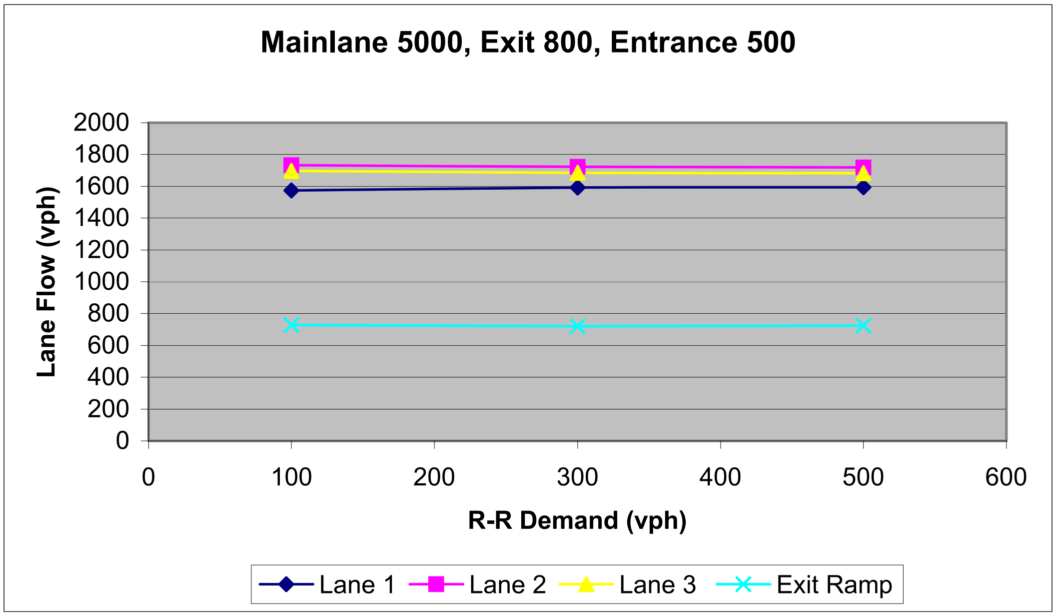

Figure 6.17 R-R Demand vs Lane Flow for Scenario 1

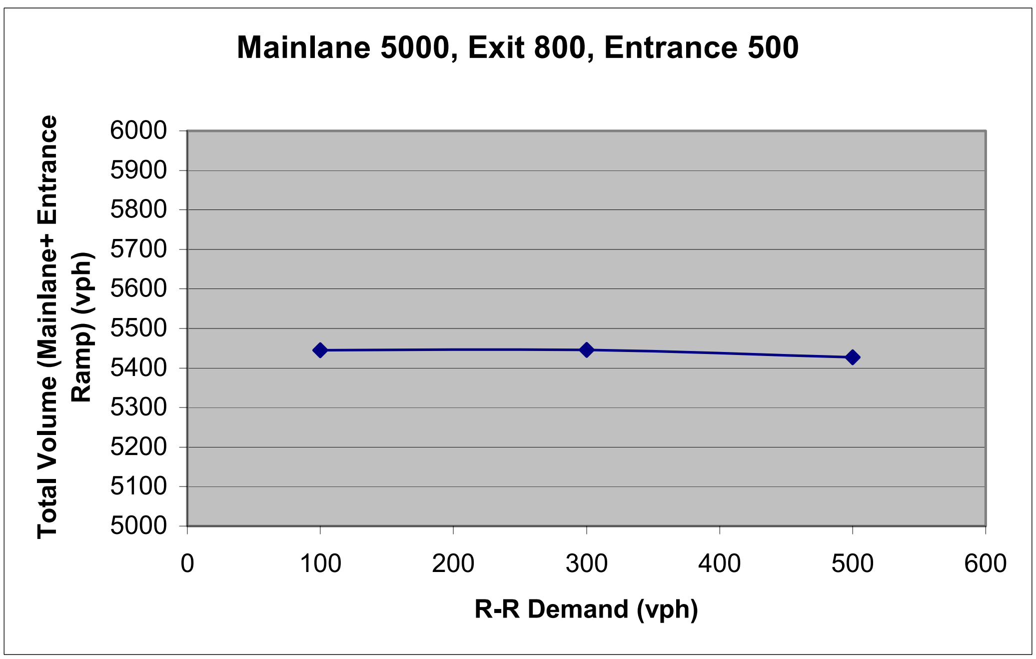

Figure 6.18 R-R Demand vs Total Volume (V) for Scenario 1

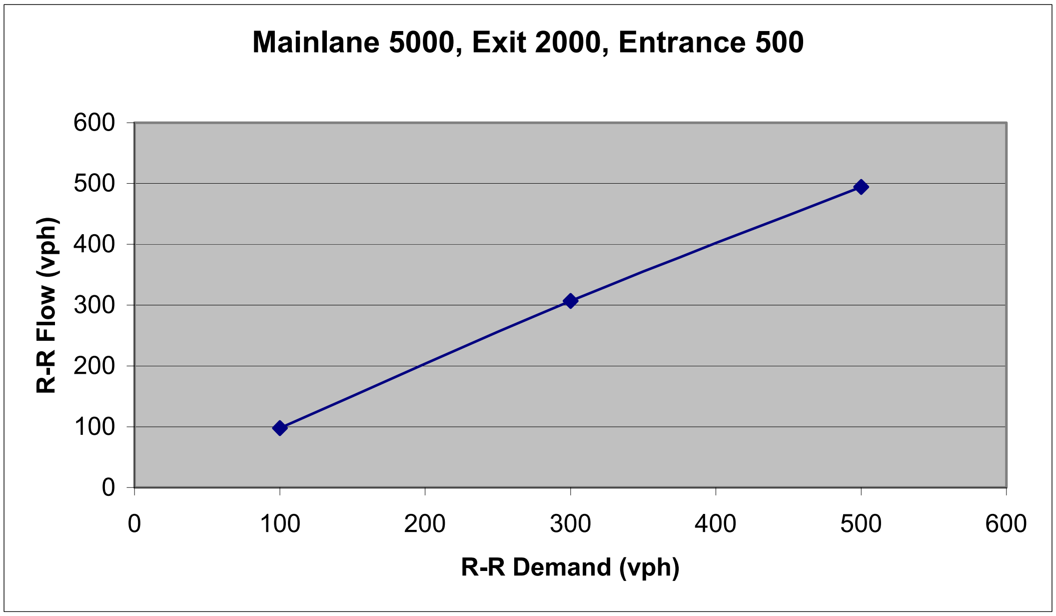

Figure 6.19 R-R Demand vs R-R Flow for Scenario 4

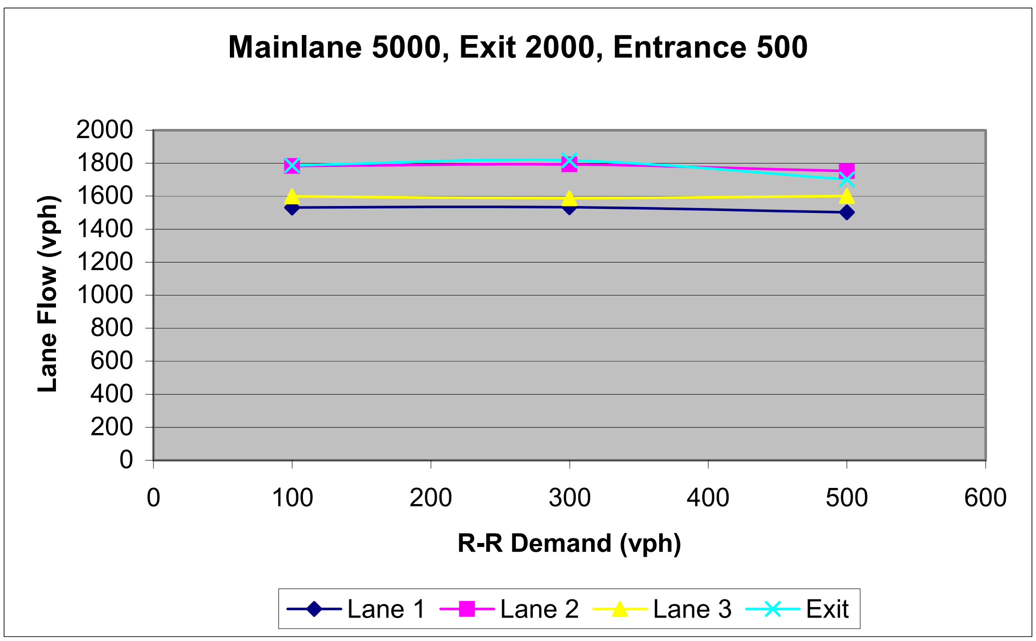

Figure 6.20 R-R Demand vs Lane Flow for Scenario 4

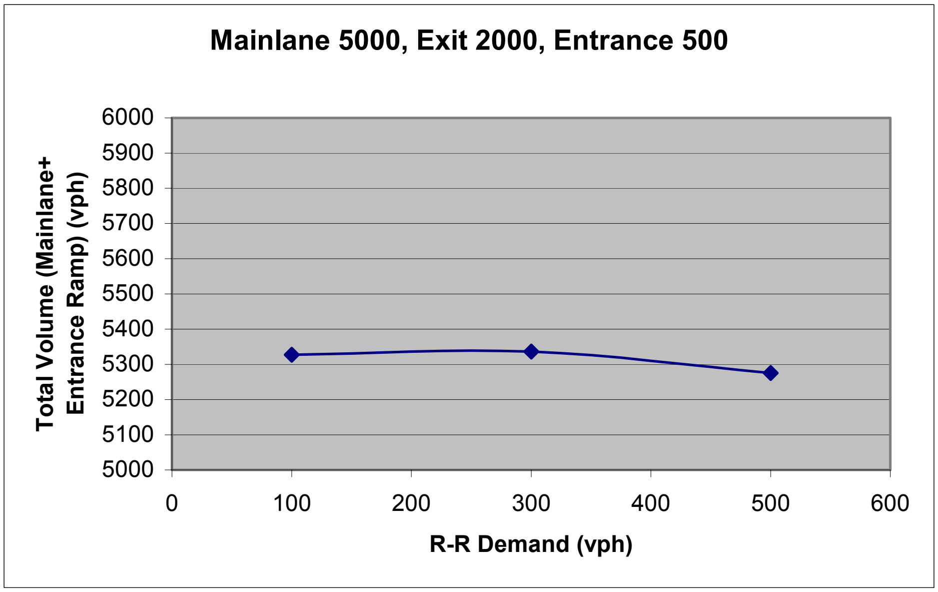

Figure 6.21 R-R Demand vs Total Volume (V) For Scenario 4

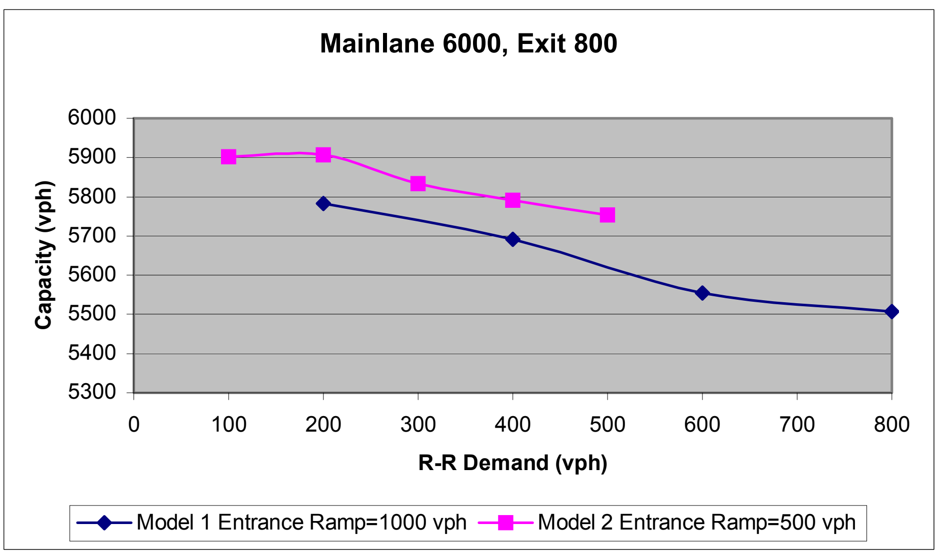

Figure 6.22 Capacity Comparison Between Model 1 and Model 2

The entrance ramp volume of model 2 is just half of model 1's volume; therefore, model 2 allows more throughput vehicles than model 1. Figure 6.22 is an example of the difference in capacity between these two models when mainlane volume is 6000 vph and exit ramp demand is 800 vph. As R-R demands are at 200 and 400 vph, the corresponding capacities of model 2 are 5900 and 5800 vph, respectively, which are about 100 vph higher than model 1. The combined flow for lane 1 and entrance ramp for all scenarios for both models is about 2100 vph.

From this model, the following conclusions can be reached:

- As in model 1, as R-R increases, total volume (V) decreases.
- As exit ramp demand increases, V also decreases because more vehicles from the mainlane and/or entrance ramp will weave to the left lane to exit, which creates more weaving activities in the system. Also, when exit ramp demand increases, lane 3 flow is always smaller than lane 2 flow.
- Lane 1 capacity is 1600 vph if the system is at capacity (i.e. the mainlane demand exceeds 5000 vph); however, sum of entrance ramp and lane 1 flows is always around 2000 vph.
- In stage 2, capacity occurs when the mainlane volumes are larger than 4500 vph for all scenarios. Therefore, examining the case when the entrance ramp volume is greater than 1000 vph is not necessary because stage 1 results indicates that the entrance ramp capacity is 1000 vph when the mainlane flow exceeds 4500 vph.

6.3 Regression Analysis

Regression analysis is conducted using the simulation outputs from both simulation models introduced above and a stepwise regression method is used to analyze the relationship between capacity and the list of potential explanatory variables (mainlane flow, exit ramp flow, entrance ramp flow, R-R flow, weaving volume, and weaving ratio). This method starts with one independent variable at a time and then it repeatedly searches for variables, which should be included in the model. The variable added is the one that results in the greatest reduction in error. It terminates the search when a specified maximum number of steps has been reached or when stepping is no longer possible given the stepping criteria. The stepwise regression is accomplished using SPSS software and the entire outputs are attached in Appendix C. The best regression model for predicting weaving capacity is given below where the sample size is 142 data points.

$$C_w = 5113 + 0.187 * V_{ML} - 0.317 * V_{EX} - 0.262 * V_{RR}$$

where

C_w= Weaving Capacity

V_{ML}= Mainlane Volume (vph) and $4500 \leq V_{ML} \leq 6500$

V_{EX}= Exit Ramp Volume (vph) and $800 \leq V_{ER} \leq 2000$

V_{RR}= R-R Flow and $100 \leq V_{RR} \leq 1000$

The ranges for each variable represent the data used to estimate the model.

Table 6.6 Regression Statistics

R	R^2	Adjusted R^2	Std. Error of the Estimate
0.990	0.980	0.980	25.590

Table 6.7 Model Coefficients

	Coefficients	Standard Error	t Stat	P-value
Intercept	5113.520	49.092	104.163	0.000
V_{ML}	0.187	0.009	21.085	0.000
V_{EX}	-0.317	0.007	-45.182	0.000
V_{R-R}	-0.262	0.011	-23.192	0.000

R^2 value for this model is very high (0.98), which means that 98% of the variation in weaving area capacity can be explained by this model. Moreover, the value of standard error of the estimate in Table 6.6 (positive square root of variance of the errors), which typically measures the difference between predicted capacity with the "true" capacity, is relatively small. The coefficients for both exit ramp and R-R are negative, which indicates that the weave capacity decreases when exit ramp and R-R traffic increase. In contrast, the coefficient for mainlane volume is positive; therefore, the capacity increases when mainlane flow increases.

Similar to the standard error of estimate in Table 6.6, the standard error in Table 6.7 presents the standard error of the coefficient estimates. Essentially, they measure how these coefficients vary from sample to sample. The model is more reliable as the standard error decreases. The reported t-statistic is the coefficient divided by its standard error. This is based on the following assumption: If the standard errors of the estimate (population errors) are normally distributed, then it can be shown that the sample estimates for coefficients of the model follow a t-distribution [Ref. 34]. The t-statistic represents the size of the standard error relative to the estimated coefficient; therefore, the model quality improves as the absolute value of t-statistic increases. Finally, the P value helps to answer the question "What is the likelihood of getting a sample value of 5113.520 (for intercept) when, in fact, the true value is 0" [Ref. 34]. Since the P value in this case is very close to 0 one can conclude that the probability of getting 5113.520 for the intercept in this model when its true value is 0 is nearly 0%; i.e., each of the coefficients is significantly different from zero. For those reasons, one can conclude that this model is adequate for predicting the capacity of the weave.

6.4 Model Limitation

The regression model was developed with simulated data with the following characteristic:

- Three freeway mainlanes
- 0.52 mile weaving section, and
- With 98% trucks and 2% passenger cars (traffic mix)

Each of these limitations represents future work. The first limitation, number of freeway mainlane, is briefly examined in the next chapter.

6.5 Density

Density is used to define levels of service in the weaving chapter of the 2000 Highway Capacity Manual (HCM). The objective of this section is to present the density associated with the simulation runs summarized above and to compare these values with the values in the HCM. The density used in this section is found directly from the VISSIM link evaluation (*.STR) output. Figures 6.23 and 6.24 present the relationship between density of each lane in the weaving area and the whole weaving section with capacity. The entire results are attached in Appendix D.

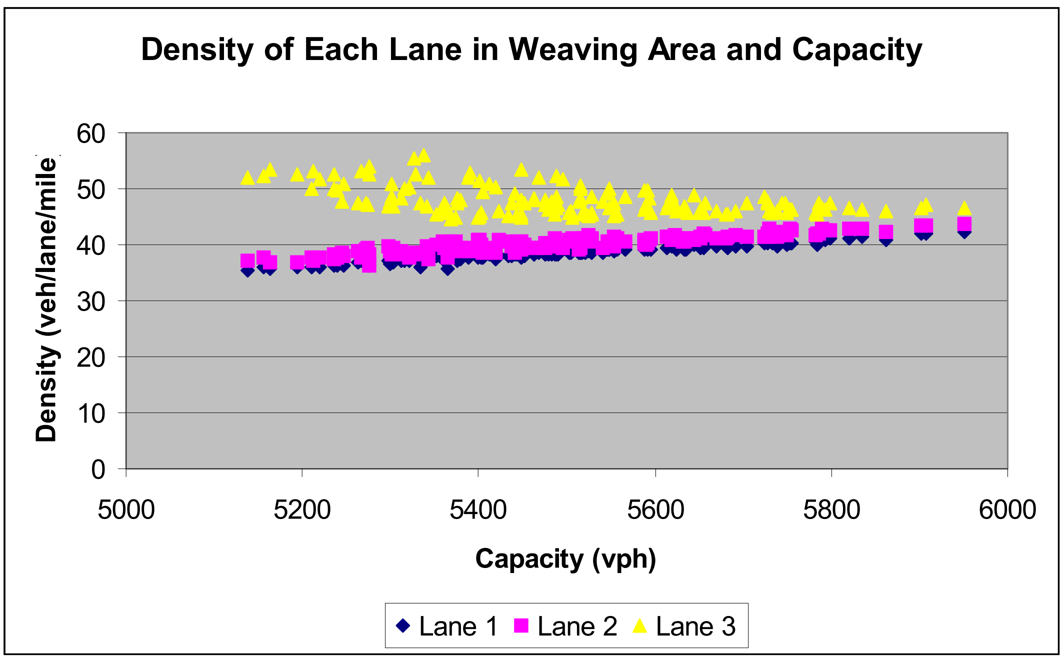

Figure 6.23 Lane Density and Capacity

In figure 6.23, the densities of lanes 1 and 2 are scattered from 36 to 44 vpmpl while that of lane 3 is higher due to the high exit ramp demand. However, densities average over the three lane (Figure 6.24) are around 43 vpmpl, which is the value used in the 2000 HCM for capacity. The 2000 HCM use passenger car rather than vehicle; however, since there is only 2% truck, the pcpmpl is quite close to vpmpl. When flows are less than capacity, the densities vary 30 to 41 veh/mile/lane (see Appendix D).

In this chapter, simulation runs for a wide range of flows are conducted and the simulation outputs are used to develop a regression model for estimating the capacity of a two-sided Type C weave with three mainlanes. The densities extracted from the simulation match the value used in the 2000 HCM. Chapter 7 extends the study to the case when the freeway has 4 mainlanes.

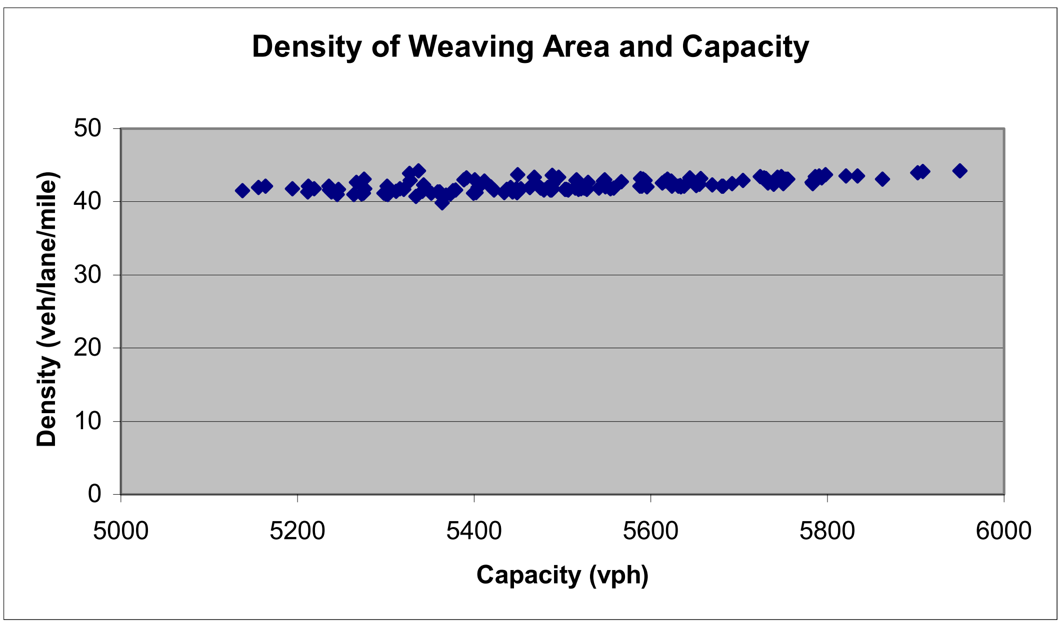

Figure 6.24 Weaving Density and Capacity

CHAPTER 7

MODEL EXTENSION

This chapter investigates the potential application of the regression model developed in chapter 6 to the case with four mainlane. The regression model for the scenario when the mainlane has three lanes is:

$$C_w = 5113 + 0.187 * V_{ML} - 0.317 * V_{EX} - 0.262 * V_{RR} \quad (6.1)$$

If the constant in the above equation is divided by 3 (number of lanes), the resulting model is:

$$C_w = 1705 * N + 0.187 * V_{ML} - 0.317 * V_{EX} - 0.262 * V_{RR} \quad (7.1)$$

where

C_w= Weaving Capacity (vph)

N= Number of lanes on the mainlane

V_{ML}= Mainlane Volume (vph)

V_{EX}= Exit Ramp Volume (vph)

V_{RR}= R-R Flow (vph)

Several simulation runs were conducted and the capacities from simulation outputs were compared with the values predicted from the equation 7.1. Table 7.1 showed traffic inputs from nine simulation scenarios. Figure 7.1 shows the comparison between values from the simulation outputs and values predicted by equation 7.1. The results indicated that the maximum error between model prediction and simulation output is less than two percent. Therefore, it appears to be feasible to use this model to predict the capacity for the case when mainlane has three or four lanes.

Table 7.1 Traffic Inputs for Simulation Scenarios

Scenario	Mainlane Volume (vph)	Exit Ramp Volume (vph)	Entrance Volume (vph)	R-R Flow (vph)
1	6400	1500	1000	600
2	6400	1500	1000	1000
3	6400	1500	1200	600
4	7200	1500	1000	600
5	7200	1500	1000	1000
6	8000	1500	1000	600
7	8000	1500	1000	1000
8	8800	1500	1000	600
9	8800	1500	1000	1000

Table 7.2 Capacity Comparisons

Scenario	Capacity from Simulation (vph)	Predicted Capacity from Model (vph)	Difference (%)
1	7298	7403	0.01
2	7171	7295	0.02
3	7341	7441	0.01
4	7367	7450	0.01
5	7184	7308	0.02
6	7416	7484	0.01
7	7168	7315	0.02
8	7425	7503	0.01
9	7219	7329	0.02

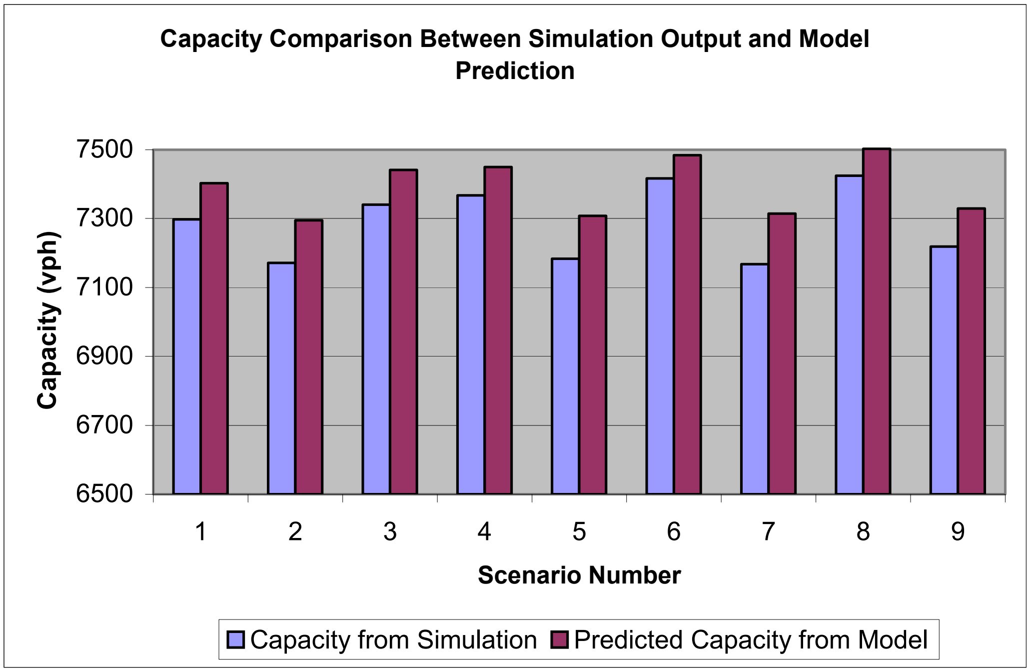

Figure 7.1 Capacity Comparisons for 4-lane Freeway

While the sample size of the runs in chapter 7 was small, the results appeared to be very promising. However, the simulation model was not separately calibrated for the 4-lane case. For more reliable results, it should be calibrated from the field data. Next, chapter 8 will conclude the work of this dissertation.

CHAPTER 8

CONCLUSIONS

A weaving section is a common design on major freeway facilities that has been of interest to researchers for years. Weaving areas are characterized by frequent lane changes which significantly reduce the capacity of the freeway systems. Therefore, they are generally considered to be critical points in urban freeway system. The 2000 HCM defined weaving capacity as "any combination of flows that causes the density to reach the LOS E/F boundary condition of 43 pcpmpl for freeways" based on configuration, number of lanes on weaving section, free-flow speed, length of the weave, and volume ratio (VR). However, no research has been focused on the two-sided Type C weaves, which is the most difficult weave to maneuver, where one weaving movement must cross all the freeway lanes.

Recent studies on general weaving area use average vehicle travel speed as a primary parameter to determine the capacity of a weave. However, studies from California [Ref. 30] showed that average speed appears to be rather insensitive to flow up to 1,600 passenger cars per hour per lane (pcphpl). They strongly suggested that average travel speed is not a good measure of effectiveness for freeway weaving area. The research described in this dissertation is not a speed-based model. It has been developed to predict capacity of two-sided Type C weaves for the feasible range of flow combinations within the weaving area. Simulation has been used due to the difficulty of finding field data condition would allow the prediction of capacity.

In this dissertation, simulation is the key element and the VISSIM model was calibrated using data collected in San Antonio to assure that the model behavior reflects observed traffic in the field. Calibration is important because no single simulation model is expected to have the ability to equally represent all possible traffic conditions. The feasible range of flow combinations was used as input in order to determine the capacity for various scenarios. The following conclusions were drawn:

- A capacity model for a two-sided Type C weaving area was developed as a function of mainlane flow, exit ramp flow, and R-R flow.
- The sum of entrance ramp flow and lane 1 flow is always about 2000 vph, which is close to the capacity of one freeway lane.
- Entrance ramp capacity and lane 1 capacity are each about 1000 vph if the mainlane demand exceeds 4500 vph.
- At capacity condition, as R-R volume increases, total volume (V) decreases.
- At capacity condition, as exit ramp demand increases, V also decreases because more vehicles from the mainlane and/or entrance ramp will weave to the left lane to exit, which creates more weaving activities in the system.
- Also, when exit ramp demand increases, lane 3 (left lane in 3 mainlanes) flow is always smaller than lane 2 flow.
- Densities of the weaving area at capacity found in this study are around 43 vphpl, which is close to the density at capacity specified by the 2000 HCM.

- The model proposed in this study not only works well when the freeway mainlane has three lanes but it appears that it also can be applied when the freeway has four lanes.

The resulting model is straightforward and appears to behave reasonably, i.e., the signs of the parameters appear to be reasonable. However, this work has by no means answered all questions concerning two-sided Type C weaves. Suggested future works include:

- Even though this model works well when the mainlane section has four lanes, VISSIM should be calibrated using field data collected at a weaving section with 4 mainlanes in order to have more reliable results. Then the comparison between the capacities from simulation output and model predictions should be conducted again to see if they fit better.
- Similarly, this model needs to be tested with the case when the mainlane has only two lanes.
- The calibration process is affected by human driving behavior. As such, it would be interesting to see how well the model works with a 3 lane two-sided Type C weave elsewhere in the US.
- Density is the measure of effectiveness used by the 2000 HCM to assess level of service. In general, the level of service represents the degree of comfort and satisfaction of drivers under specified roadway condition. As a result, a full range of density is needed to build a level of service model. Although some work relating to density has been done in this dissertation,

the development of a level of service model would require many additional runs to identify specific flow conditions which correspond to the specific level of service defined in the 2000 HCM.

Even though two-sided weaving areas are discouraged in current highway design; it is commonly found in managed-lane operations, where vehicles have to cross all freeway mainlanes to get in or out of the managed lane facility. The access point to and from a managed lane is limited; thus, it is important to estimate the capacity of a freeway segment between two access points, which matches the configuration of a two-sided Type C weave.

APPENDIX A

SIMULATION RESULTS FOR STAGE 1

Appendix A includes simulation results of 7 scenarios in stage 1. The inputs of each scenario are summarized in the following table:

Input Volumes (vph)				
Scenario	Mainlane	Exit Ramp	Entrance Ramp	R-R (% of Entrance Ramp Volume)
1	3500	800	50-2000	10%
2	5000	800	50-2000	25%
3	6000	1200	100-2000	25%
4	5000	800	50-2000	10%
5	5500	1200	100-2000	25%
6	6000	800	50-2000	25%
7	4500	800	100-2000	25%

Scenarios 1:

- Total mainlane volume is 3500 vph
- Exit ramp volume is 800 vph
- 10% of entrance ramp volume is the R-R demand
- Entrance ramp is from 50 to 2000 vph

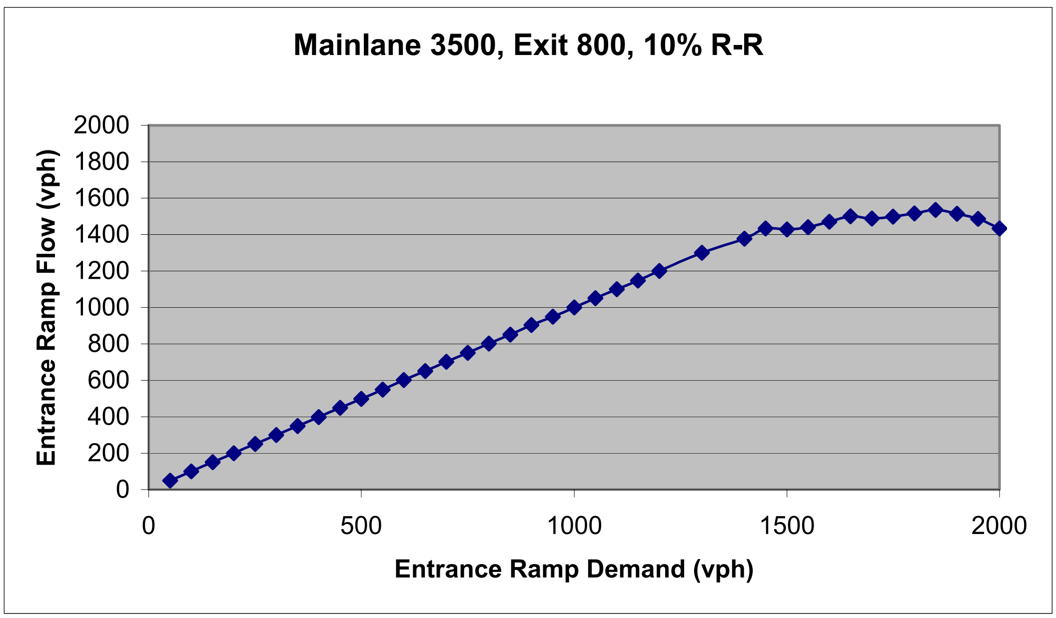

Figure A.1 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 1

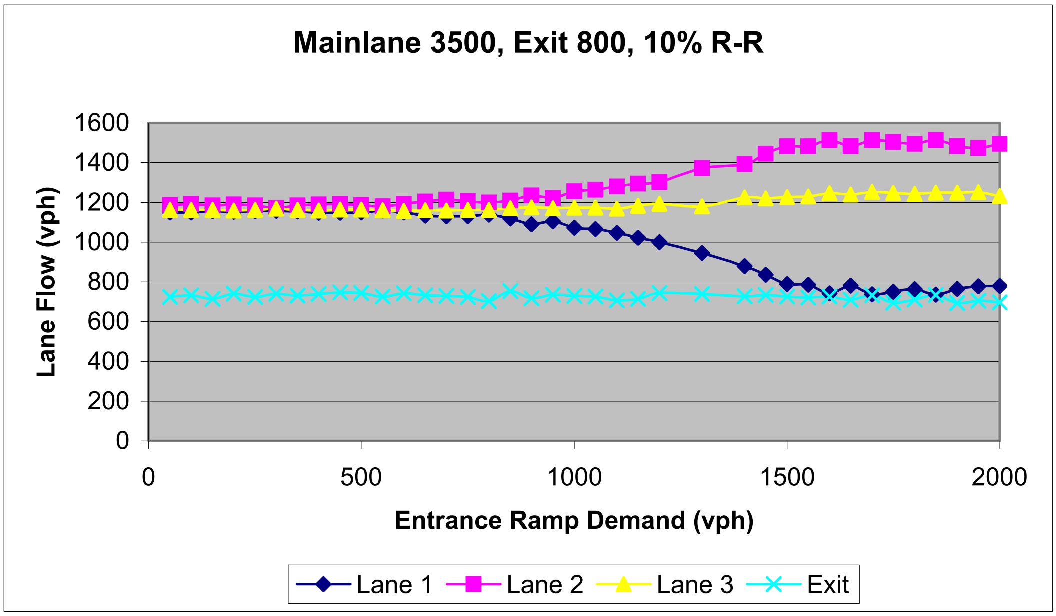

Figure A.2 Entrance Ramp Demand vs Lane Flow for Scenario 1

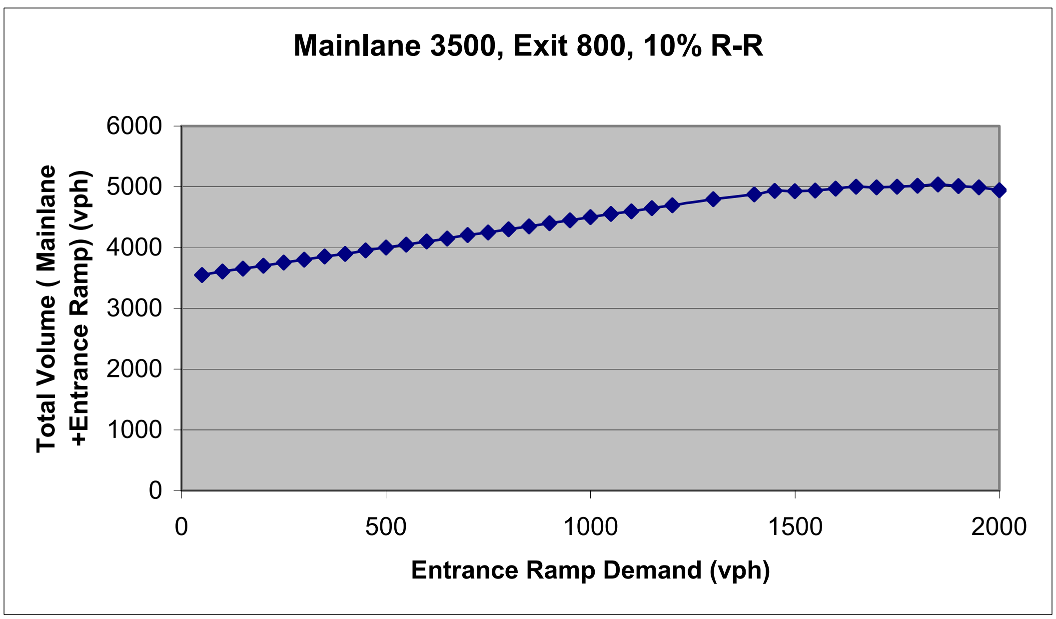

Figure A.3 Entrance Ramp Demand vs Total Volume for Scenario 1

Scenarios 2:

Total mainlane volume is 5000 vph

Exit ramp volume is 800 vph

25% of entrance ramp volume is the R-R demand

Entrance ramp is from 50 to 2000 vph

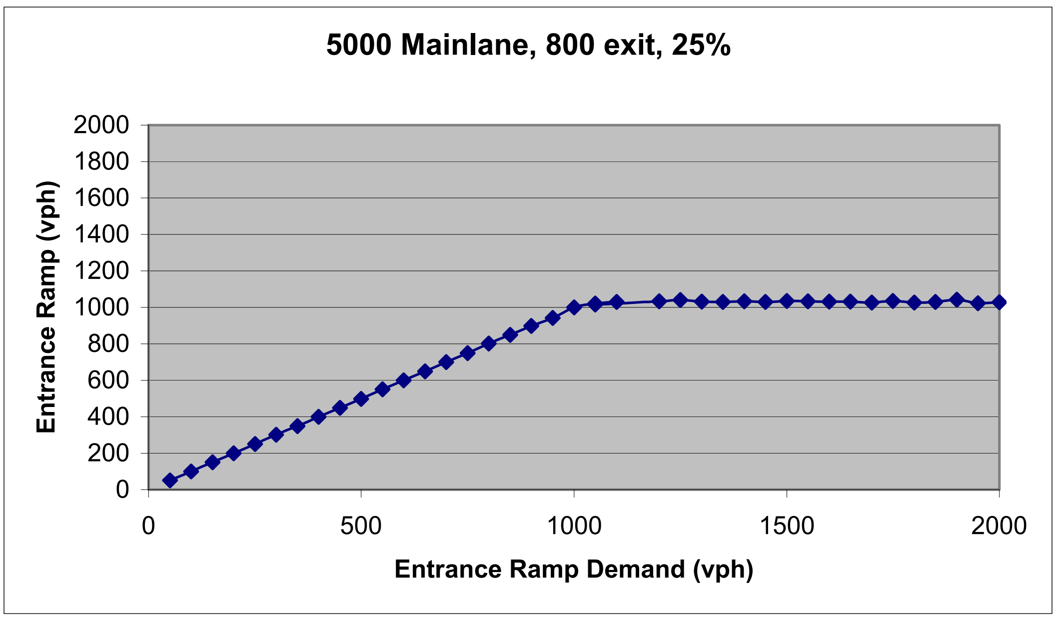

Figure A.4 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 2

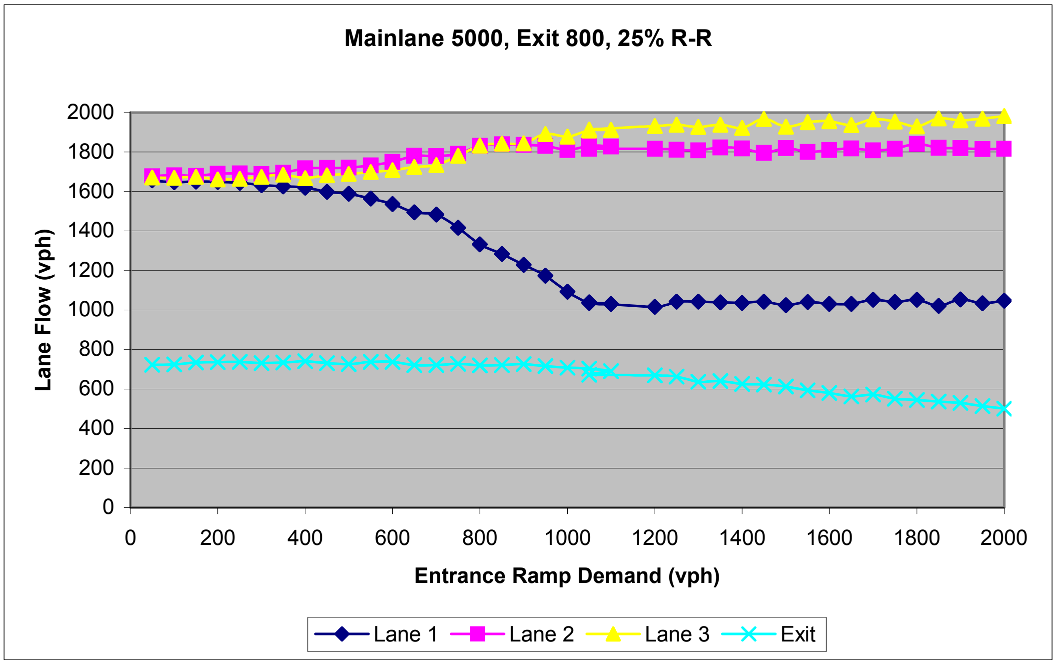

Figure A.5 Entrance Ramp Demand vs Lane Flow for Scenario 2

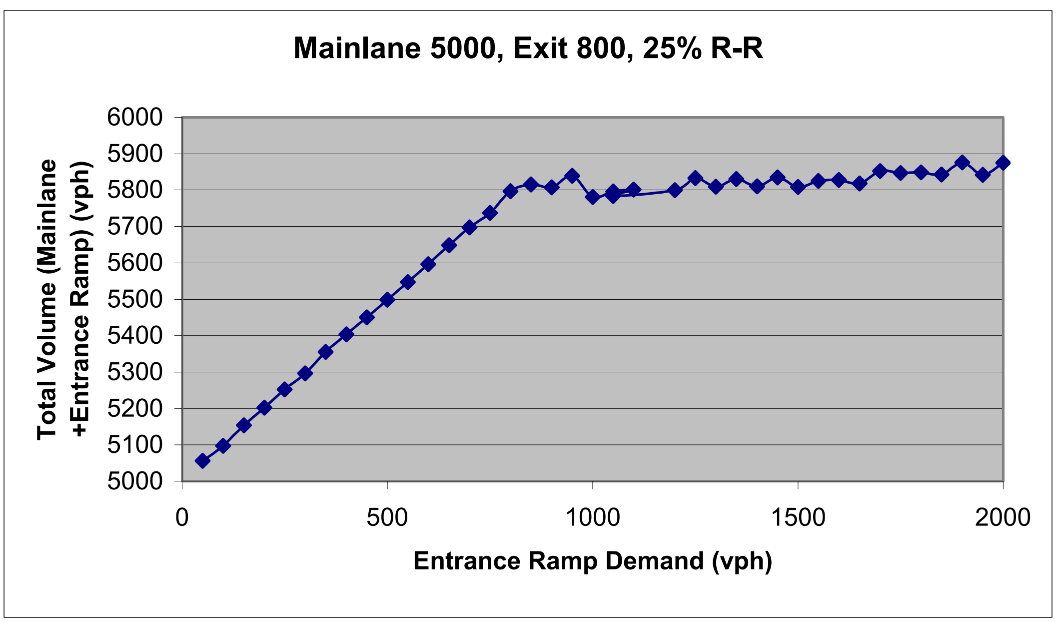

Figure A.6 Entrance Ramp Demand vs Capacity for Scenario 2

Scenarios 3:

Total mainlane volume is 6000 vph

Exit ramp volume is 1200 vph

25% of entrance ramp volume is the R-R demand

Entrance ramp is from 50 to 2000 vph

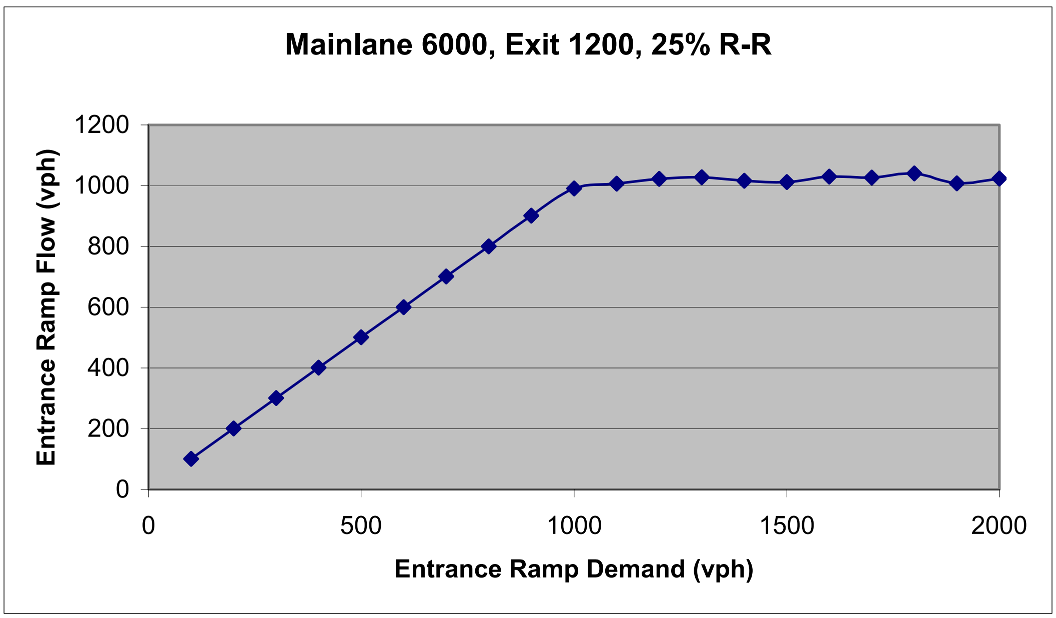

Figure A.7 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 3

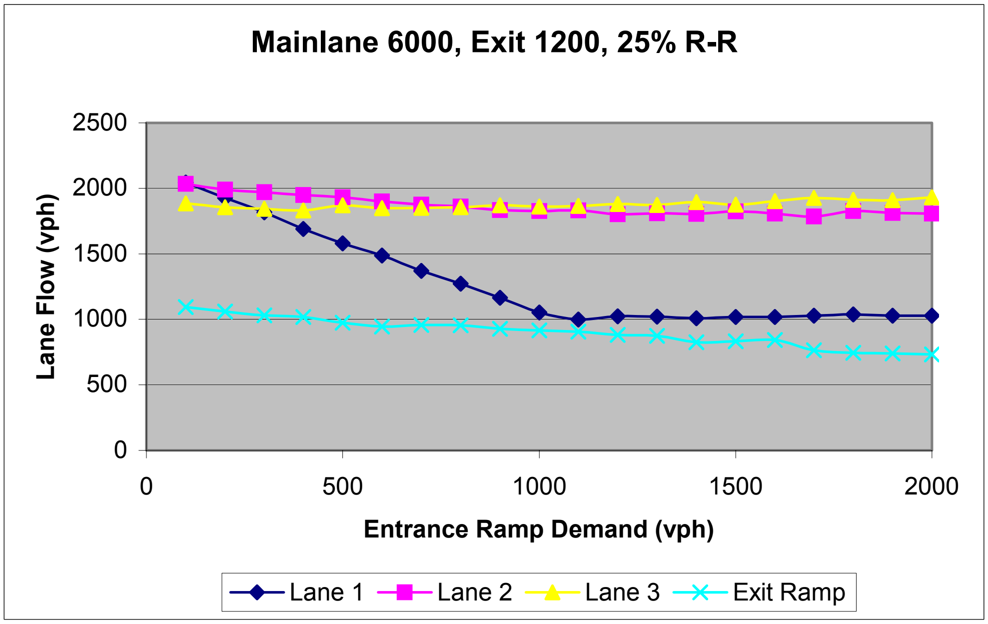

Figure A.8 Entrance Ramp Demand vs Lane Flow for Scenario 3

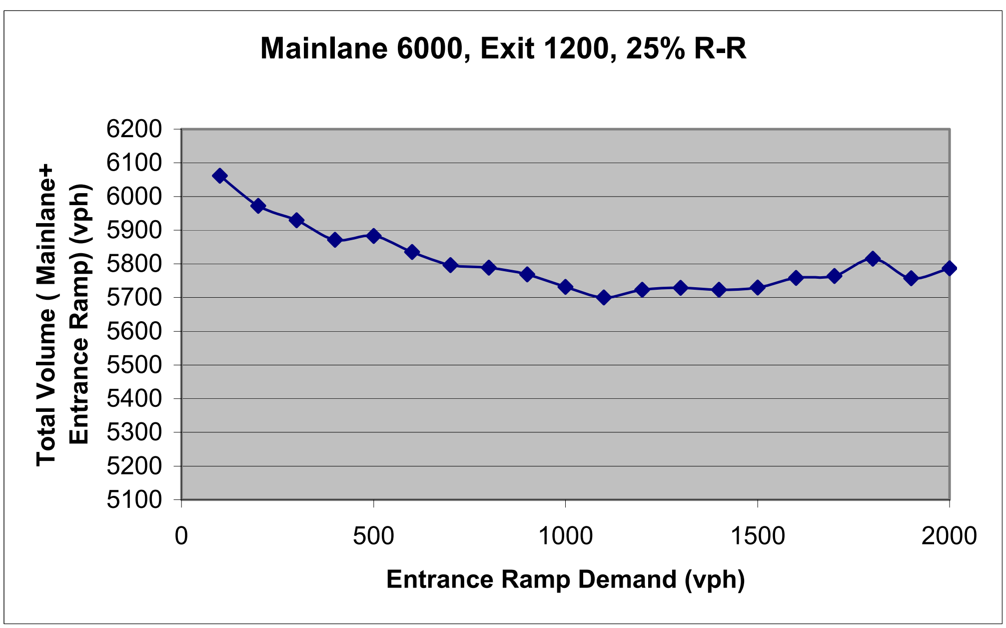

Figure A.9 Entrance Ramp Demand vs Capacity for Scenario 3

Scenarios 4:

Total mainlane volume is 5000 vph

Exit ramp volume is 800 vph

10% of entrance ramp volume is the R-R demand

Entrance ramp is from 50 to 2000 vph

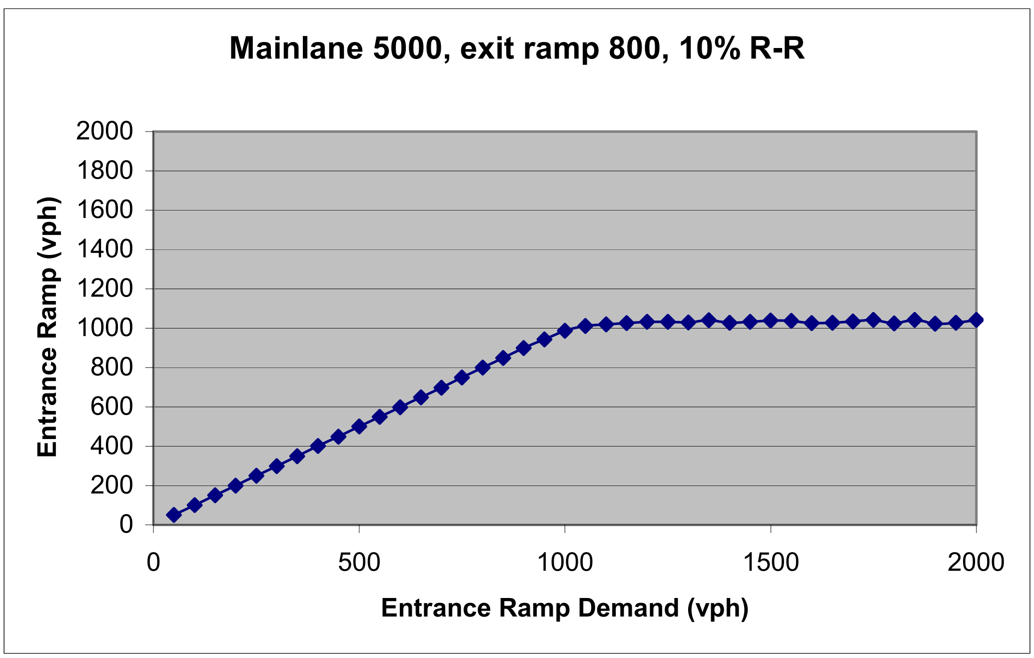

Figure A.10 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 4

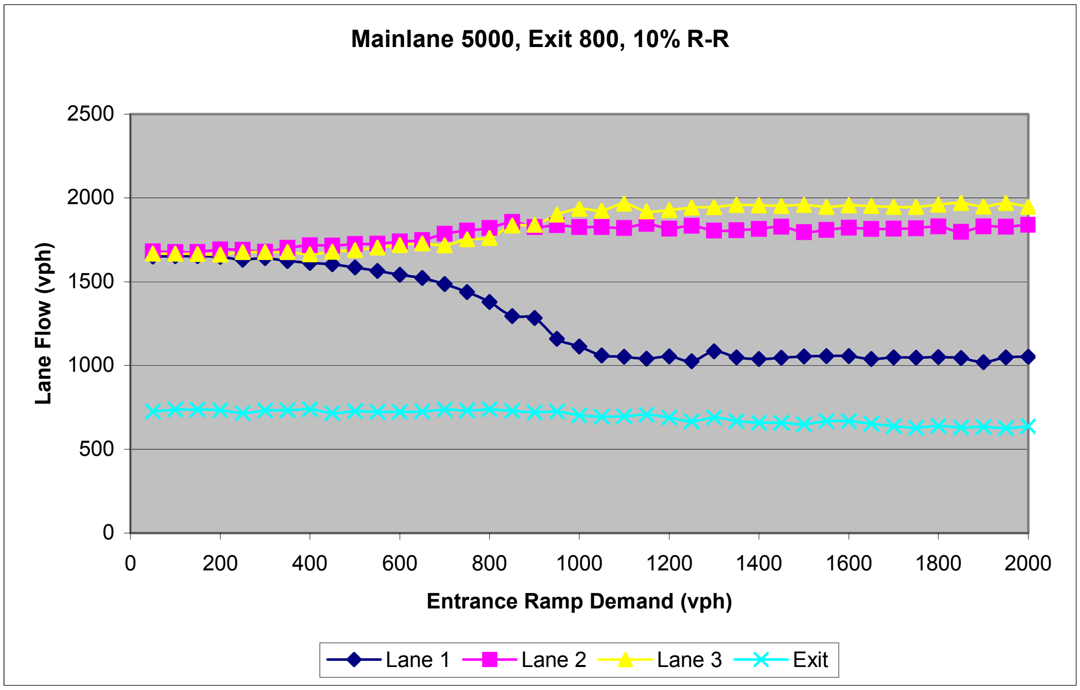

Figure A.11 Entrance Ramp Demand vs Lane Flow for Scenario 4

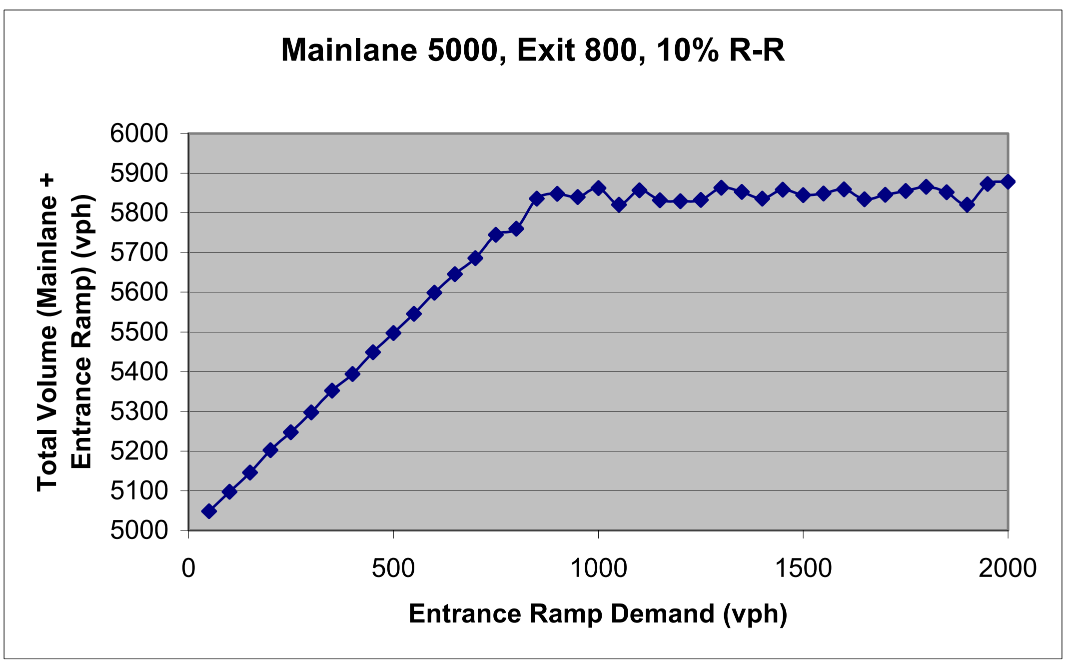

Figure A.12 Entrance Ramp Demand vs Capacity for Scenario 4

Scenarios 5:

Total mainlane volume is 5500 vph

Exit ramp volume is 1200 vph

25% of entrance ramp volume is the R-R demand

Entrance ramp is from 50 to 2000 vph

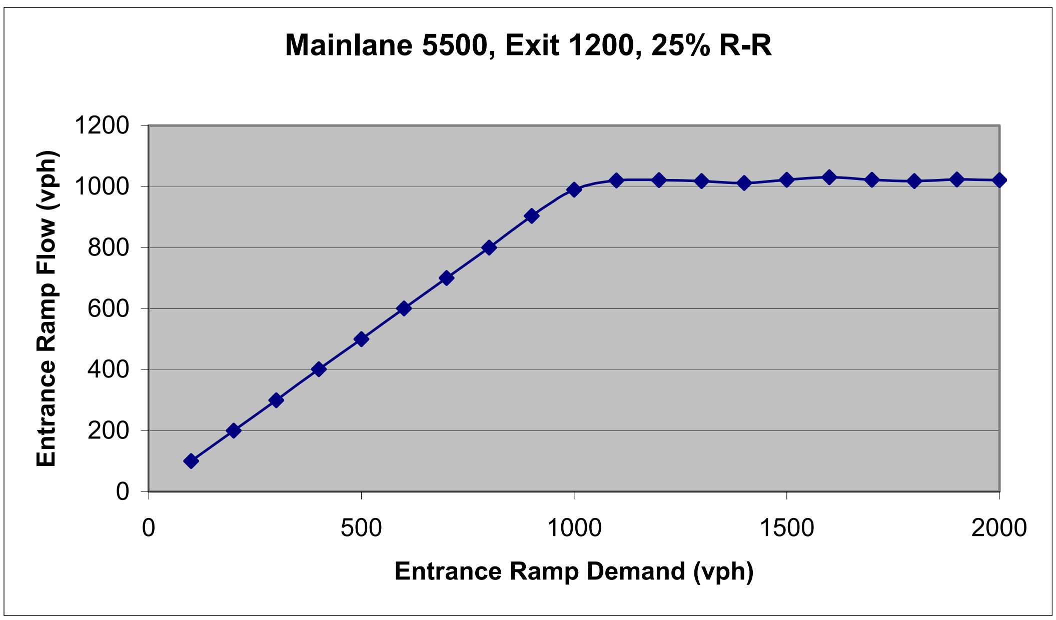

Figure A.13 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 5

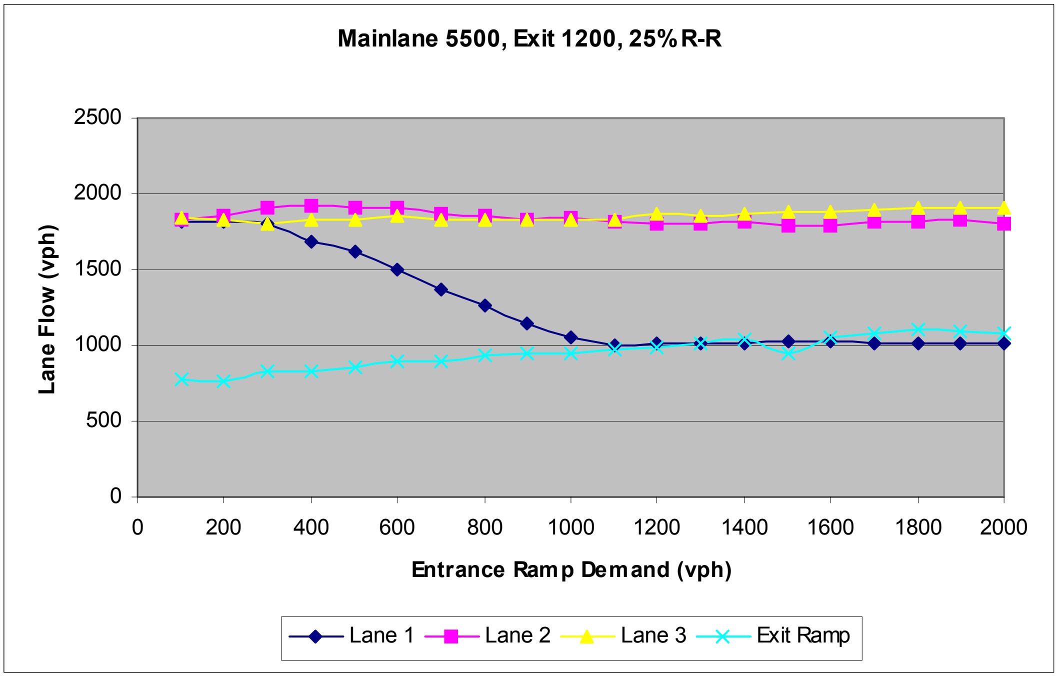

Figure A.13 Entrance Ramp Demand vs Lane Flow for Scenario 5

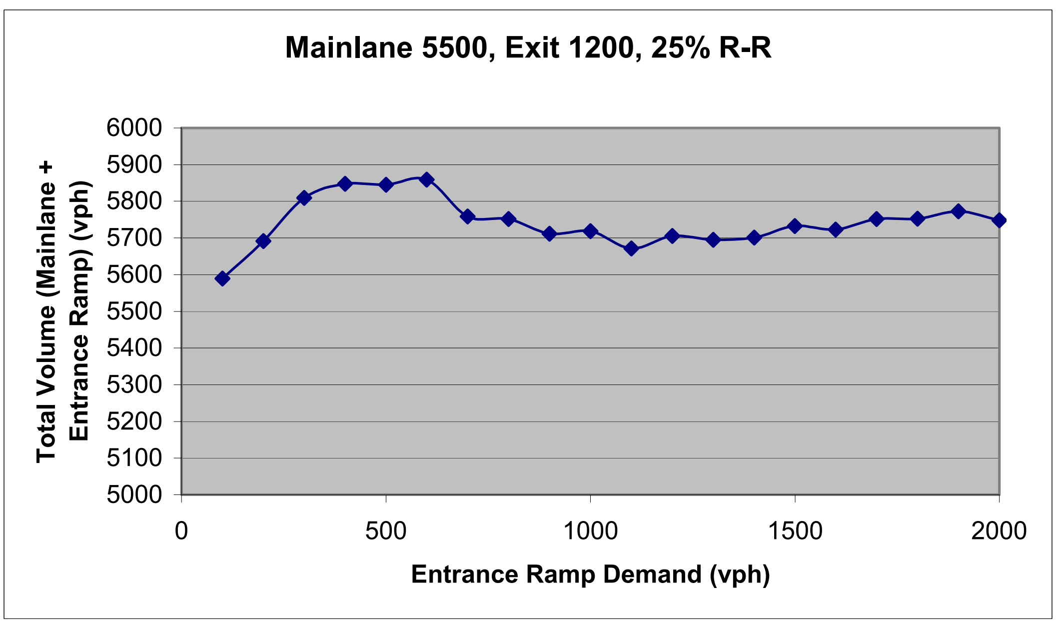

Figure A.14 Entrance Ramp Demand vs Capacity for Scenario 5

Scenarios 6:

Total mainlane volume is 6000 vph

Exit ramp volume is 800 vph

25% of entrance ramp volume is the R-R demand

Entrance ramp is from 50 to 2000 vph

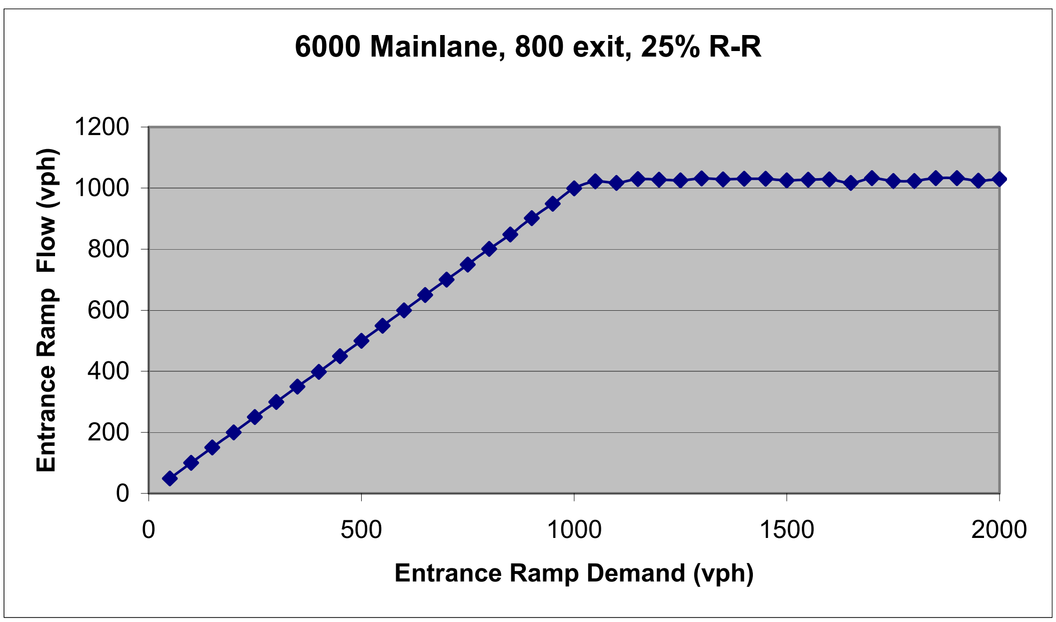

Figure A.15 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 6

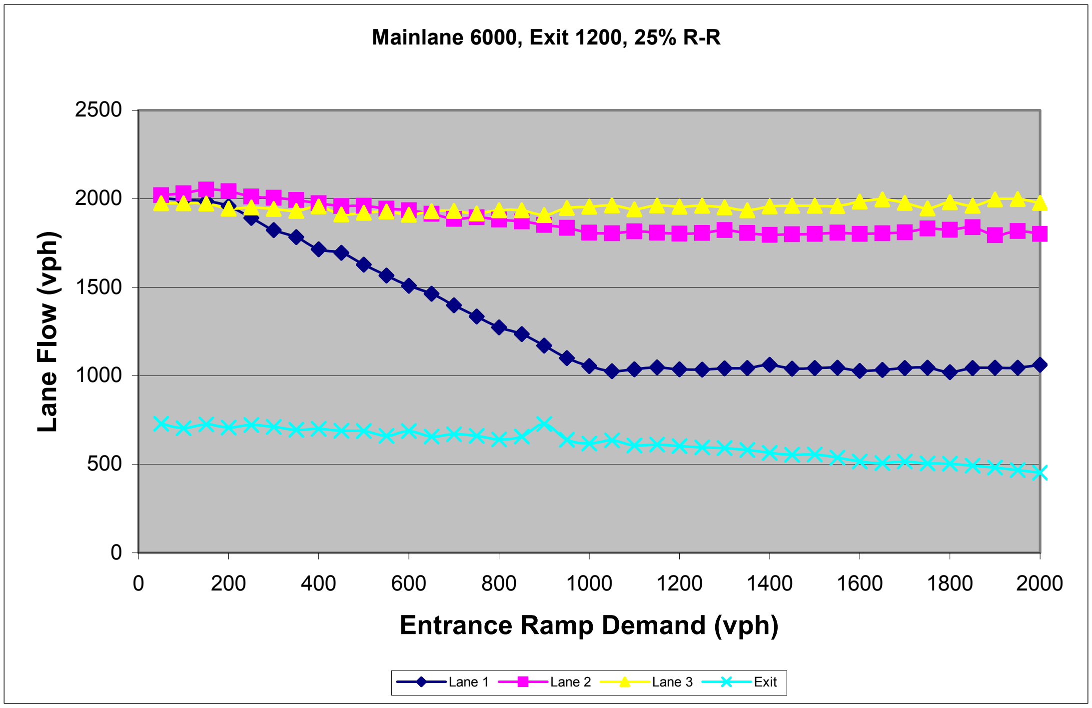

Figure A.16 Entrance Ramp Demand vs Lane Flow for Scenario 6

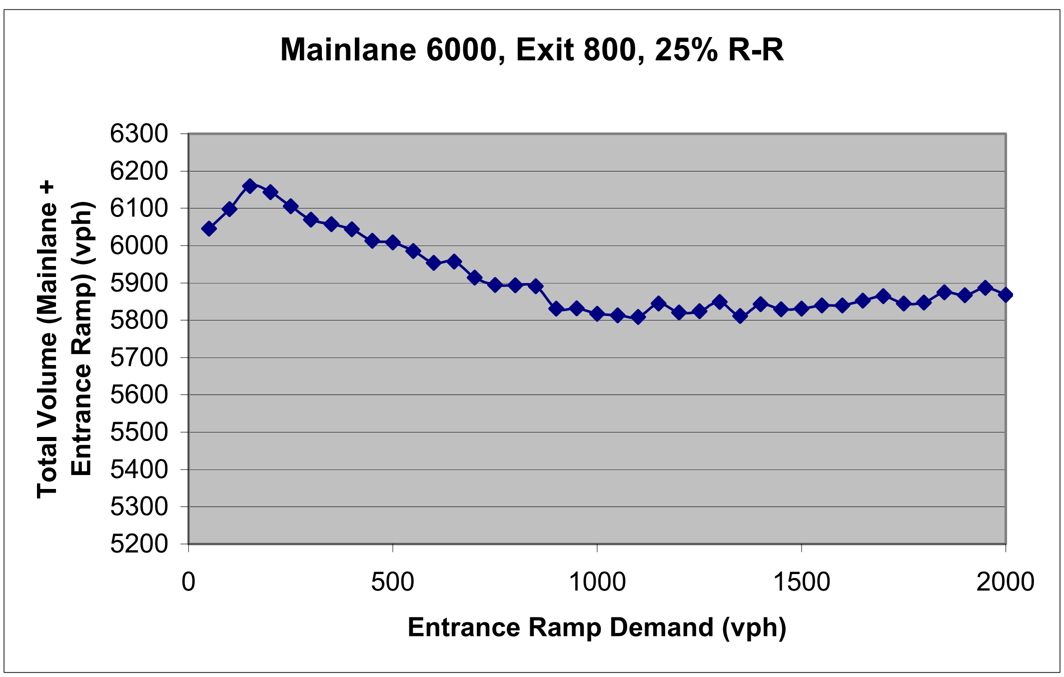

Figure A.17 Entrance Ramp Demand vs Capacity for Scenario 6

Scenarios 7:

Total mainlane volume is 4500 vph

Exit ramp volume is 800 vph

25% of entrance ramp volume is the R-R demand

Entrance ramp is from 100 to 2000 vph

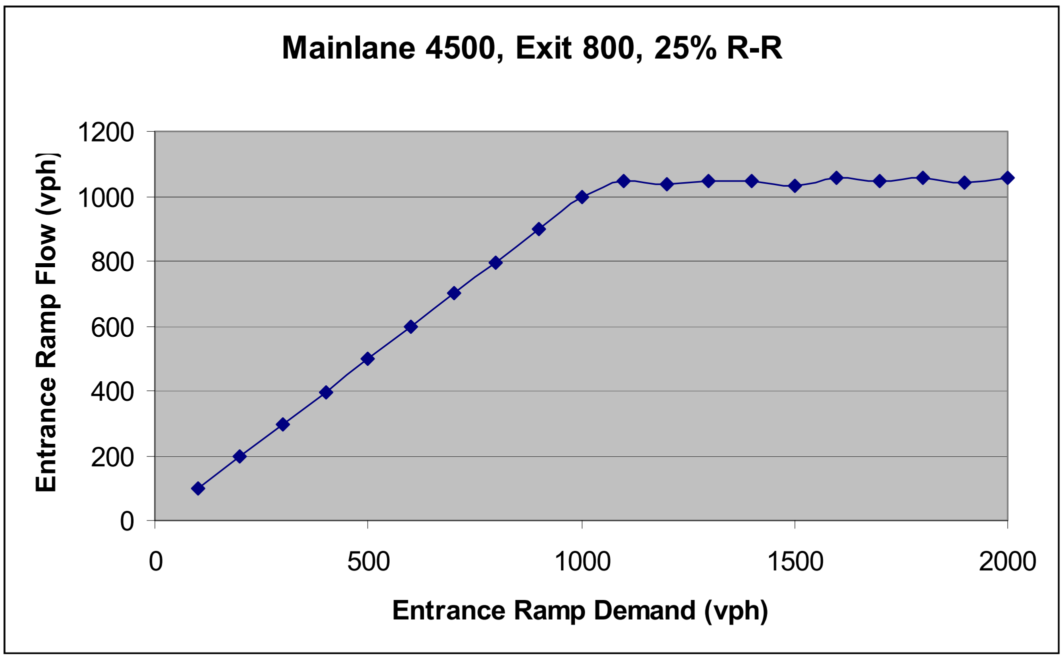

Figure A.18 Entrance Ramp Demand vs Entrance Ramp Flow for Scenario 7

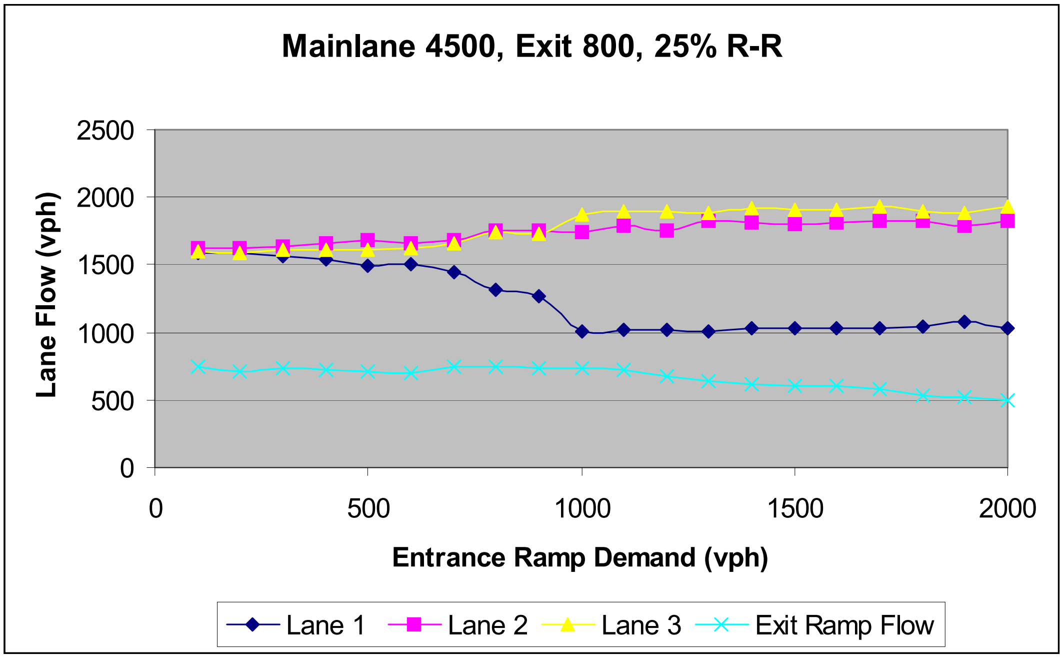

Figure A.19 Entrance Ramp Demand vs Lane Flow for Scenario 7

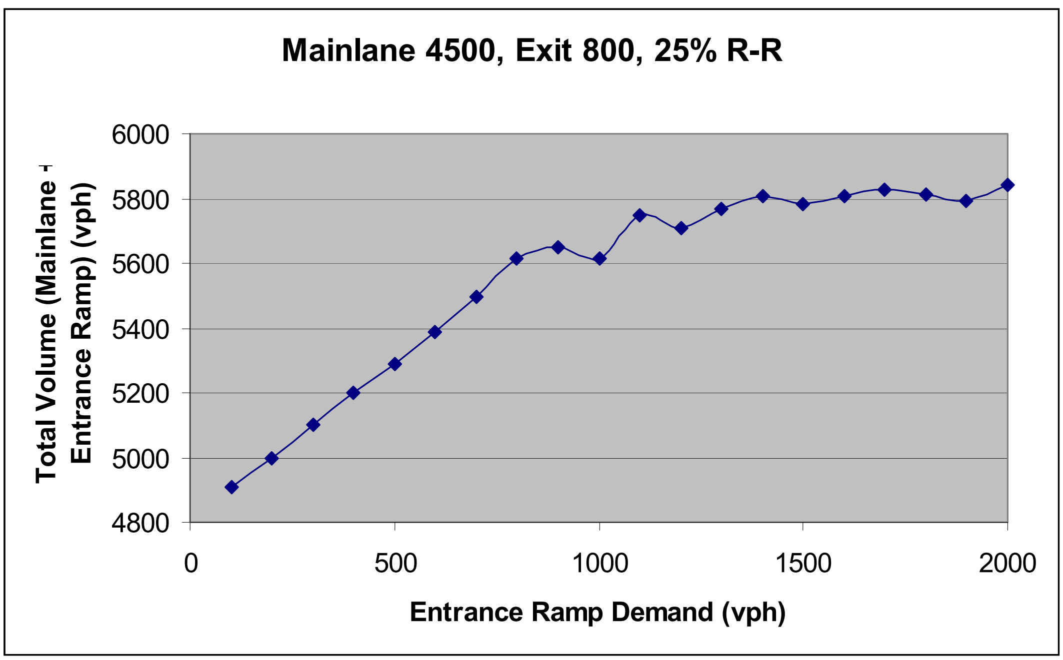

Figure A.20 Entrance Ramp Demand vs Capacity for Scenario 7

APPENDIX B

SIMULATION RESULTS FOR STAGE 2- MODEL 1

Appendix B includes simulation results of 26 scenarios in stage 2 model 1. The inputs of each scenario are summarized in the following table:

Input Volumes (vph)				
Scenario	Mainlane	Exit	Entrance	R-R
1	3500	1000	1000	200-1000
2	3500	1500	1000	200-1000
3	3500	2000	1000	200-1000
4	4000	1000	1000	200-1000
5	4000	1500	1000	200-1000
6	4000	2000	1000	200-1000
7	4500	1000	1000	200-1000
8	4500	1500	1000	200-1000
9	4500	2000	1000	200-1000
10	4800	1500	1000	200-1000
11	4800	2000	1000	200-1000
12	5000	800	1000	200-1000
13	5000	1000	1000	200-1000
14	5000	1500	1000	200-1000
15	5500	1000	1000	200-1000
16	5500	1200	1000	200-1000
17	5500	1500	1000	200-1000
18	5500	2000	1000	200-1000
19	6000	800	1000	200-1000
20	6000	1000	1000	100-1000
21	6000	1200	1000	200-1000
22	6000	1500	1000	200-1000
23	6000	1750	1000	200-1000
24	6000	2000	1000	200-1000
25	6500	1000	1000	200-1000
26	6500	1500	1000	200-1000

Scenarios 1:

- Total mainlane volume is 3500 vph
- Exit ramp volume is 1000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
3500	1000	1000	200	3463	898	990	184	4453	2932	0.658
3500	1000	1000	400	3463	909	987	398	4449	3349	0.753
3500	1000	1000	600	3467	904	988	591	4455	3745	0.841
3500	1000	1000	800	3458	917	985	798	4443	4136	0.931
3500	1000	1000	1000	3468	909	985	909	4453	4377	0.983

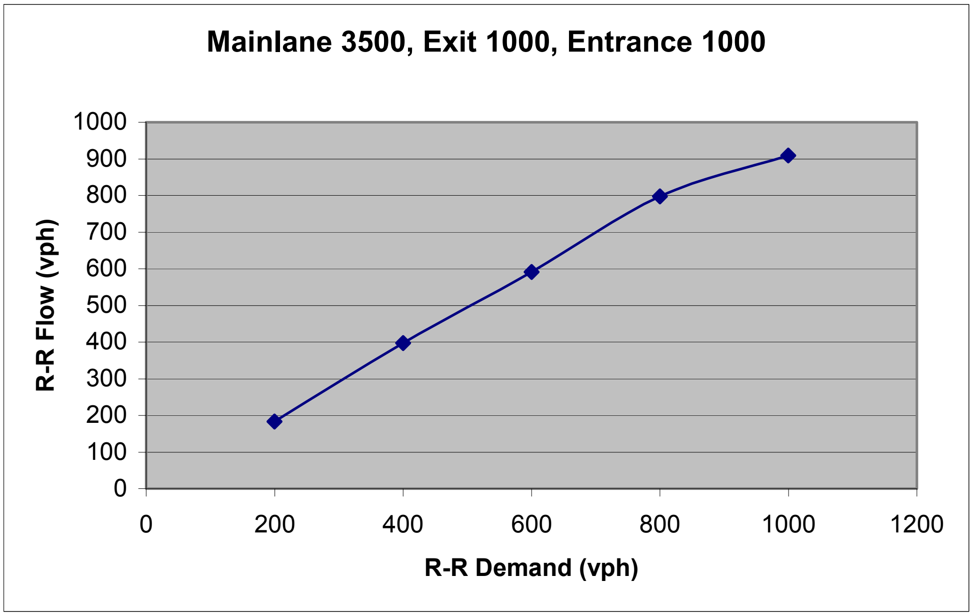

Figure B1 R-R Demand vs R-R Flow For Scenario 1

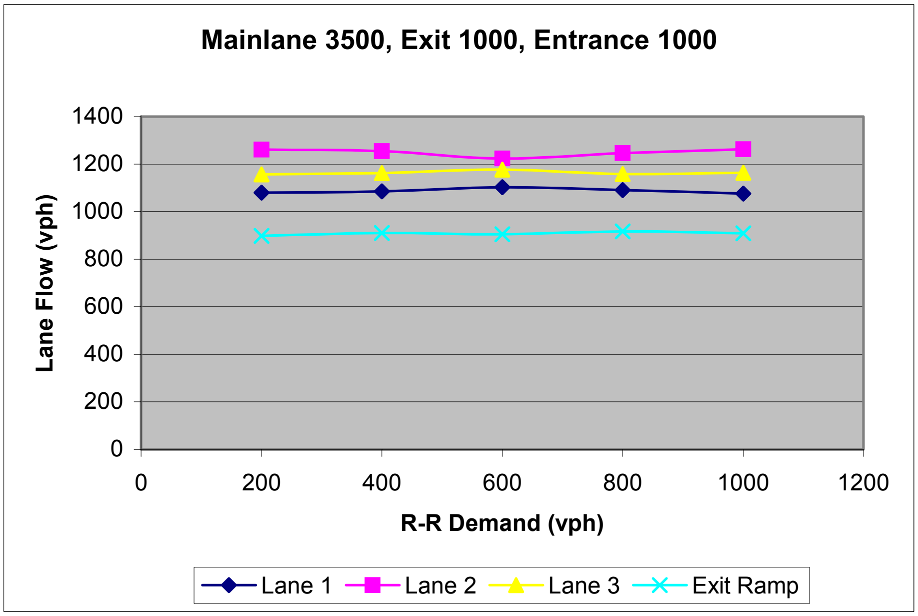

Figure B2 R-R Demand vs Lane Flow For Scenario 1

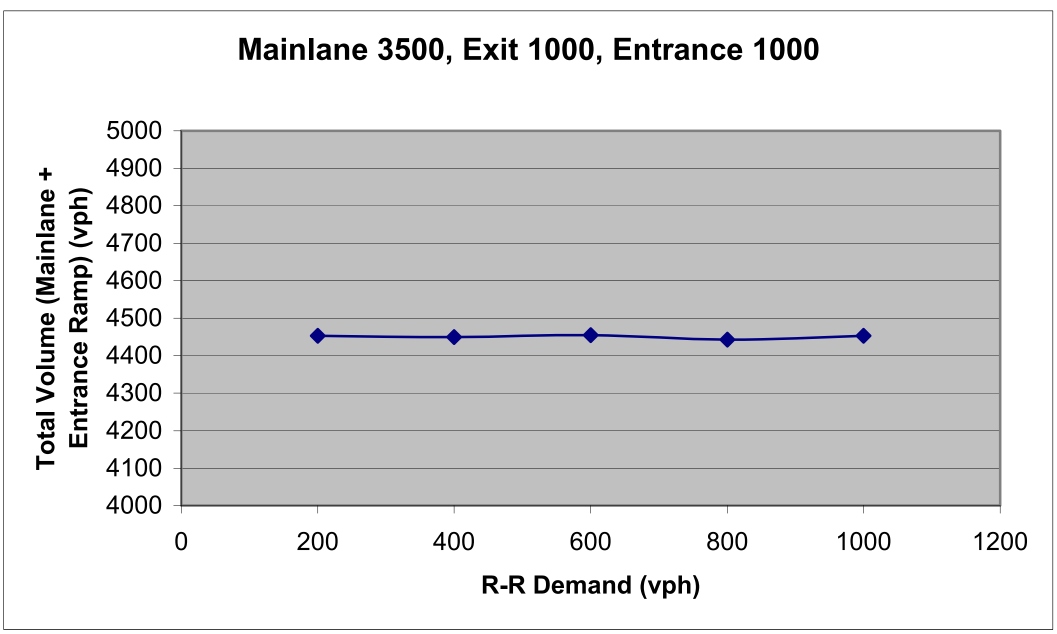

Figure B3 R-R Demand vs Total Volume (V) For Scenario 1

Scenarios 2:

- Total mainlane volume is 3500 vph
- Exit ramp volume is 1500 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
3500	1500	1000	200	3466	1356	986	201	4452	2512	0.564
3500	1500	1000	400	3463	1368	994	403	4457	2901	0.651
3500	1500	1000	600	3477	1383	988	603	4465	3300	0.739
3500	1500	1000	800	3466	1378	990	784	4456	3656	0.820
3500	1500	1000	1000	3459	1407	986	914	4445	3880	0.873

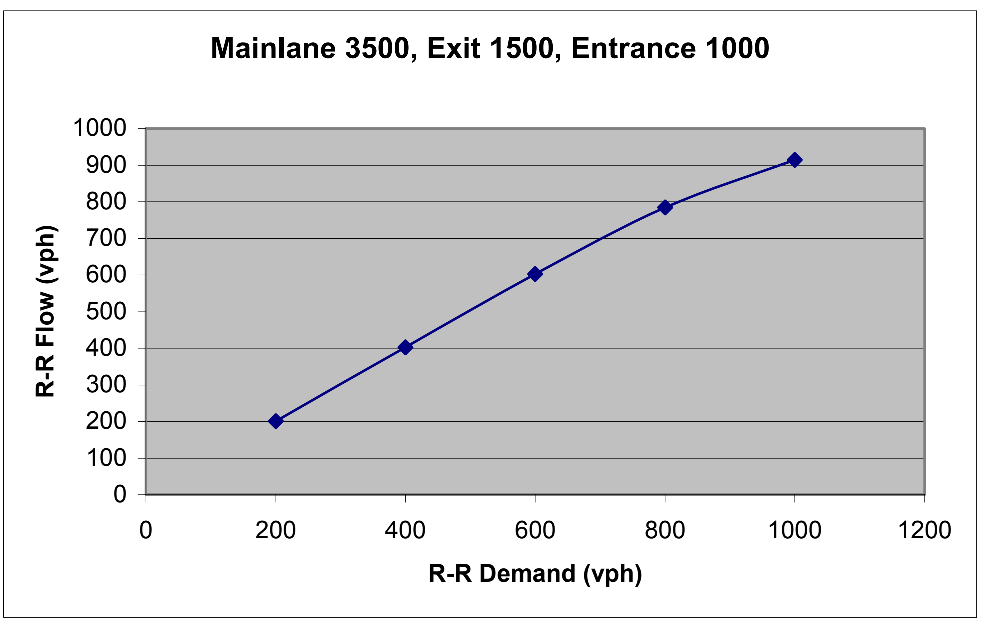

Figure B4 R-R Demand vs R-R Flow For Scenario 2

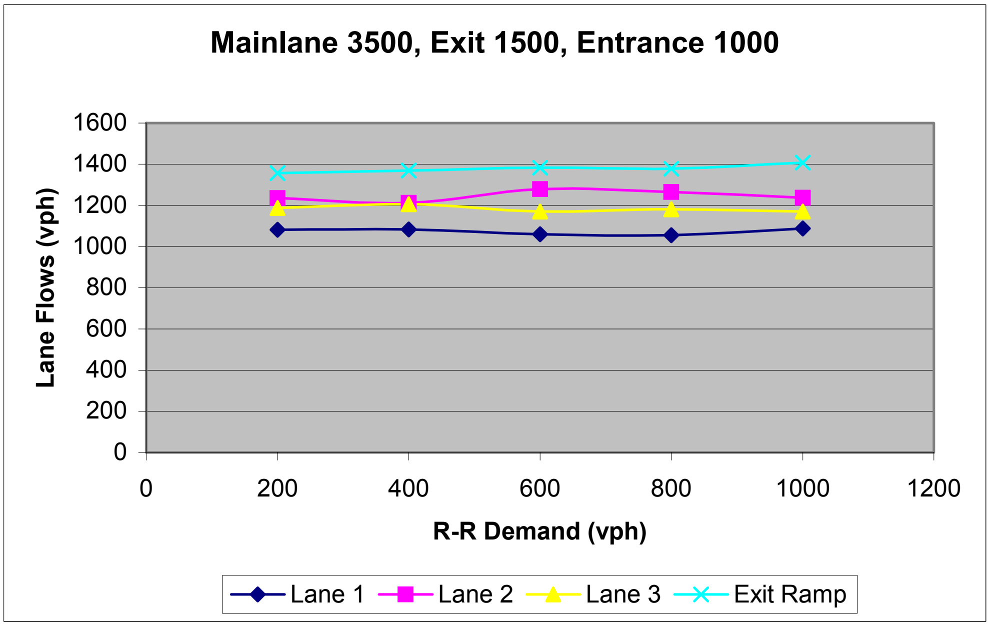

Figure B5 R-R Demand vs Lane Flow For Scenario 2

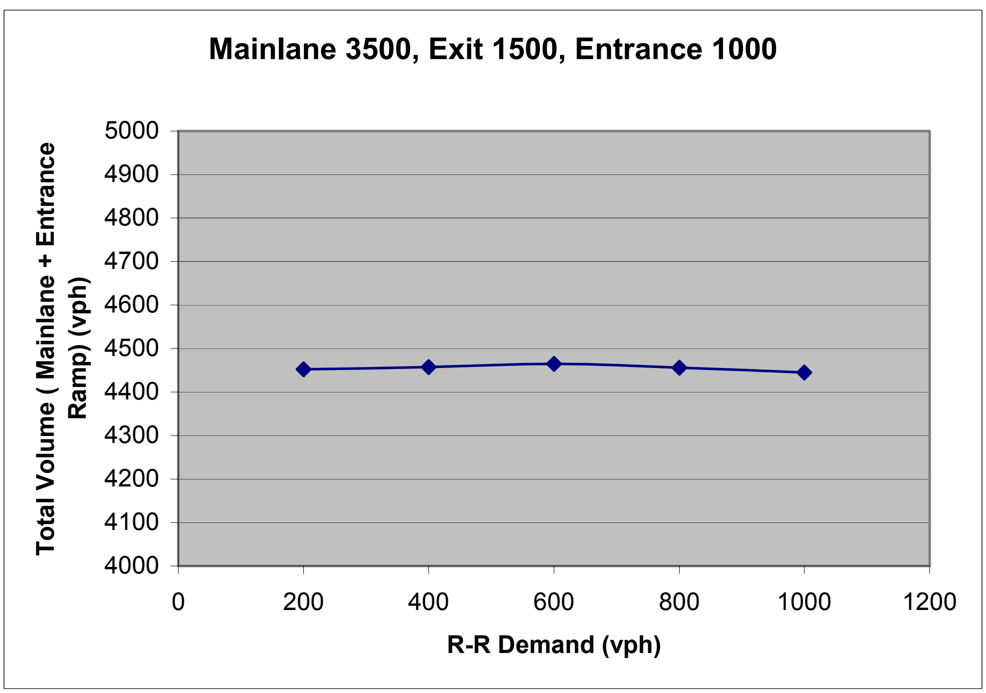

Figure B6 R-R Demand vs Total Volume (V) For Scenario 2

Scenarios 3:

- Total mainlane volume is 3500 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
3500	2000	1000	200	3466	1840	984	196	4450	2018	0.453
3500	2000	1000	400	3457	1819	983	384	4440	2406	0.542
3500	2000	1000	600	3467	1819	985	587	4452	2822	0.634
3500	2000	1000	800	3464	1804	985	789	4449	3238	0.728
3500	2000	1000	1000	3456	1842	984	917	4440	3448	0.777

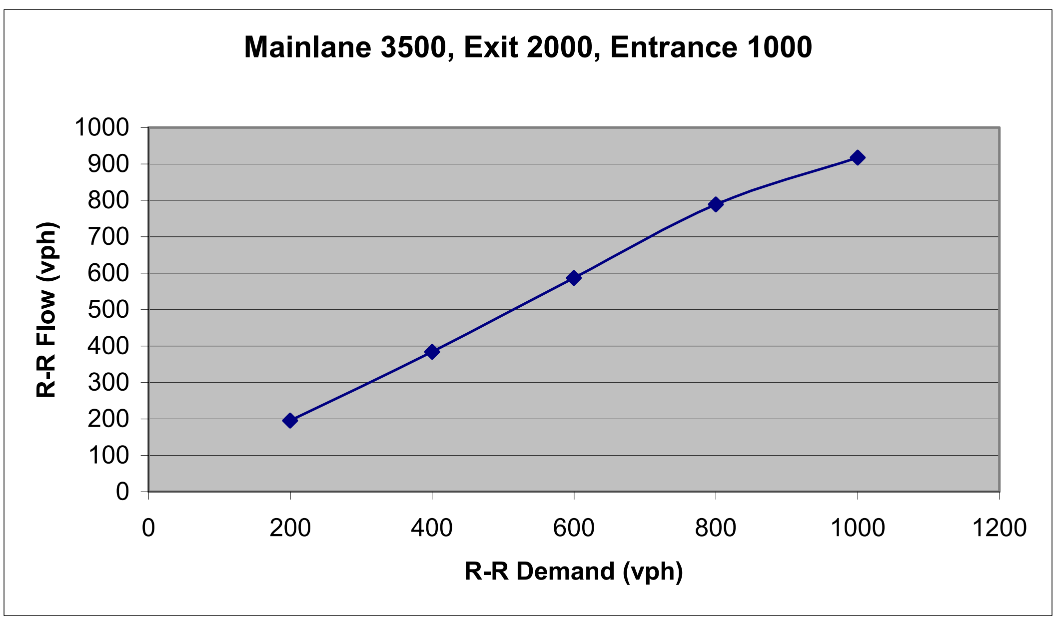

Figure B7 R-R Demand vs R-R Flow For Scenario 3

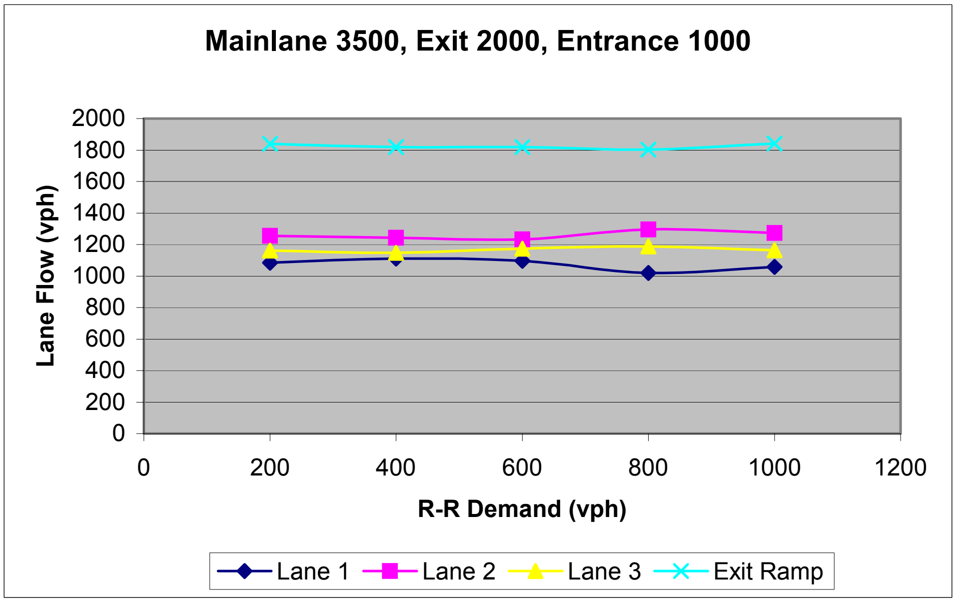

Figure B8 R-R Demand vs Lane Flow For Scenario 3

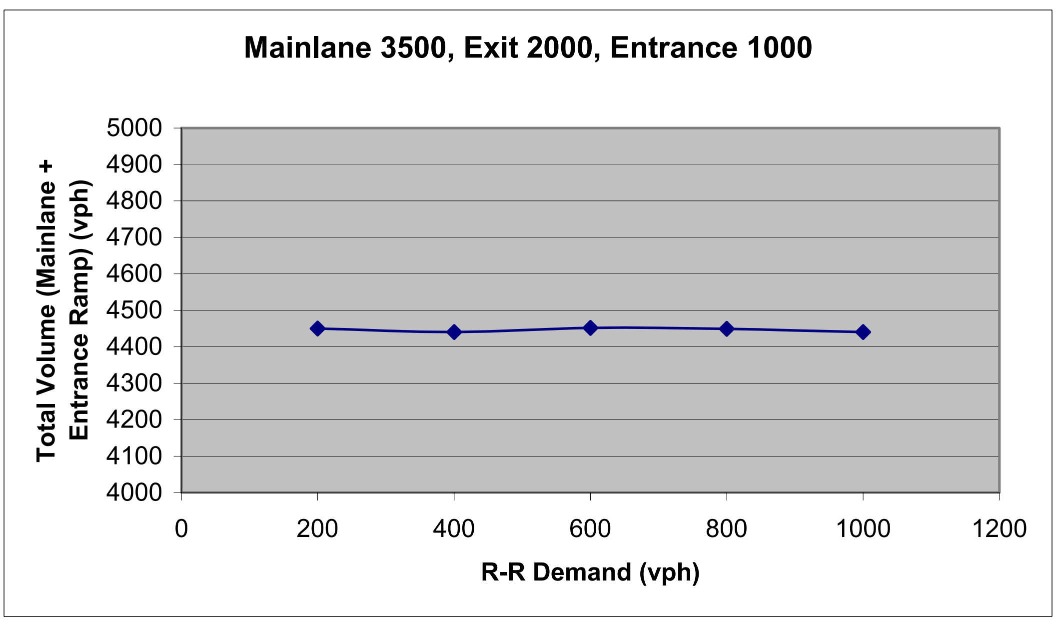

Figure B9 R-R Demand vs Total Volume (V) For Scenario 3

Scenarios 4:

- Total mainlane volume is 4000 vph
- Exit ramp volume is 1000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
4000	1000	1000	200	3981	930	982	210	4963	3471	0.699
4000	1000	1000	400	3952	895	986	412	4938	3881	0.786
4000	1000	1000	600	3957	953	990	603	4947	4210	0.851
4000	1000	1000	800	3966	922	984	795	4950	4634	0.936
4000	1000	1000	1000	3944	906	994	906	4938	4850	0.982

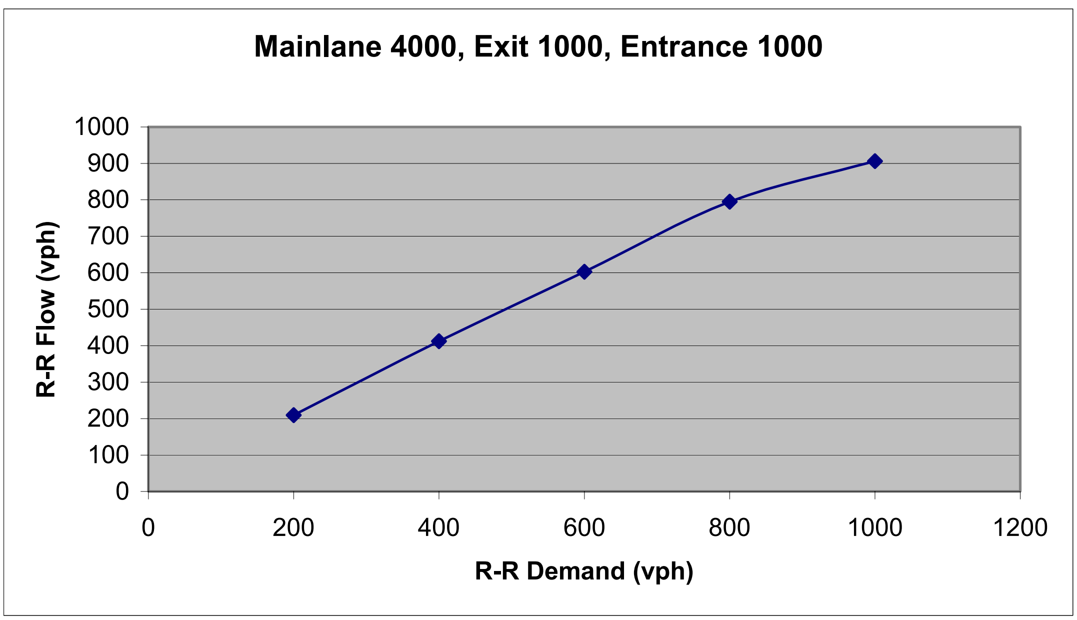

Figure B10 R-R Demand vs R-R Flow For Scenario 4

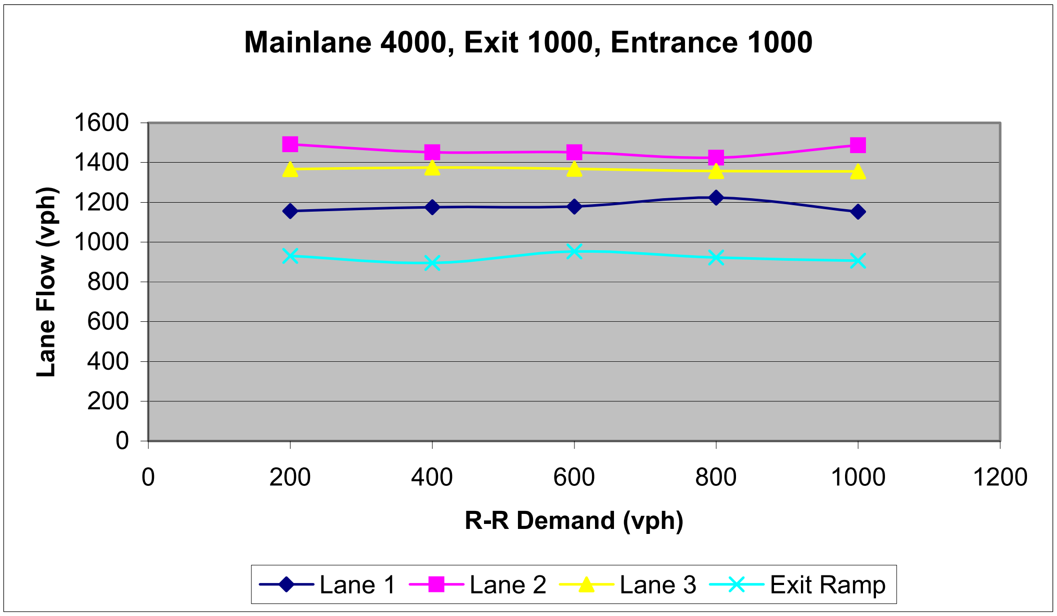

Figure B11 R-R Demand vs Lane Flow For Scenario 4

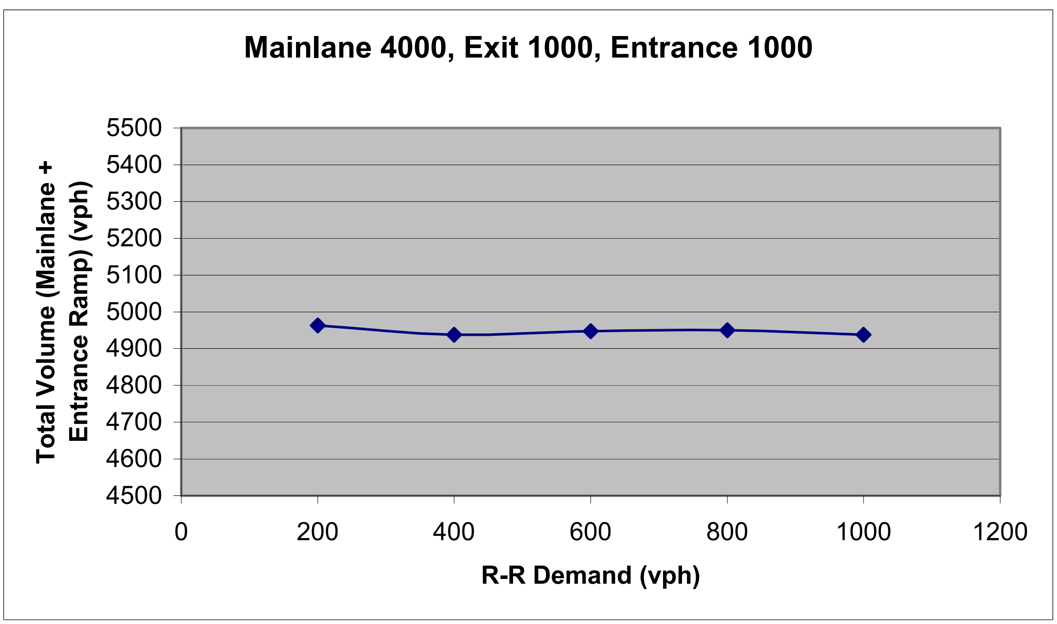

Figure B12 R-R Demand vs Total Volume (V) For Scenario 4

Scenarios 5:

- Total mainlane volume is 4000 vph
- Exit ramp volume is 1500 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
4000	1500	1000	200	3965	1388	985	211	4950	2999	0.606
4000	1500	1000	400	3950	1339	983	384	4933	3379	0.685
4000	1500	1000	600	3950	1359	977	576	4927	3743	0.760
4000	1500	1000	800	3954	1363	983	789	4937	4169	0.844
4000	1500	1000	1000	3935	1364	991	916	4926	4403	0.894

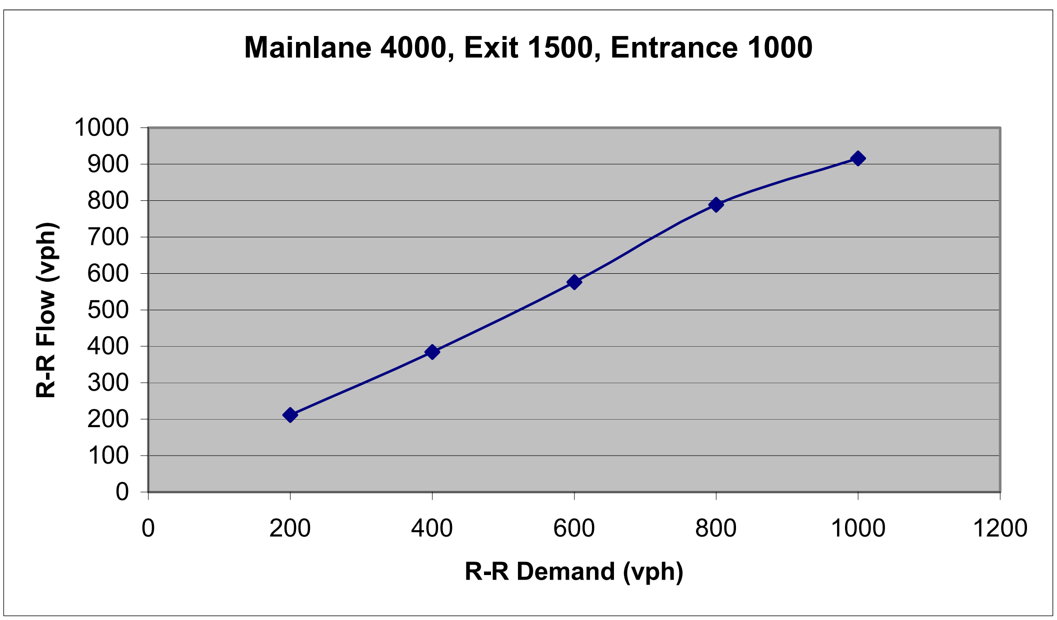

Figure B13 R-R Demand vs R-R Flow For Scenario 5

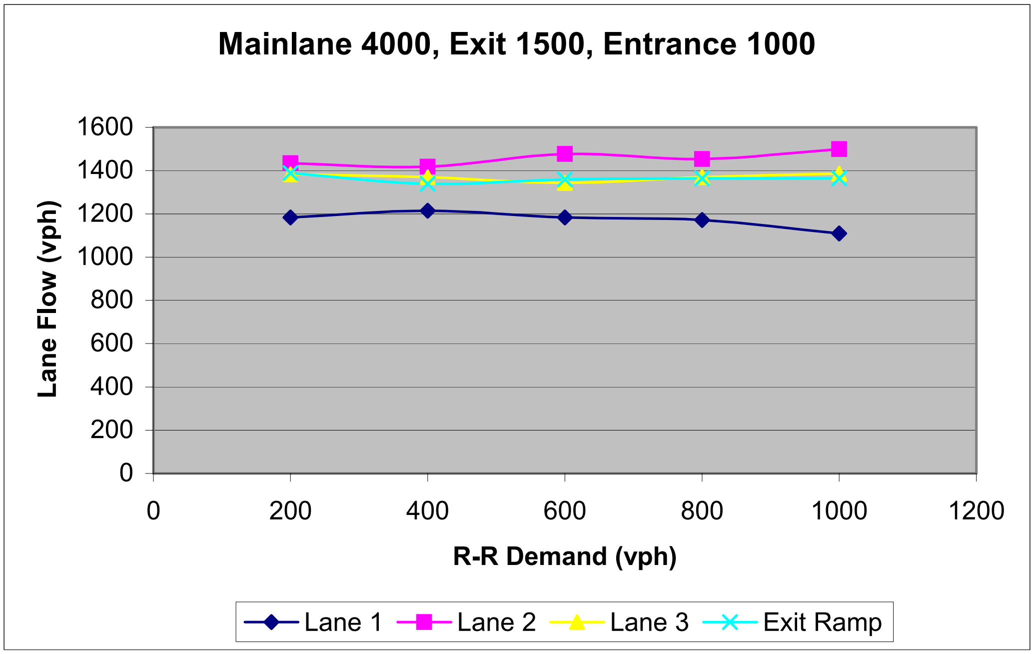

Figure B14 R-R Demand vs Lane Flow For Scenario 5

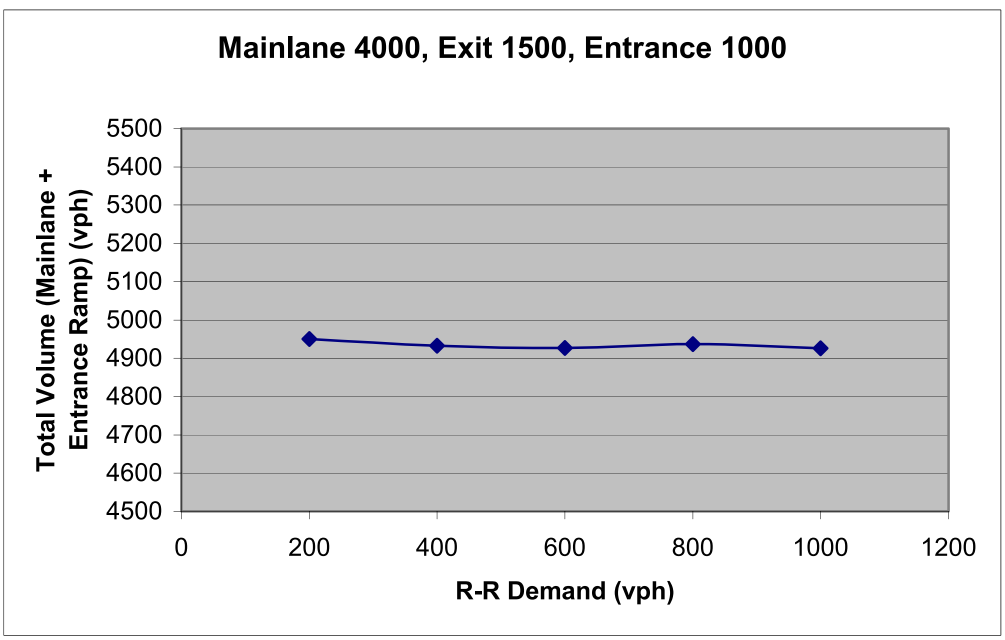

Figure B15 R-R Demand vs Total Volume (V) For Scenario 5

Scenarios 6:

- Total mainlane volume is 4000 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
4000	2000	1000	200	3936	1853	989	189	4925	2461	0.500
4000	2000	1000	400	3972	1849	988	395	4960	2913	0.587
4000	2000	1000	600	3964	1843	982	604	4946	3329	0.673
4000	2000	1000	800	3936	1814	982	779	4918	3680	0.748
4000	2000	1000	1000	3905	1767	982	890	4887	3918	0.802

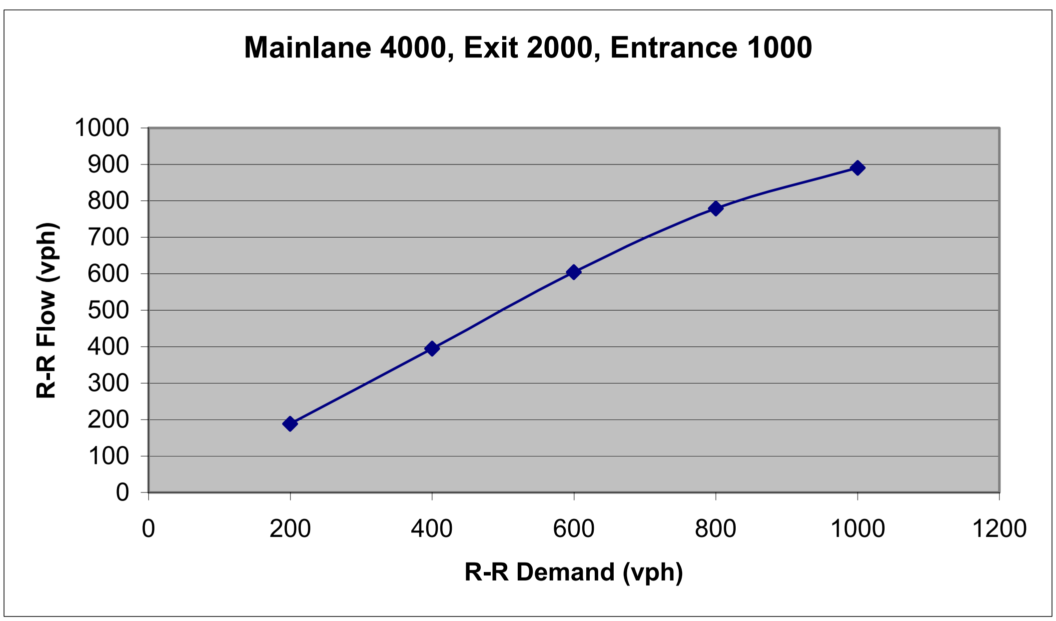

Figure B16 R-R Demand vs R-R Flow For Scenario 6

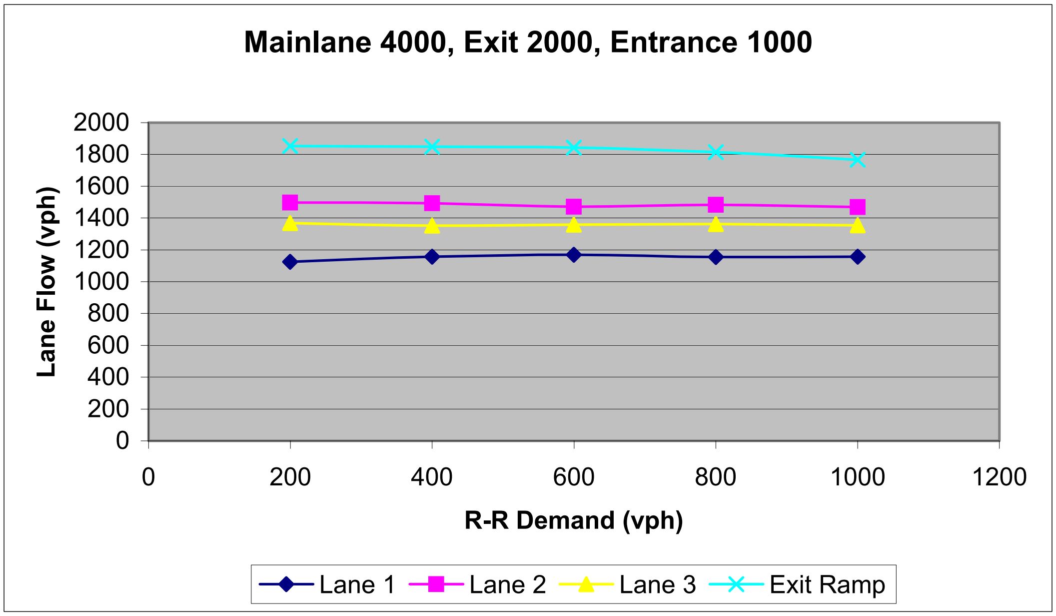

Figure B17 R-R Demand vs Lane Flow For Scenario 6

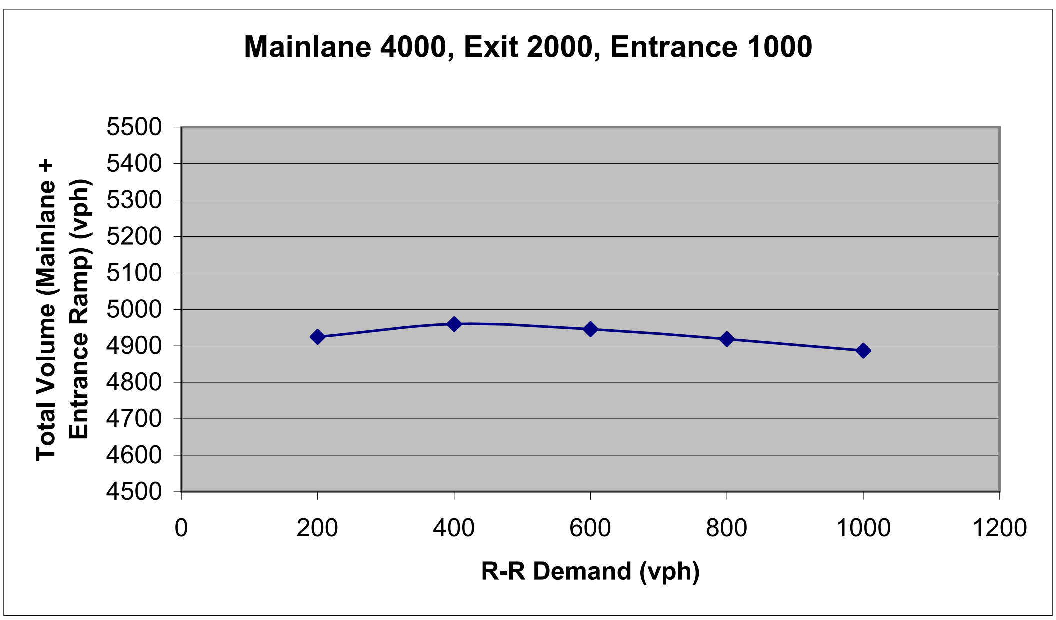

Figure B18 R-R Demand vs Total Volume (V) For Scenario 6

Scenarios 7:

- Total mainlane volume is 4500 vph
- Exit ramp volume is 1000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
4500	1000	1000	200	4455	925	986.333	206	5441	3942	0.724
4500	1000	1000	400	4431	934	970	373	5401	4243	0.786
4500	1000	1000	600	4405	905	993	585	5398	4670	0.865
4500	1000	1000	800	4437	914	983	785	5420	5093	0.940
4500	1000	1000	1000	4407	875	947	875	5354	5282	0.987

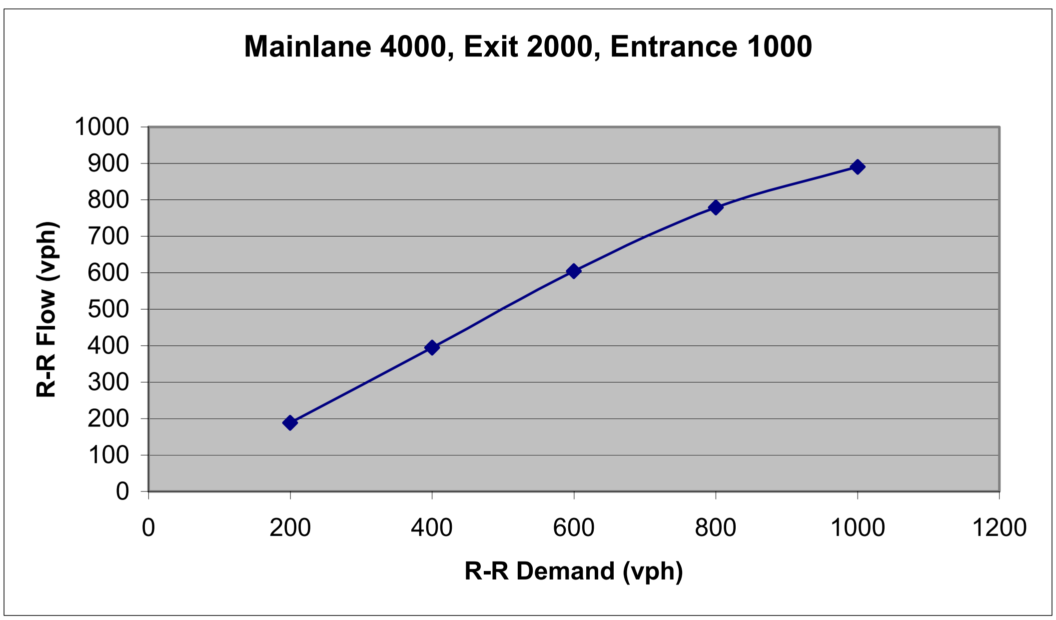

Figure B19 R-R Demand vs R-R Flow For Scenario 7

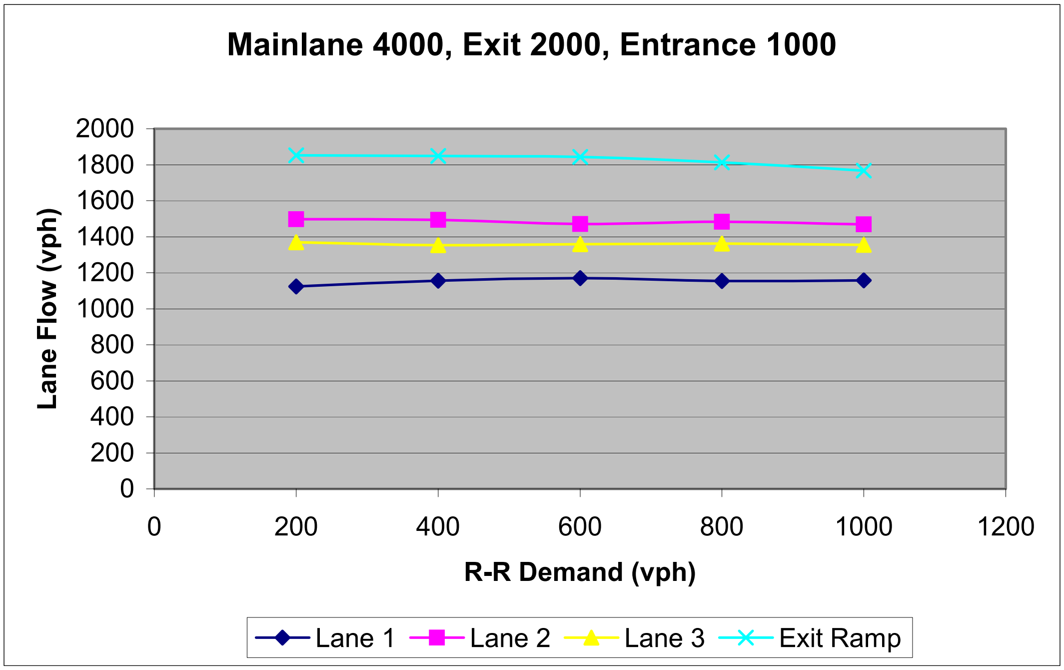

Figure B20 R-R Demand vs Lane Flow For Scenario 7

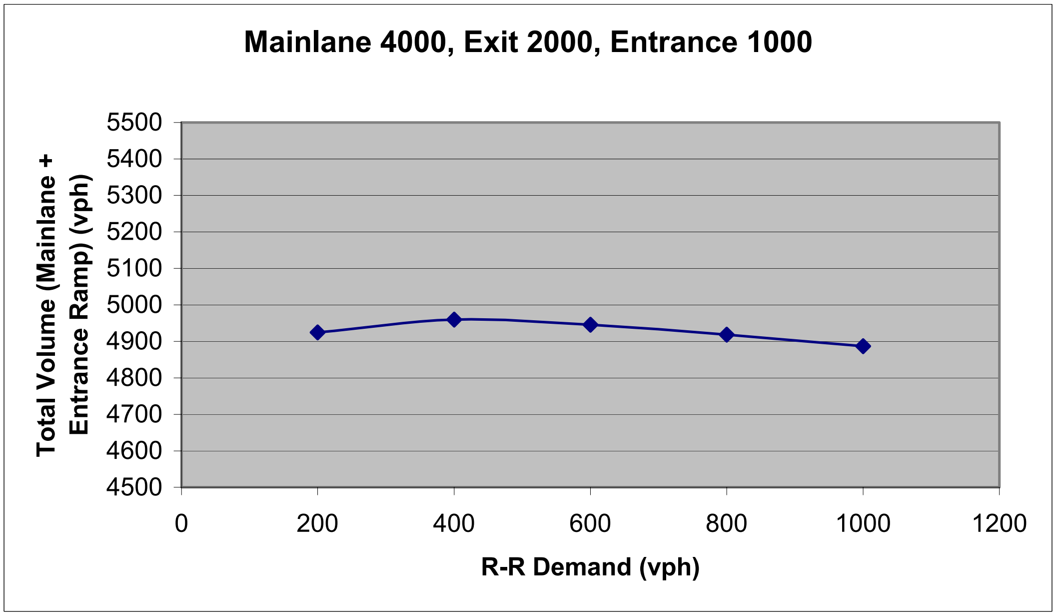

Figure B21 R-R Demand vs Total Volume (V) For Scenario 7

Scenarios 8:

- Total mainlane volume is 4500 vph
- Exit ramp volume is 1500 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
4500	1500	1000	200	4440	1366	984	197	5424	3468	0.639
4500	1500	1000	400	4434	1332	986	386	5420	3874	0.715
4500	1500	1000	600	4390	1303	974	545	5364	4177	0.779
4500	1500	1000	800	4320	1367	982	777	5302	4507	0.850
4500	1500	1000	1000	4356	1324	978	888	5334	4808	0.901

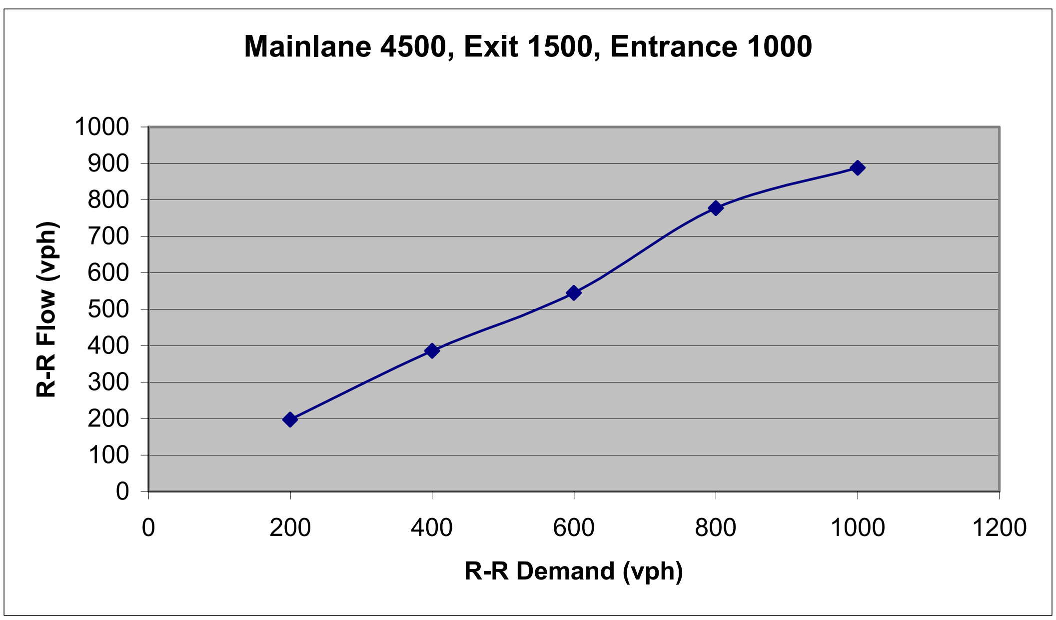

Figure B22 R-R Demand vs R-R Flow For Scenario 8

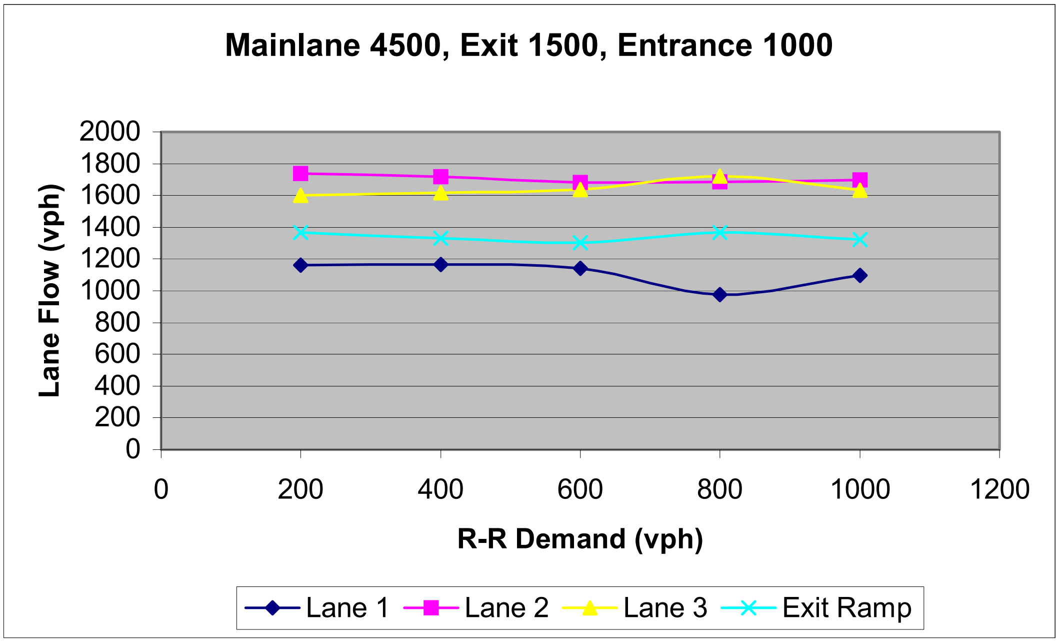

Figure B23 R-R Demand vs Lane Flow For Scenario 8

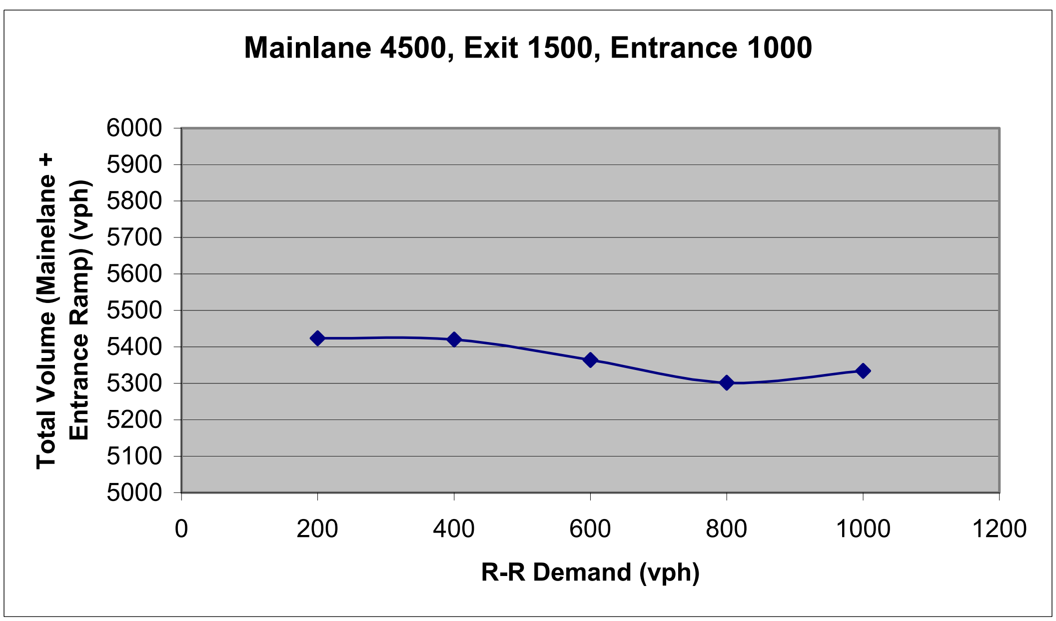

Figure B24 R-R Demand vs Total Volume (V) For Scenario 8

Scenarios 9:

- Total mainlane volume is 4500 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
4500	2000	1000	200	4318	1740	958	204	5276	2986	0.566
4500	2000	1000	400	4250	1748	962	393	5212	3288	0.631
4500	2000	1000	600	4232	1724	962	574	5194	3656	0.704
4500	2000	1000	800	4193	1773	971	799	5164	4018	0.778
4500	2000	1000	1000	4171	1735	967	887	5138	4210	0.819

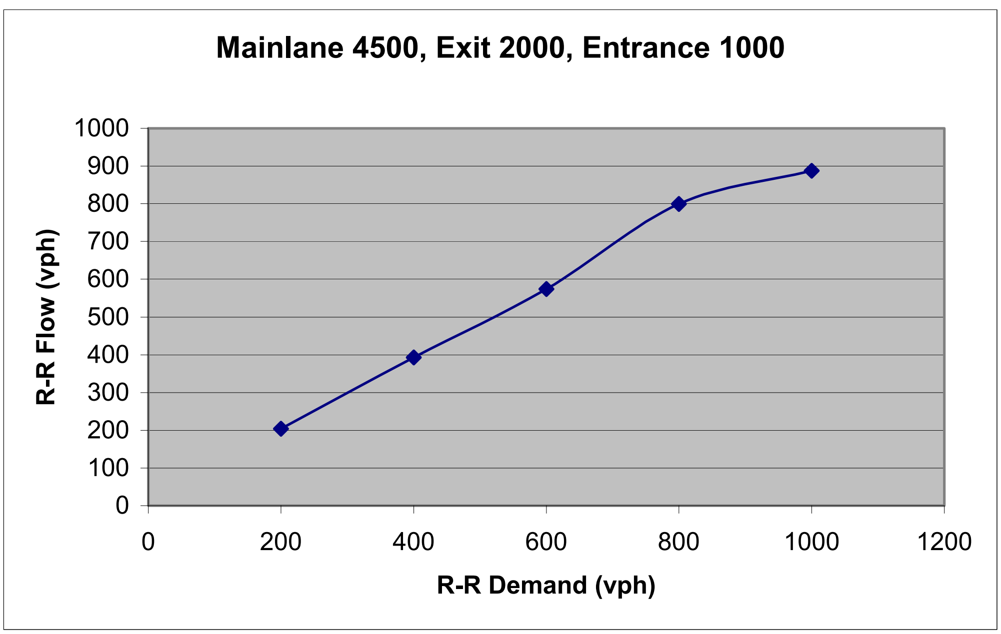

Figure B25 R-R Demand vs R-R Flow For Scenario 9

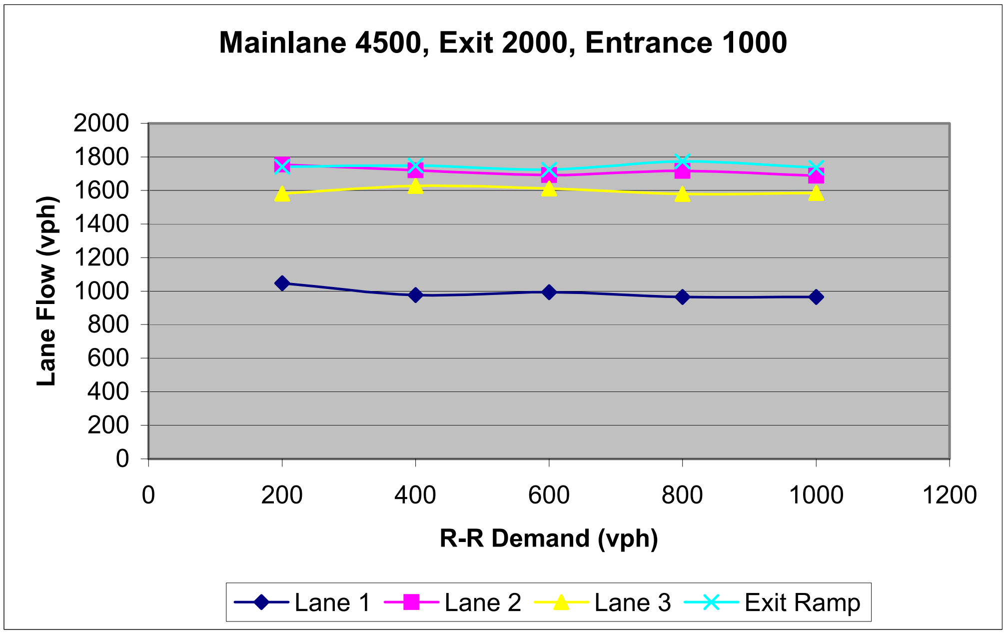

Figure B26 R-R Demand vs Lane Flow For Scenario 9

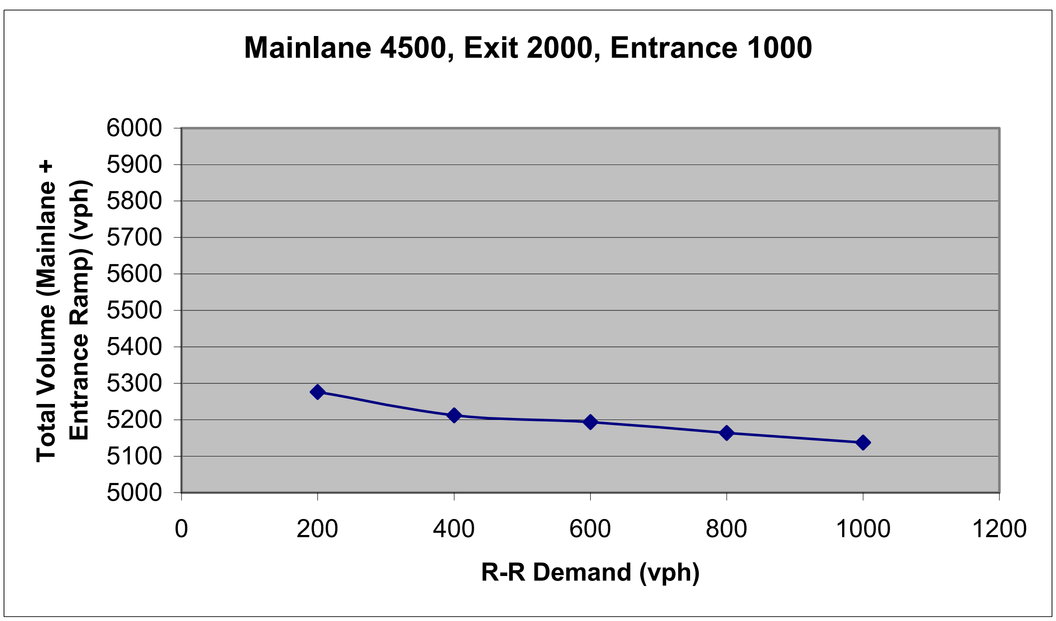

Figure B27 R-R Demand vs Total Volume (V) For Scenario 9

Scenarios 10:

- Total mainlane volume is 4800 vph
- Exit ramp volume is 1500 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
4800	1500	1000	200	4535	1306	981	196	5516	3621	0.657
4800	1500	1000	400	4502	1313	973	384	5475	3957	0.723
4800	1500	1000	600	4405	1315	974	594	5379	4278	0.795
4800	1500	1000	800	4340	1298	960	772	5300	4586	0.865
4800	1500	1000	1000	4300	1301	973	895	5273	4789	0.908

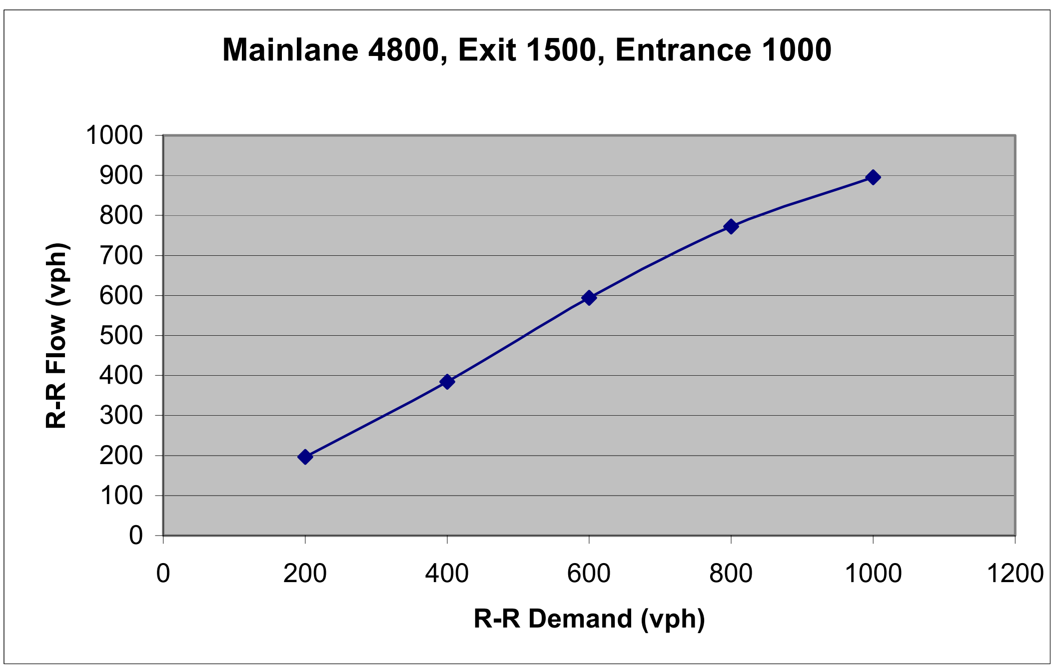

Figure B28 R-R Demand vs R-R Flow For Scenario 10

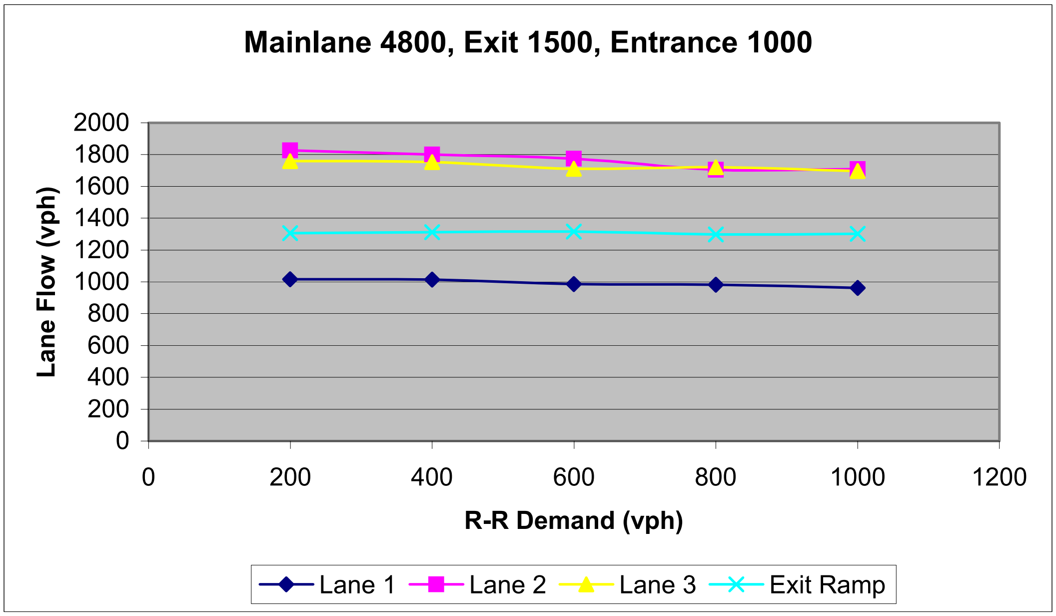

Figure B29 R-R Demand vs Lane Flow For Scenario 10

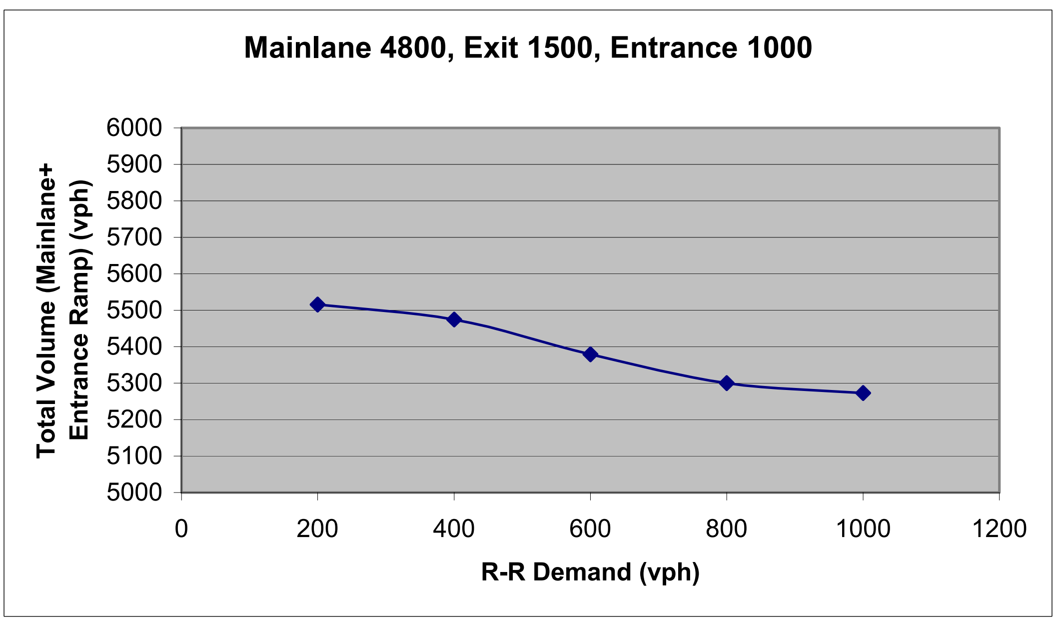

Figure B30 R-R Demand vs Total Volume (V) For Scenario 10

Scenarios 11:

- Total mainlane volume is 4800 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
4800	2000	1000	200	4374	1693	969	190	5343	3061	0.573
4800	2000	1000	400	4300	1688	966	382	5266	3376	0.641
4800	2000	1000	600	4263	1702	973	586	5236	3732	0.713
4800	2000	1000	800	4248	1693	971	784	5219	4124	0.790
4800	2000	1000	1000	4181	1700	975	903	5156	4287	0.832

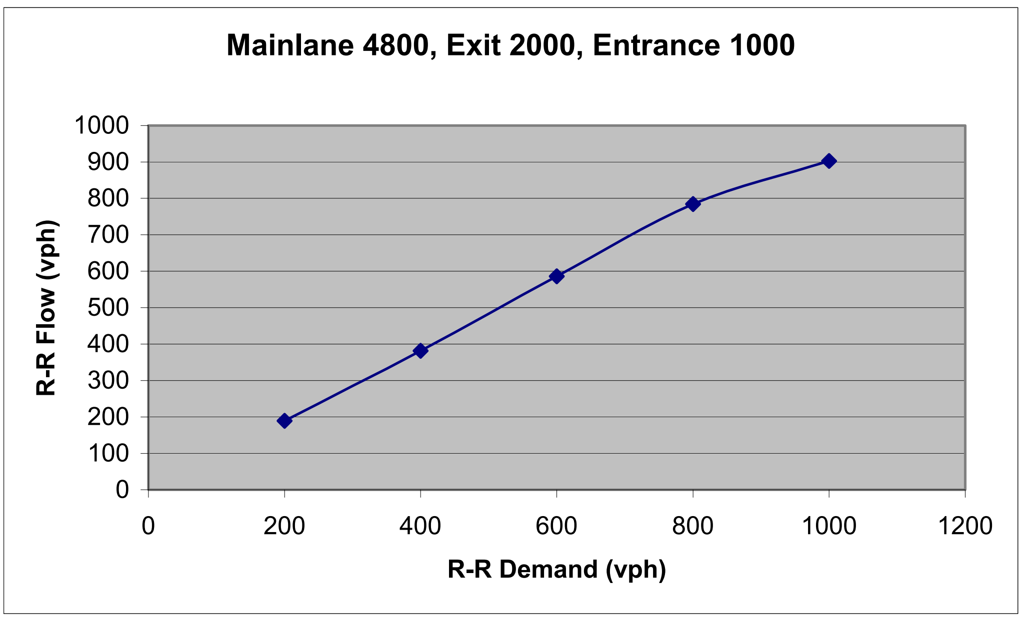

Figure B31 R-R Demand vs R-R Flow For Scenario 11

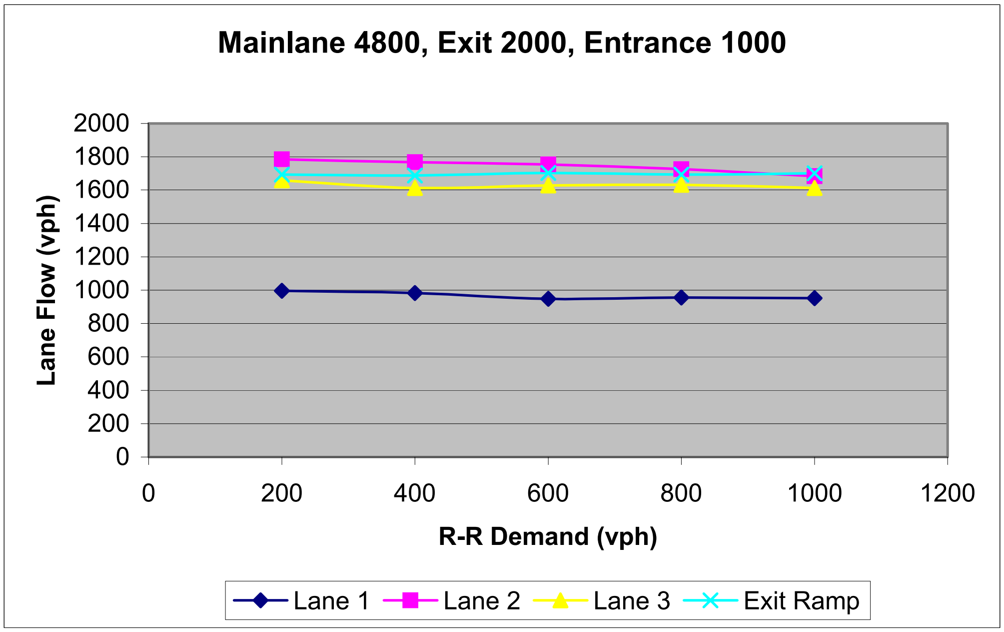

Figure B32 R-R Demand vs Lane Flow For Scenario 11

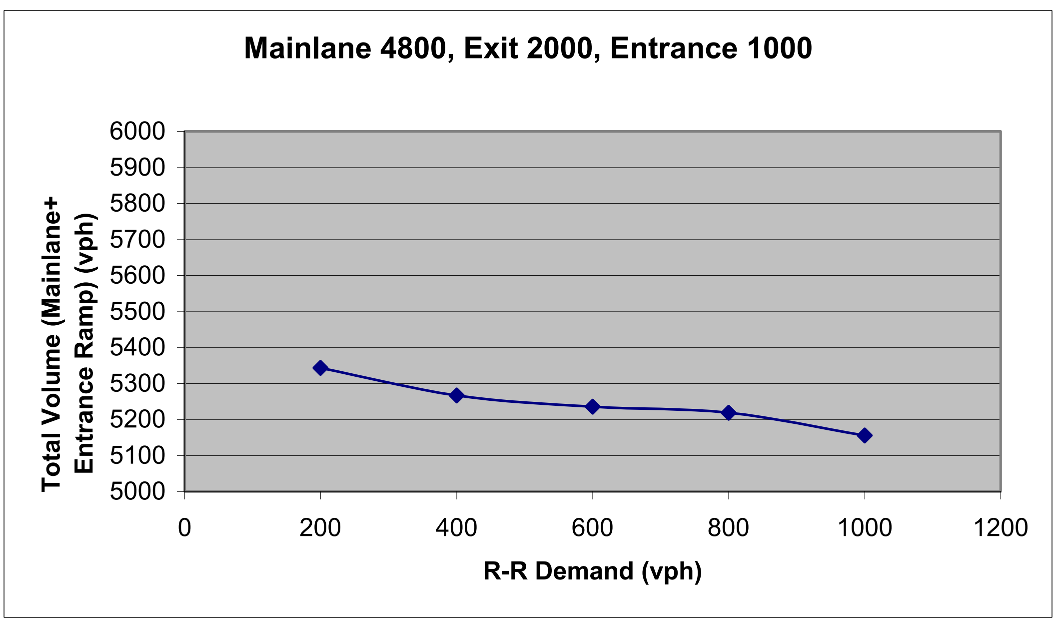

Figure B33 R-R Demand vs Total Volume (V) For Scenario 11

Scenarios 12:

- Total mainlane volume is 5000 vph
- Exit ramp volume is 800 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5000	800	1000	200	4754	701	979	192	5733	4437	0.774
5000	800	1000	400	4681	706	974	399	5655	4774	0.844
5000	800	1000	600	4584	717	974	599	5558	5066	0.912
5000	800	1000	800	4551	732	973	732	5524	5283	0.956

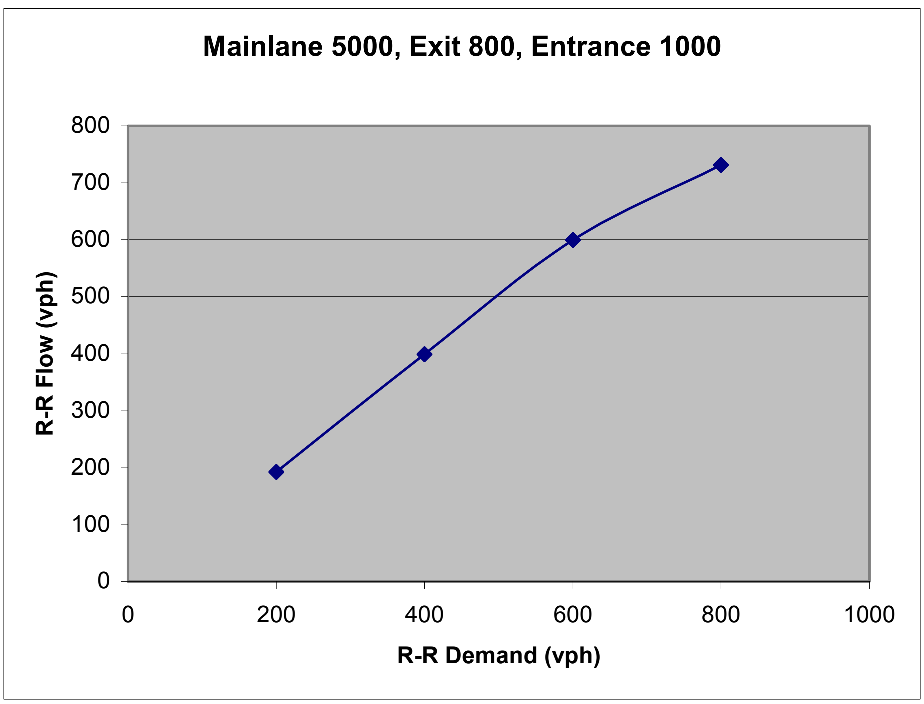

Figure B34 R-R Demand vs R-R Flow For Scenario 12

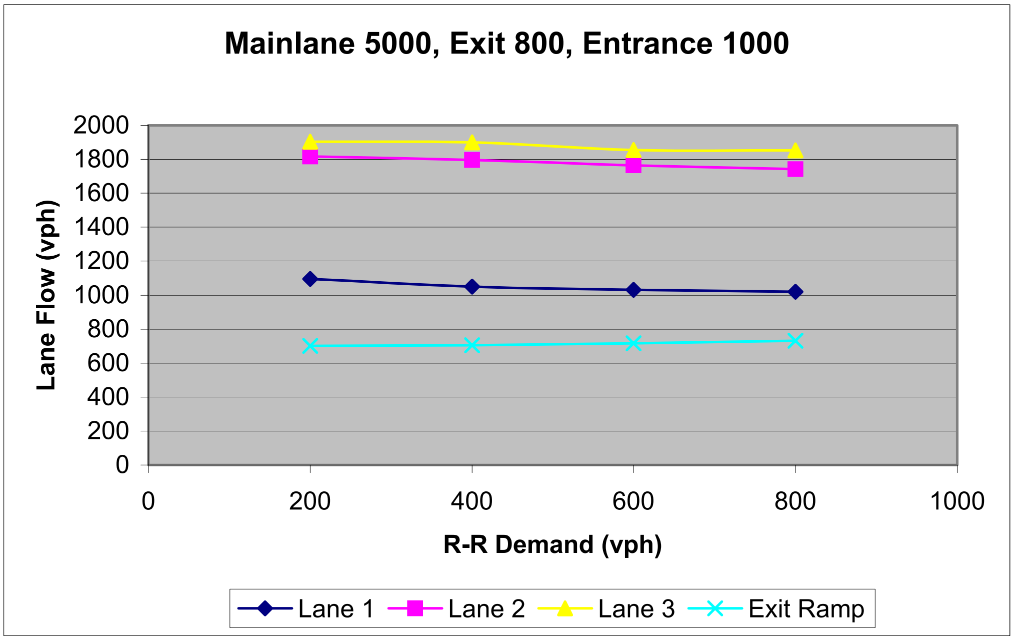

Figure B35R-R Demand vs Lane Flow For Scenario 12

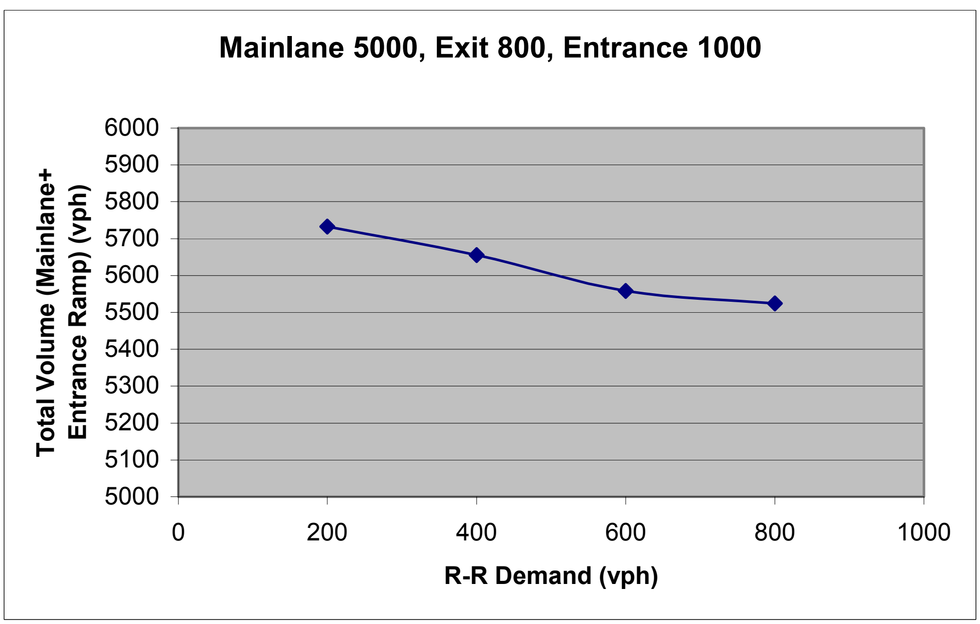

Figure B36 R-R Demand vs Total Volume (V) For Scenario 12

Scenarios 13:

- Total mainlane volume is 5000 vph
- Exit ramp volume is 1000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5000	1000	1000	200	4679	855	971	196	5651	4217	0.746
5000	1000	1000	400	4664	871	970	380	5634	4554	0.808
5000	1000	1000	600	4556	857	972	586	5528	4871	0.881
5000	1000	1000	800	4480	880	969	776	5448	5153	0.946
5000	1000	1000	1000	4402	886	966	886	5368	5287	0.985

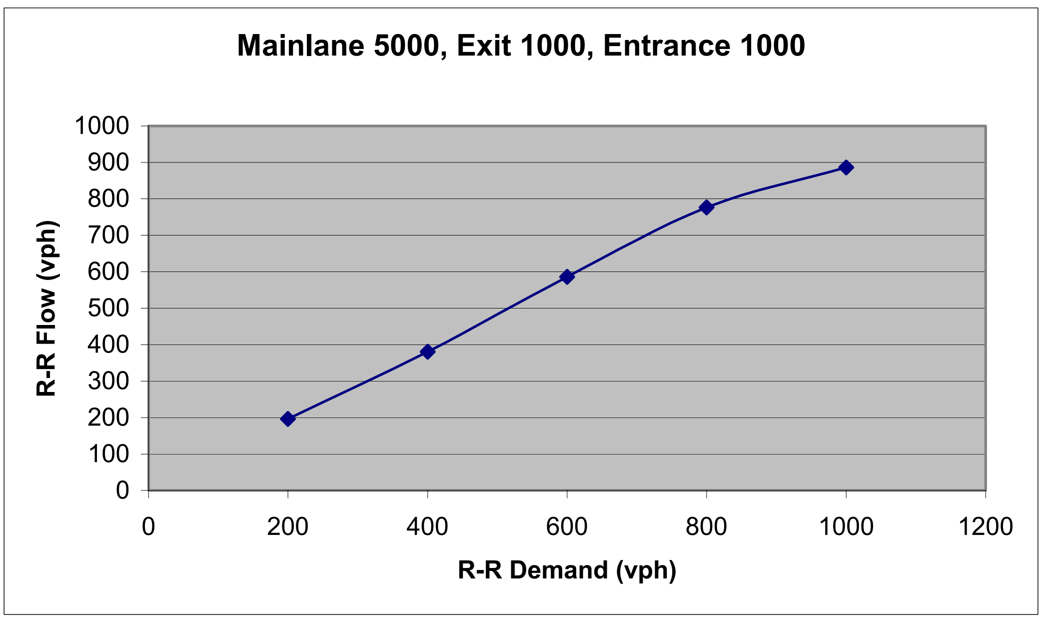

Figure B37 R-R Demand vs R-R Flow For Scenario 13

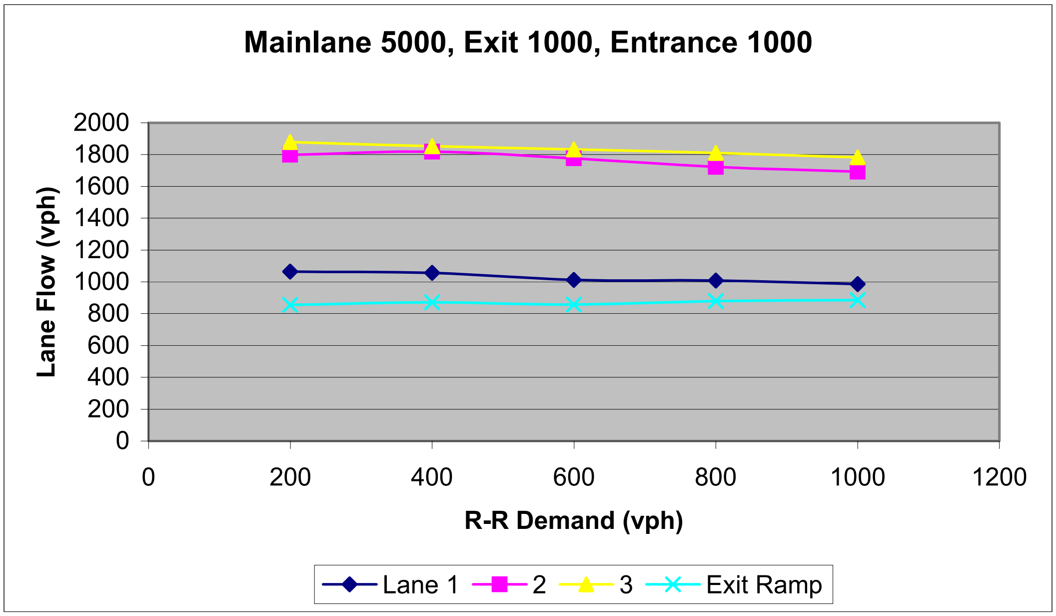

Figure B38 R-R Demand vs Lane Flow For Scenario 13

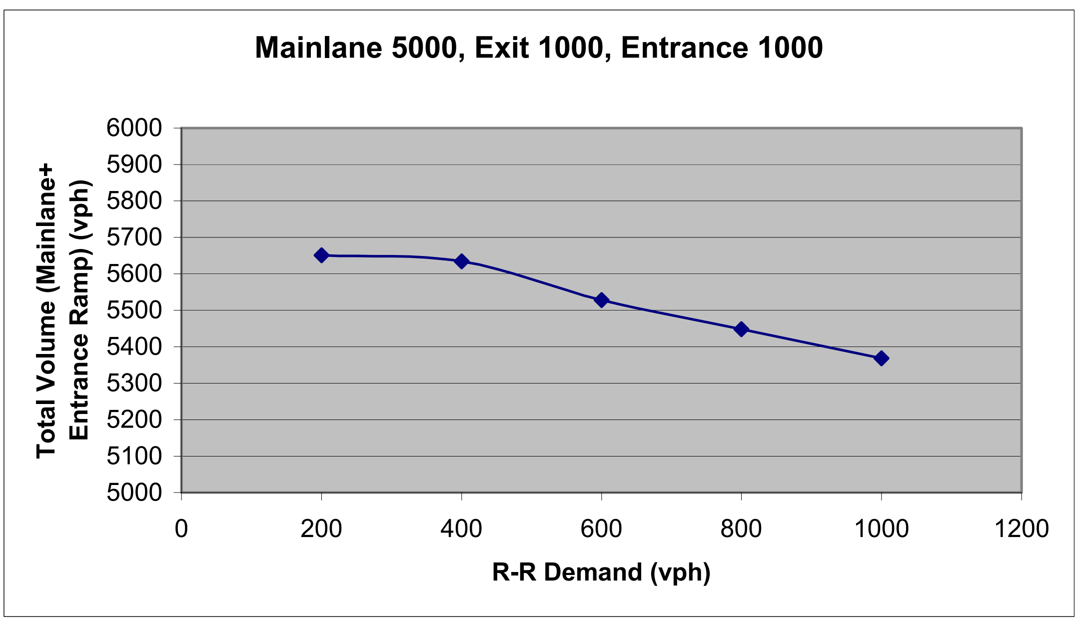

Figure B39 R-R Demand vs Total Volume (V) For Scenario 13

Scenarios 14:

- Total mainlane volume is 5000 vph
- Exit ramp volume is 1500 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5000	1500	1000	200	4558	1254	983	197	5542	3698	0.667
5000	1500	1000	400	4492	1266	972	389	5463	4003	0.733
5000	1500	1000	600	4482	1250	966	585	5448	4403	0.808
5000	1500	1000	800	4397	1265	979	776	5376	4683	0.871
5000	1500	1000	1000	4298	1307	966	899	5264	4789	0.910

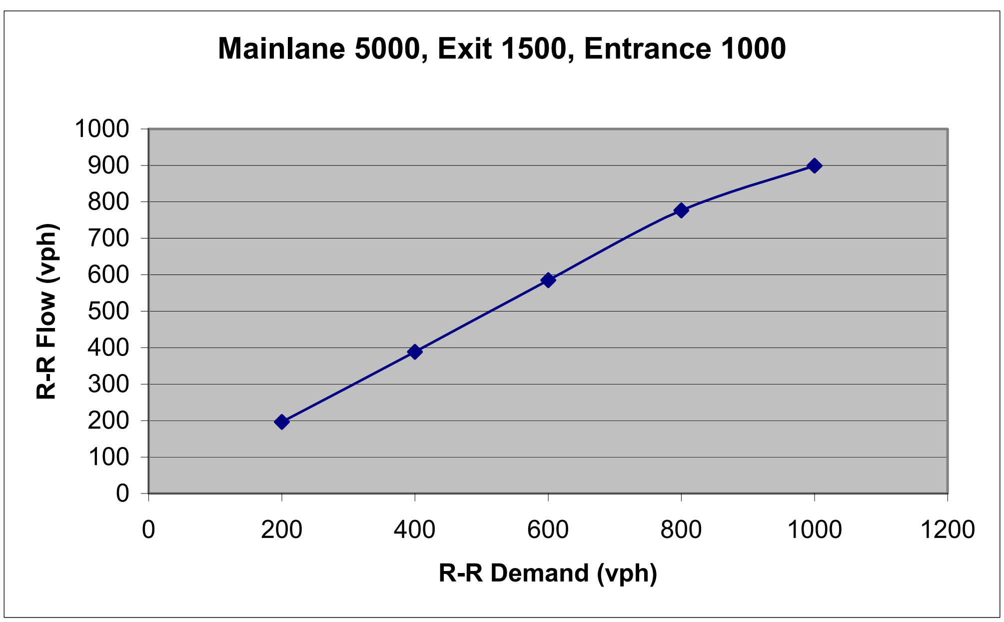

Figure B40 R-R Demand vs R-R Flow For Scenario 14

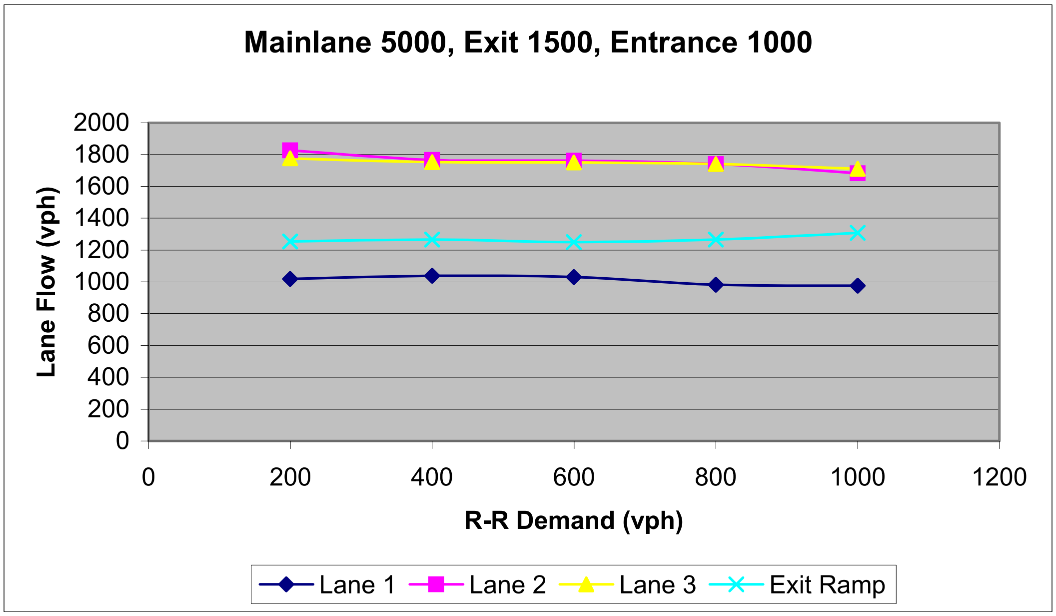

Figure B41 R-R Demand vs Lane Flow For Scenario 14

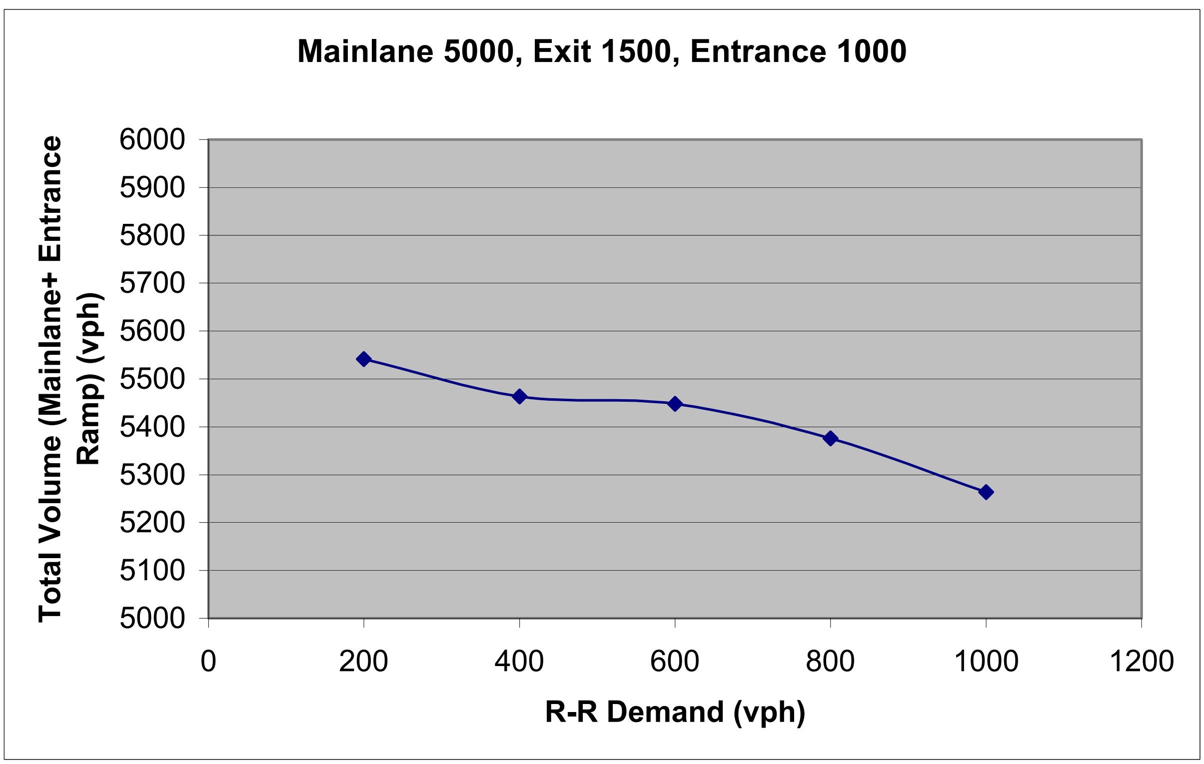

Figure B42 R-R Demand vs Total Volume (V) For Scenario 14

Scenarios 15:

- Total mainlane volume is 5500 vph
- Exit ramp volume is 1000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5500	1000	1000	200	4698	804	982	192	5680	4279	0.753
5500	1000	1000	400	4647	815	977	384	5624	4600	0.818
5500	1000	1000	600	4545	848	957	574	5503	4846	0.881
5500	1000	1000	800	4473	877	971	778	5444	5151	0.946
5500	1000	1000	1000	4427	898	972	898	5399	5325	0.986

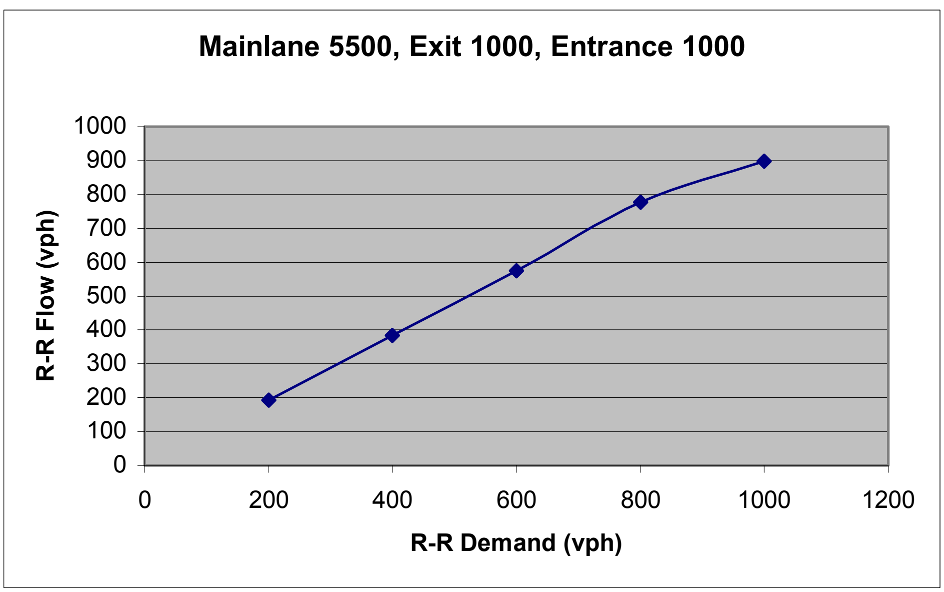

Figure B43 R-R Demand vs R-R Flow For Scenario 15

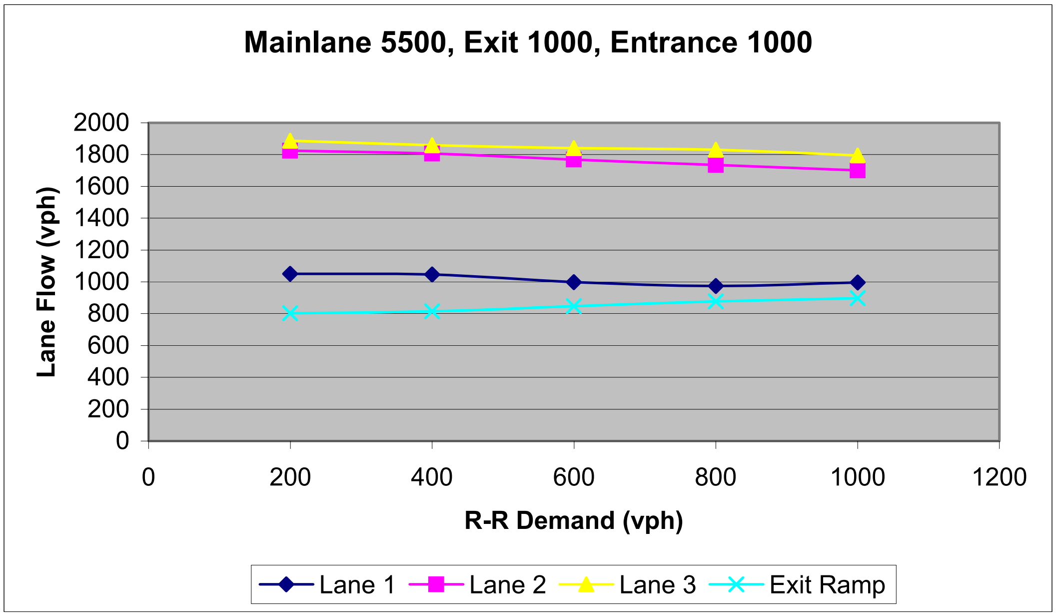

Figure B44 R-R Demand vs Lane Flow For Scenario 15

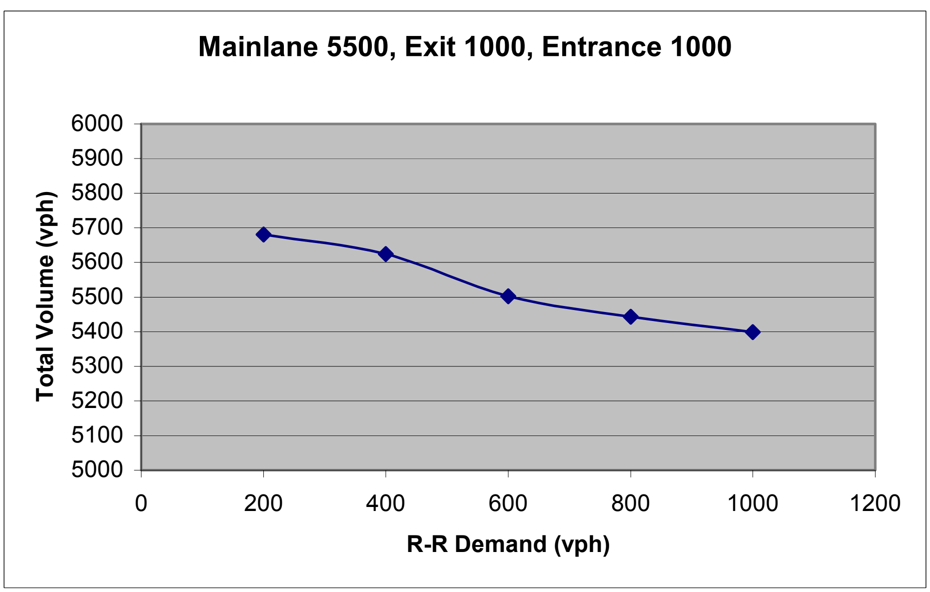

Figure B45 R-R Demand vs Total Volume (V) For Scenario 15

Scenarios 16:

- Total mainlane volume is 5500 vph
- Exit ramp volume is 1200 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5500	1200	1000	200	4650	942	981	191	5631	4091	0.726
5500	1200	1000	400	4570	977	978	394	5548	4381	0.790
5500	1200	1000	600	4501	1010	978	597	5479	4685	0.855
5500	1200	1000	800	4428	1036	974	781	5402	4955	0.917
5500	1200	1000	1000	4375	1047	977	906	5351	5140	0.961

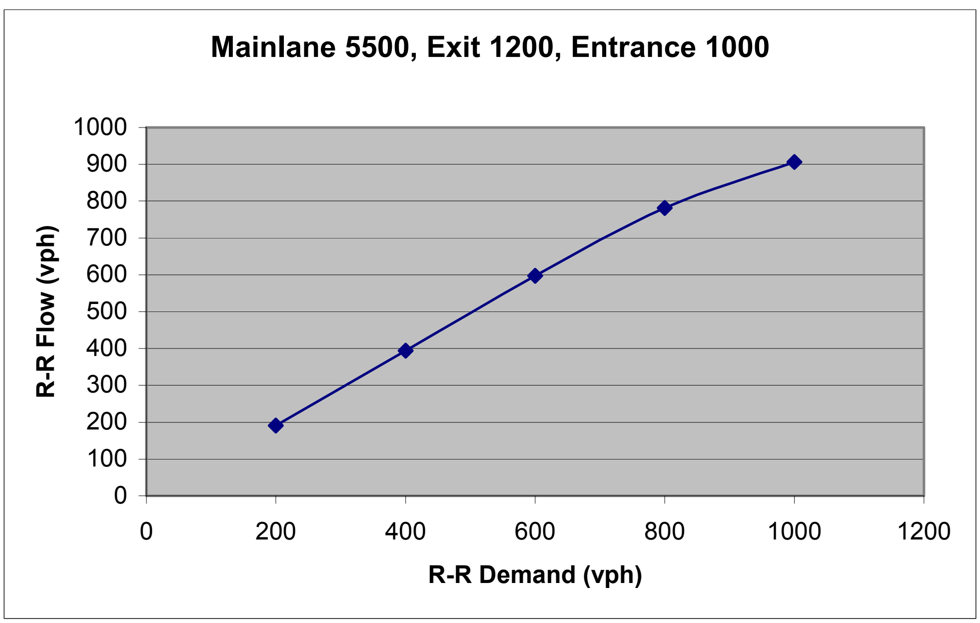

Figure B46 R-R Demand vs R-R Flow For Scenario 16

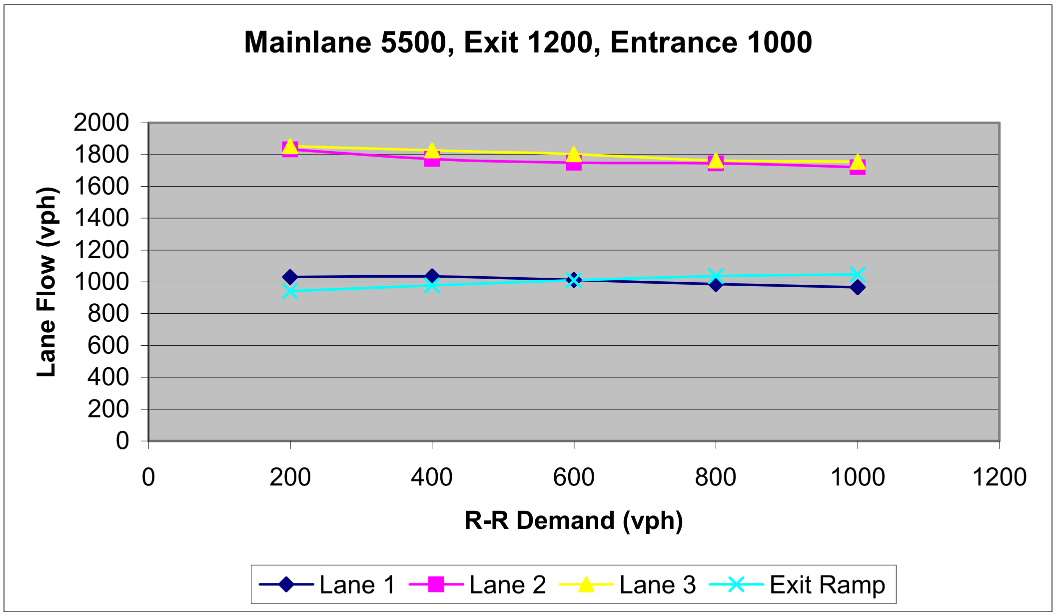

Figure B47 R-R Demand vs Lane Flow For Scenario 16

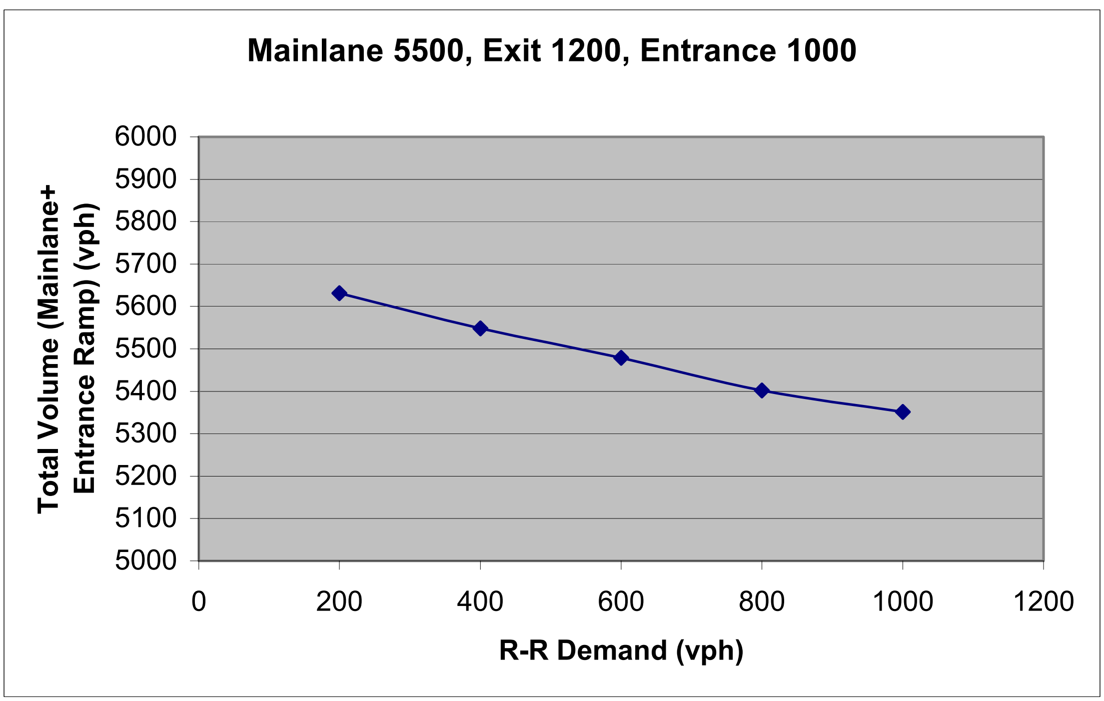

Figure B48 R-R Demand vs Total Volume (V) For Scenario 16

Scenarios 17:

- Total mainlane volume is 5500 vph
- Exit ramp volume is 1500 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5500	1500	1000	200	4566	1190	987	199	5553	3774	0.680
5500	1500	1000	400	4515	1201	968	382	5483	4077	0.744
5500	1500	1000	600	4471	1204	965	571	5437	4409	0.811
5500	1500	1000	800	4384	1245	977	787	5361	4713	0.879
5500	1500	1000	1000	4297	1261	978	912	5274	4860	0.921

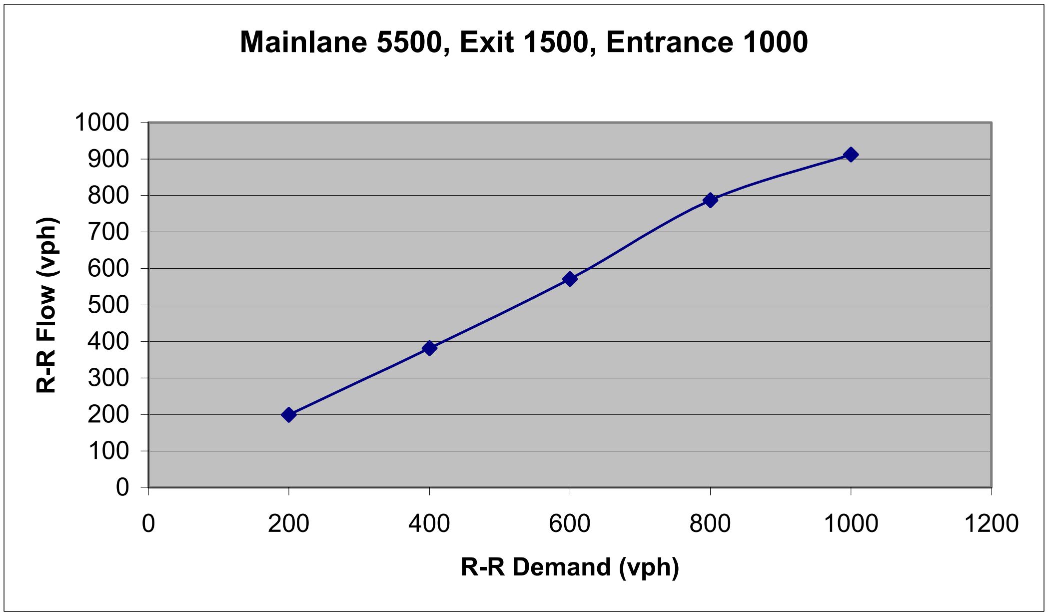

Figure B49 R-R Demand vs R-R Flow For Scenario 17

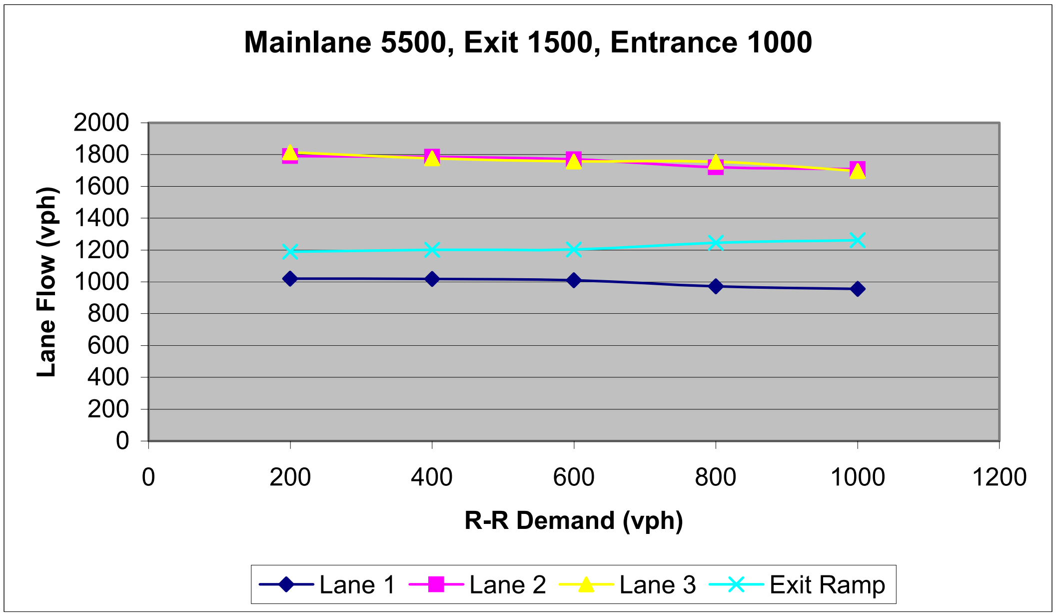

Figure B50 R-R Demand vs Lane Flow For Scenario 17

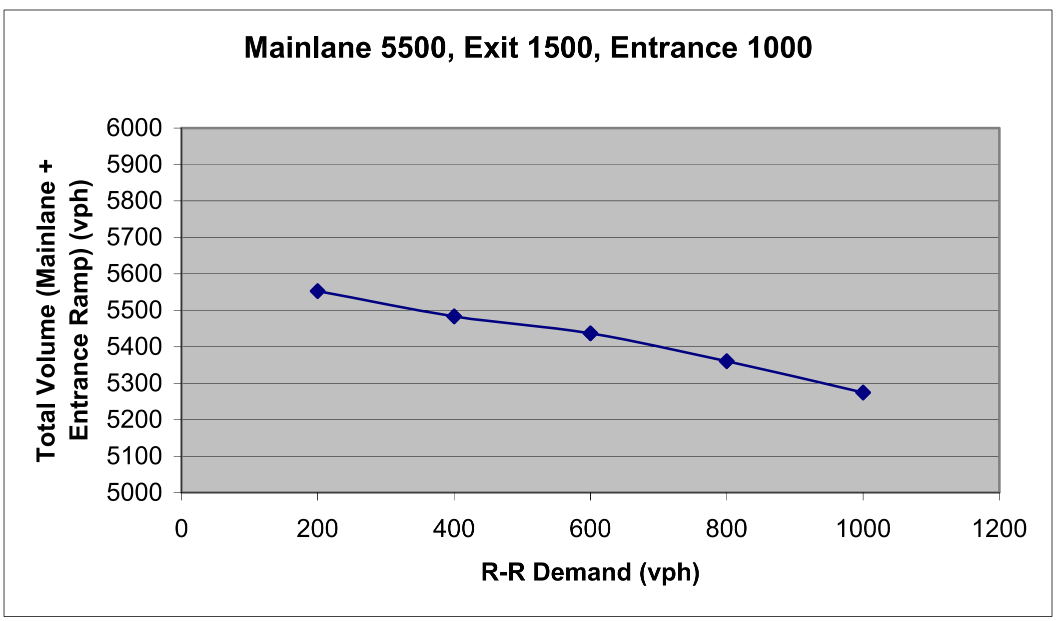

Figure B51 R-R Demand vs Total Volume (V) For Scenario 17

Scenarios 18:

- Total mainlane volume is 5500 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5500	2000	1000	200	4424	1525	995	193	5419	3285	0.606
5500	2000	1000	400	4343	1543	977	405	5320	3611	0.679
5500	2000	1000	600	4334	1569	967	573	5302	3911	0.738
5500	2000	1000	800	4261	1595	985	787	5246	4241	0.808
5500	2000	1000	1000	4242	1597	969	885	5211	4416	0.847

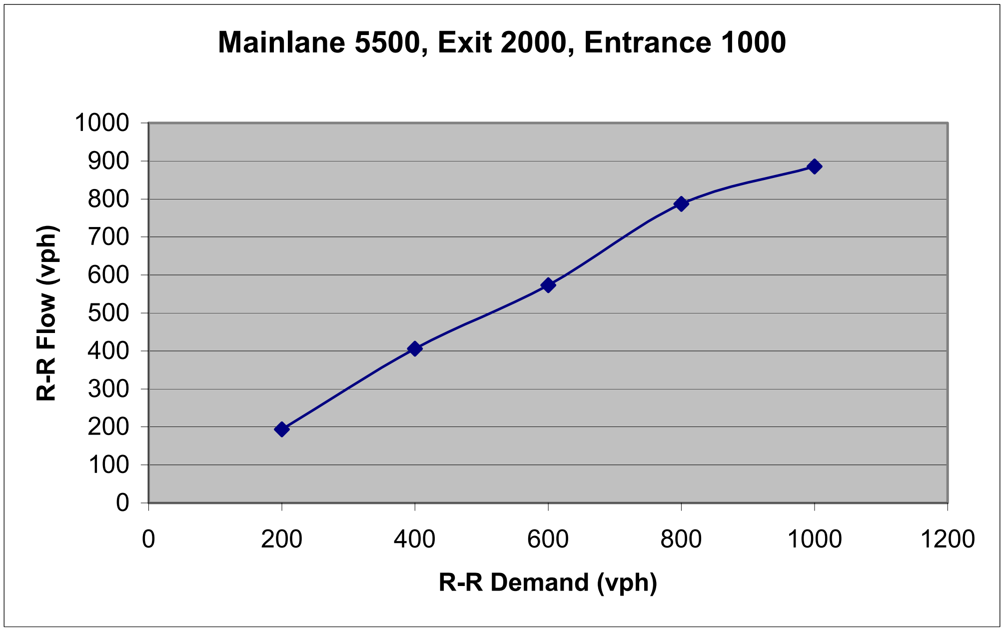

Figure B52 R-R Demand vs R-R Flow For Scenario 18

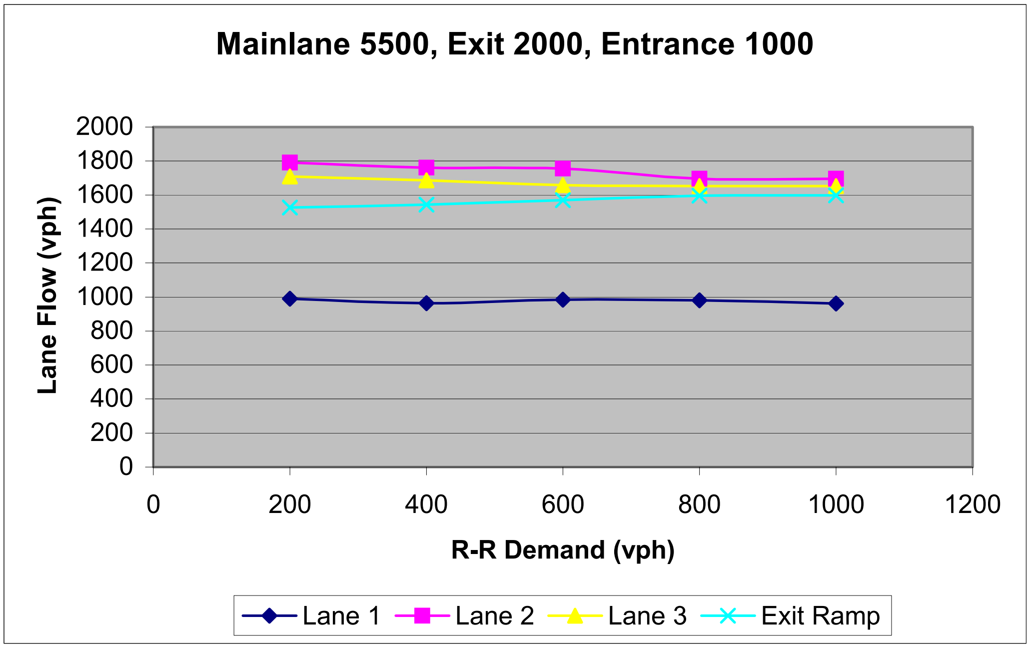

Figure B53 R-R Demand vs Lane Flow For Scenario 18

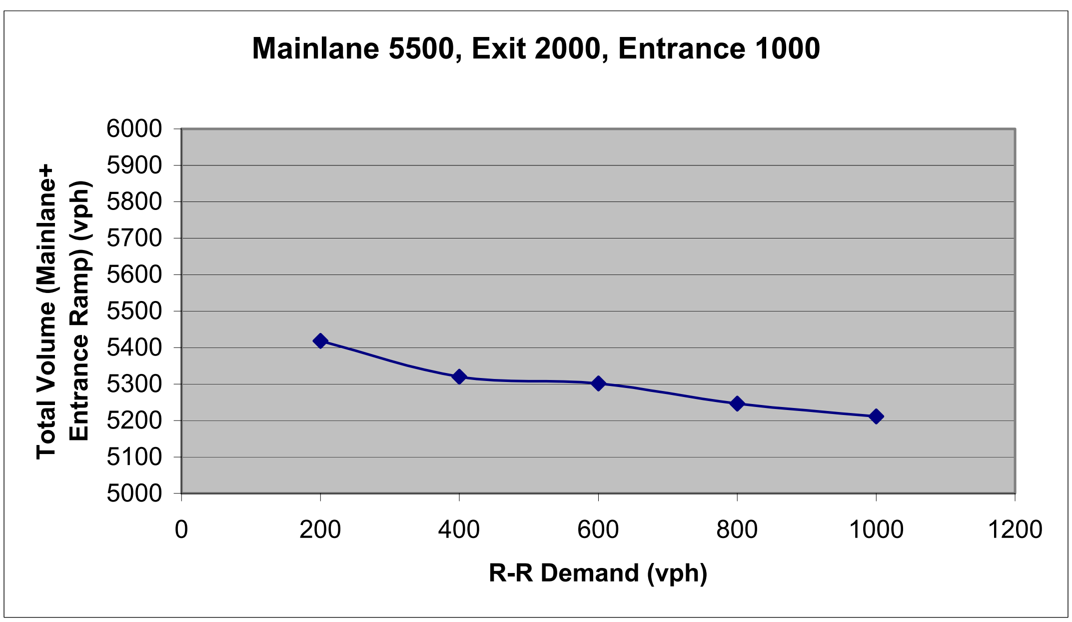

Figure B54 R-R Demand vs Total Volume (V) For Scenario 18

Scenarios 19:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 800 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	800	1000	200	4807	630	977	196	5784	4569	0.790
6000	800	1000	400	4712	664	980	396	5692	4840	0.850
6000	800	1000	600	4585	695	969	587	5554	5065	0.912
6000	800	1000	800	4526	726	981	726	5507	5252	0.954

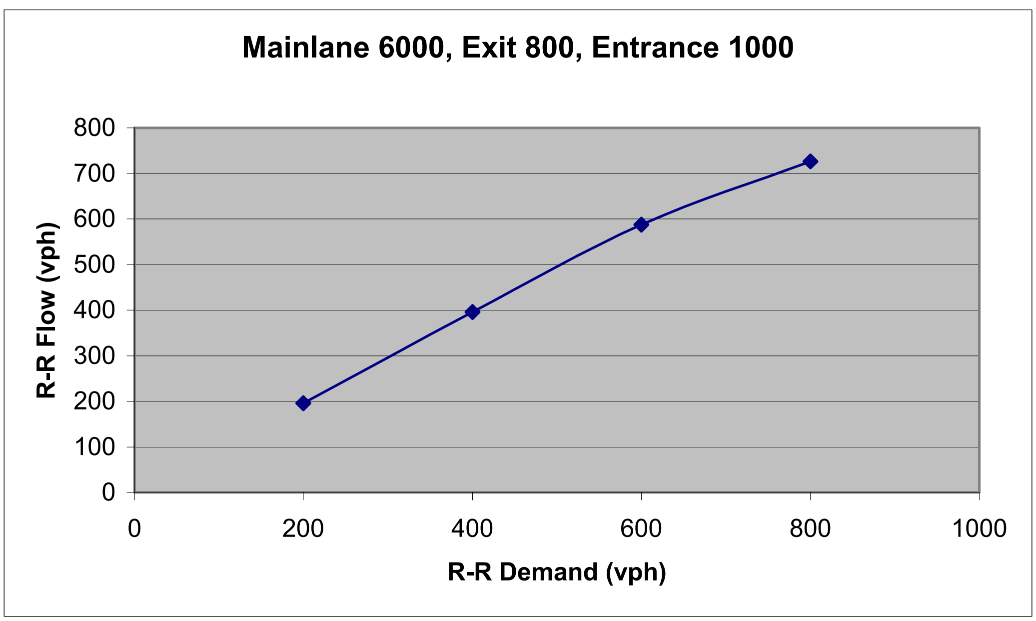

Figure B55 R-R Demand vs R-R Flow For Scenario 19

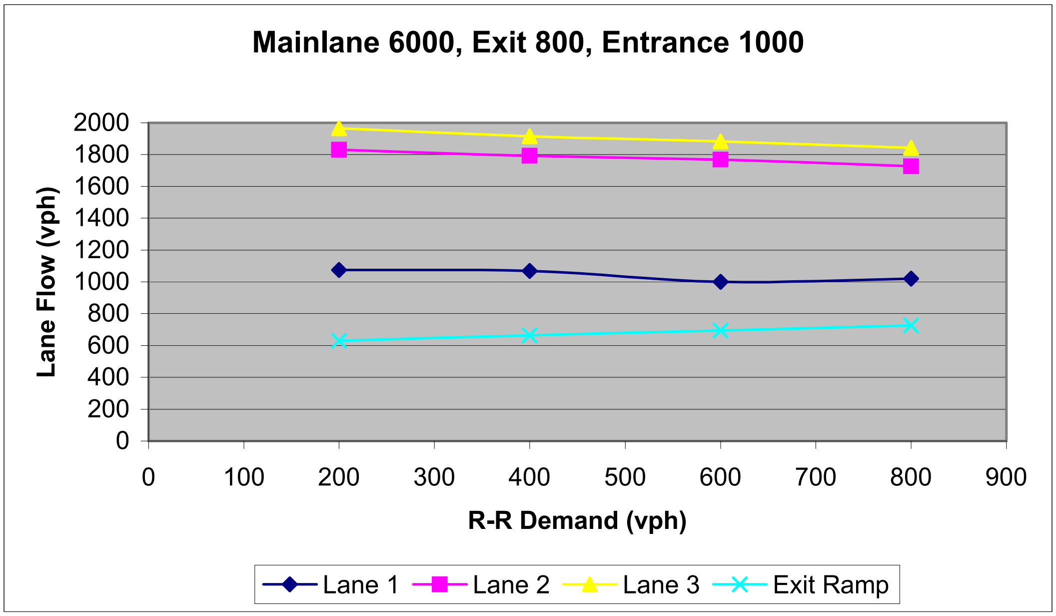

Figure B56 R-R Demand vs Lane Flow For Scenario 19

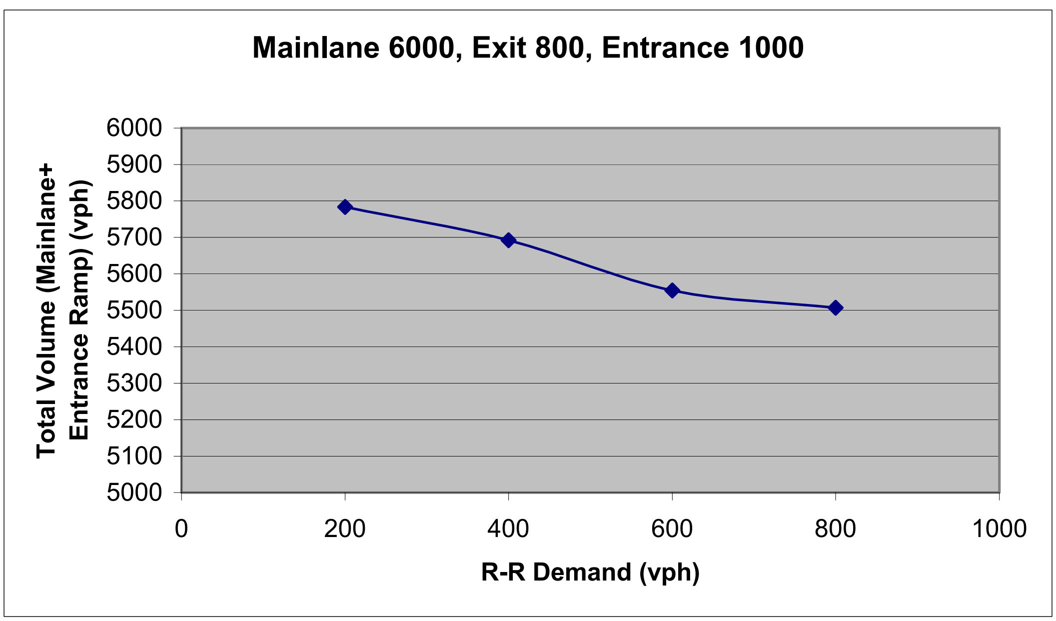

Figure B57 R-R Demand vs Total Volume (V) For Scenario 19

Scenarios 20:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 1000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	1000	1000	100	4799	737	984	84	5783	4229	0.731
6000	1000	1000	200	4770	771	969	186	5739	4372	0.762
6000	1000	1000	300	4702	770	979	270	5682	4473	0.787
6000	1000	1000	400	4657	799	981	369	5637	4596	0.815
6000	1000	1000	500	4610	815	985	451	5595	4697	0.839
6000	1000	1000	600	4547	831	974	560	5521	4837	0.876
6000	1000	1000	700	4542	861	976	647	5518	4974	0.901
6000	1000	1000	800	4475	881	973	733	5449	5060	0.929
6000	1000	1000	900	4459	885	975	816	5434	5205	0.958
6000	1000	1000	1000	4398	897	975	897	5373	5295	0.985

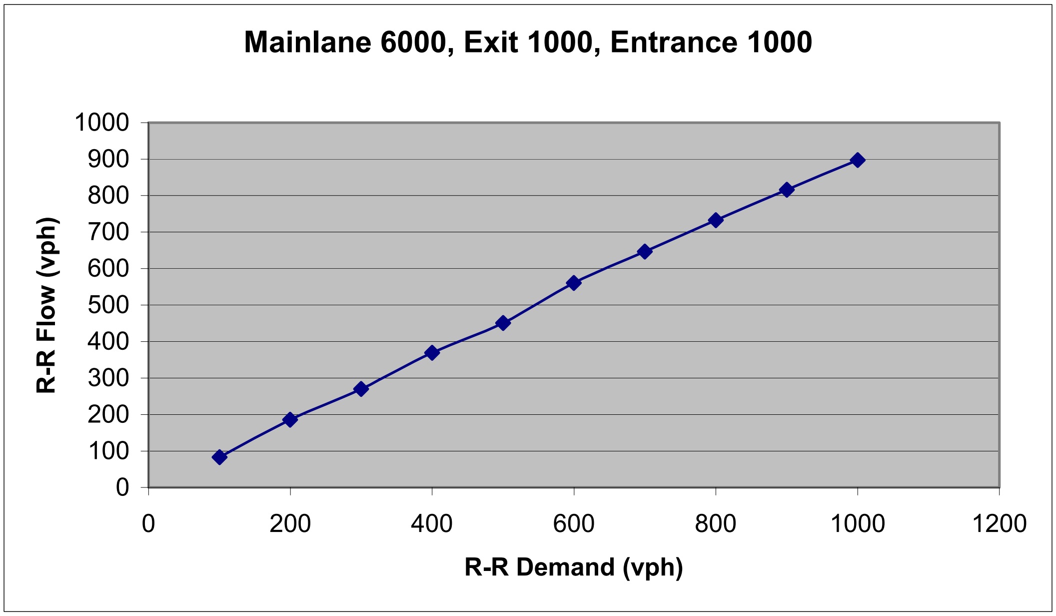

Figure B58 R-R Demand vs R-R Flow For Scenario 20

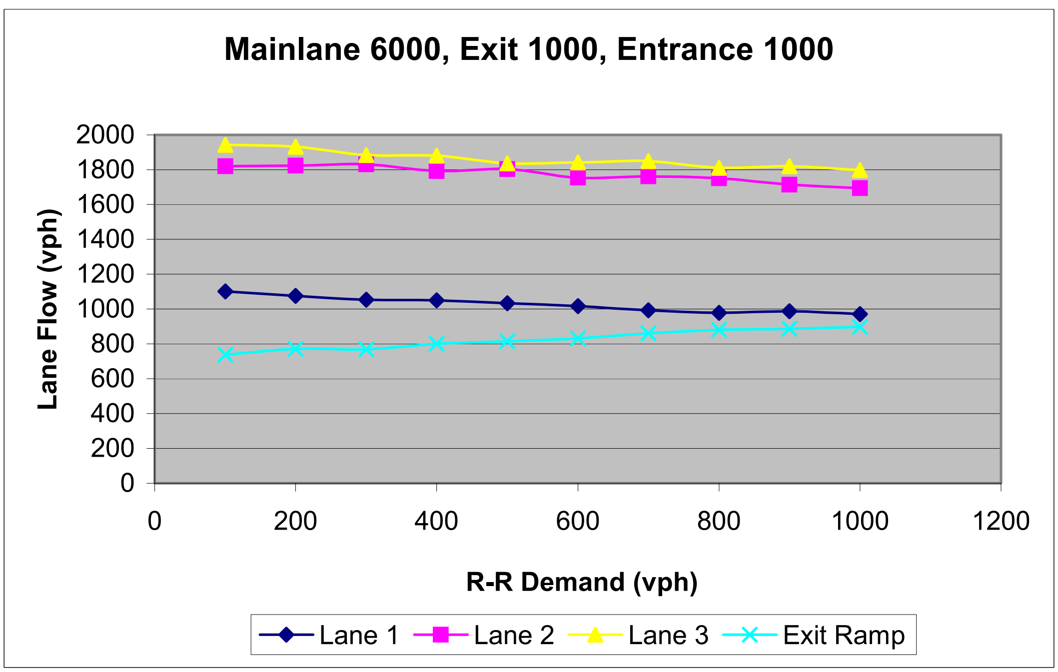

Figure B59 R-R Demand vs Lane Flow For Scenario 20

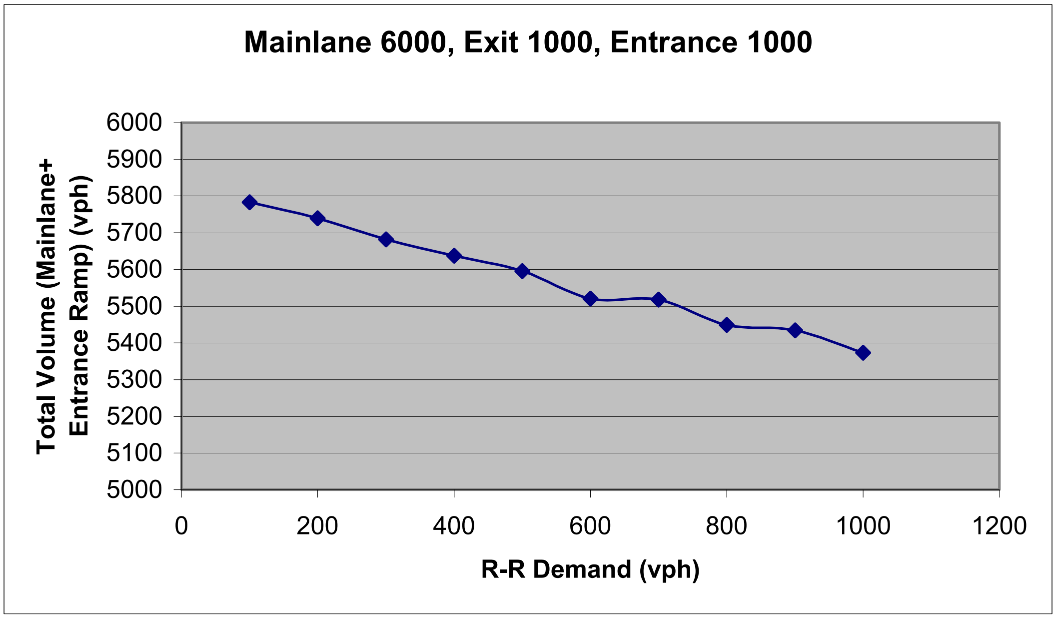

Figure B60 R-R Demand vs Total Volume (V) For Scenario 20

Scenarios 21:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 1200 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	1200	1000	200	4693	897	976	201	5669	4198	0.740
6000	1200	1000	400	4609	940	978	400	5587	4470	0.800
6000	1200	1000	600	4537	953	968	579	5505	4742	0.861
6000	1200	1000	800	4444	1003	978	781	5422	5002	0.923
6000	1200	1000	1000	4382	1044	976	908	5359	5155	0.962

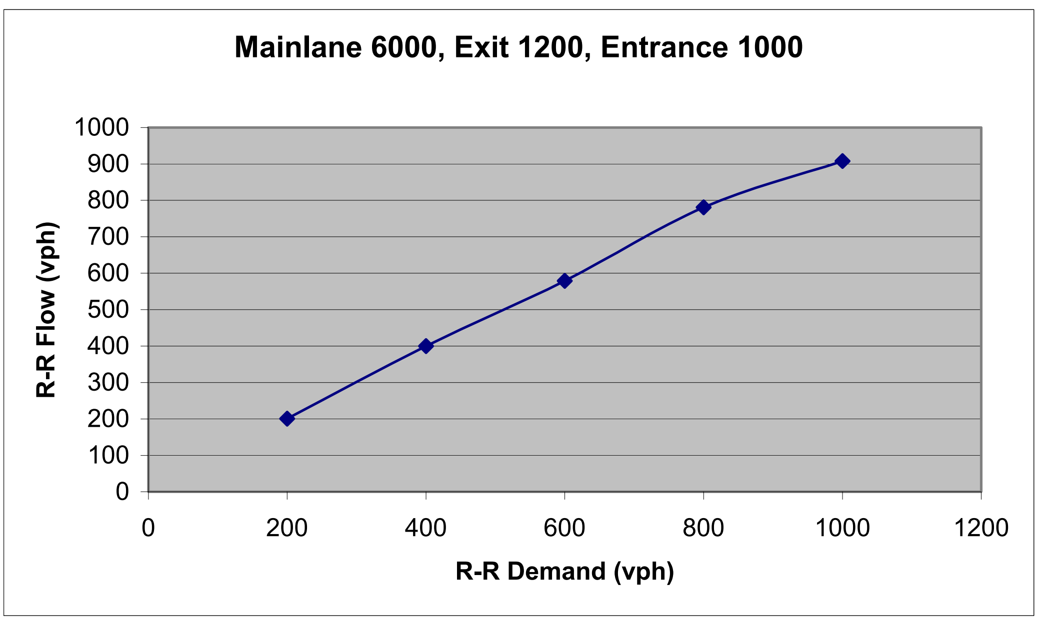

Figure B61 R-R Demand vs R-R Flow For Scenario 21

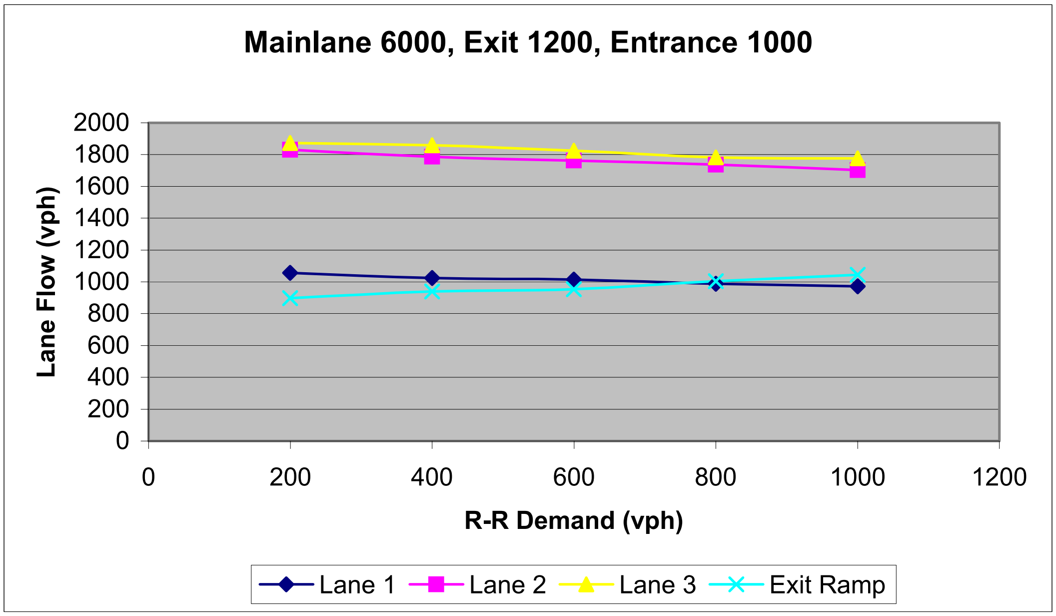

Figure B62 R-R Demand vs Lane Flow For Scenario 21

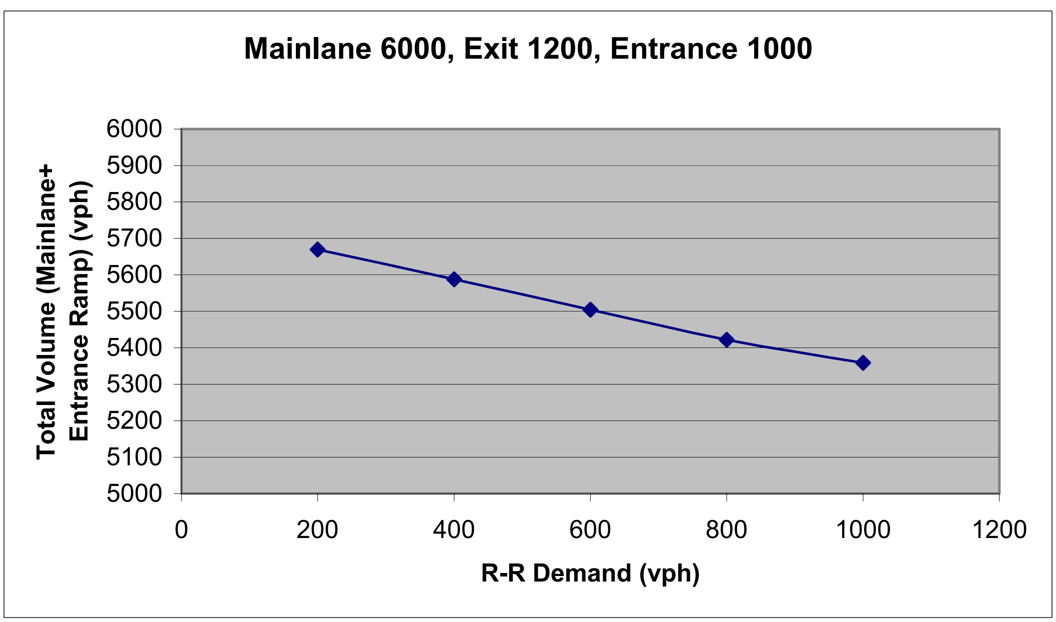

Figure B63 R-R Demand vs Total Volume (V) For Scenario 21

Scenarios 22:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 1500 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	1500	1000	200	4619	1107	971	190	5590	3893	0.696
6000	1500	1000	400	4510	1129	979	405	5489	4191	0.764
6000	1500	1000	600	4481	1165	972	584	5453	4483	0.822
6000	1500	1000	800	4382	1198	979	784	5361	4752	0.886
6000	1500	1000	1000	4318	1224	980	909	5298	4911	0.927

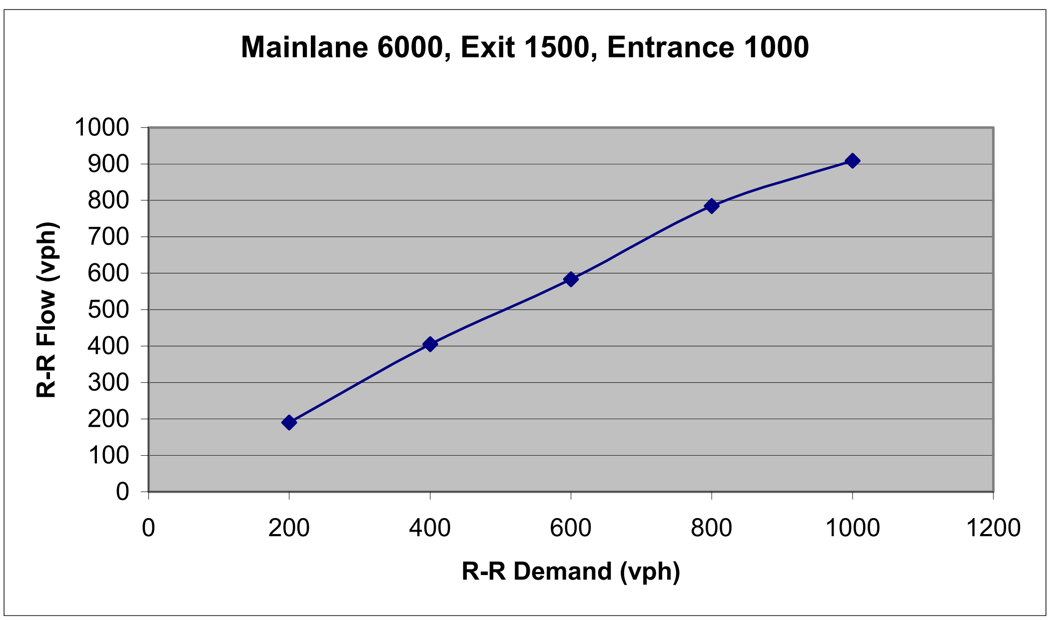

Figure B64 R-R Demand vs R-R Flow For Scenario 22

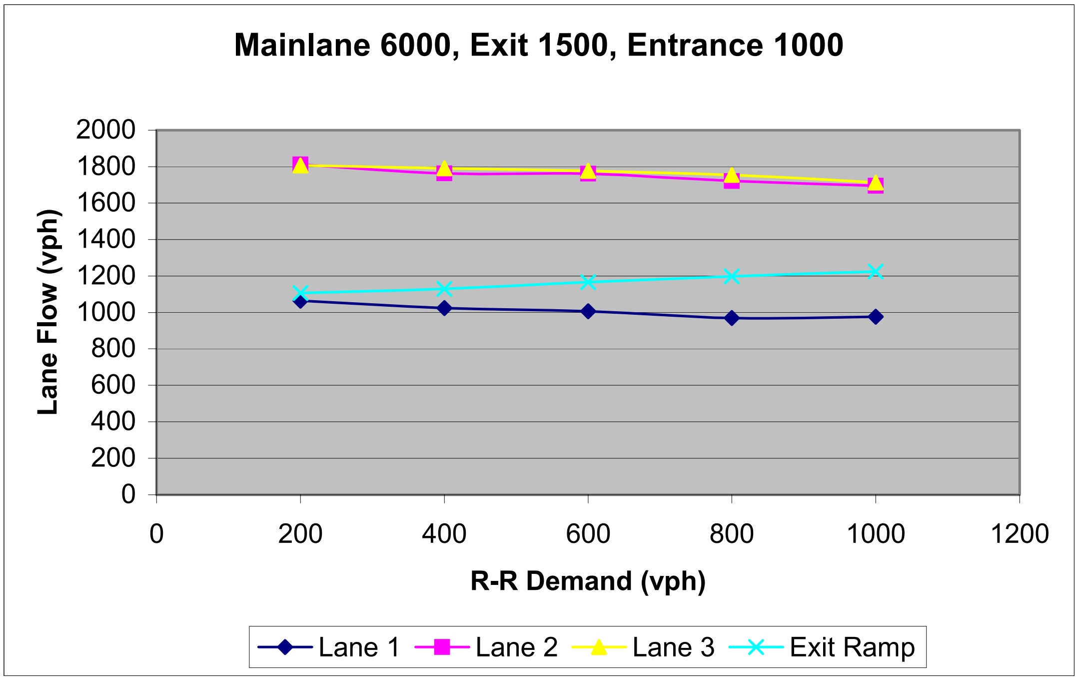

Figure B65 R-R Demand vs Lane Flow For Scenario 22

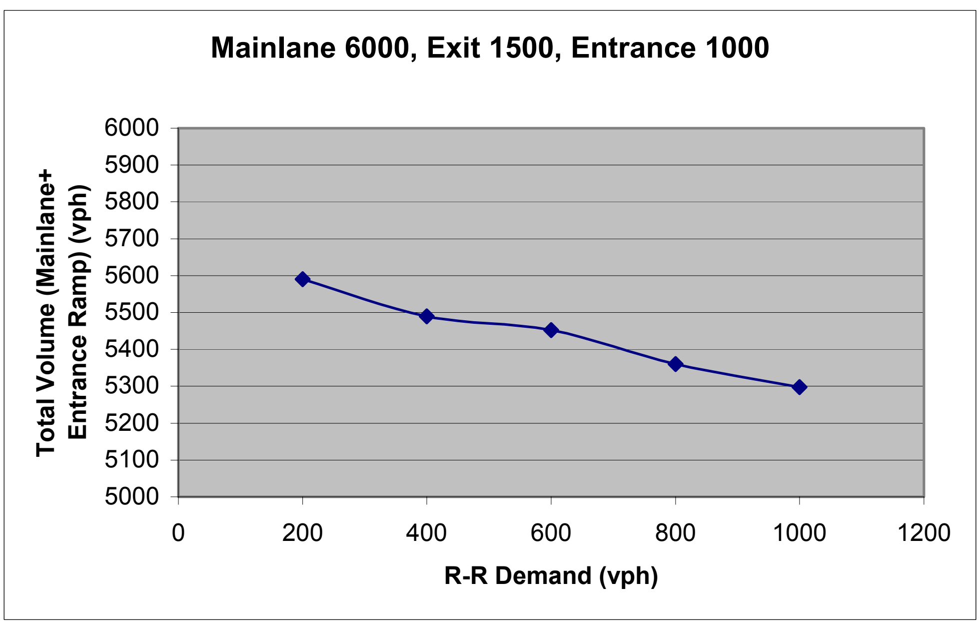

Figure B66 R-R Demand vs Total Volume (V) For Scenario 22

Scenarios 23:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 1750 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	1750	1000	200	4541	1253	973	185	5514	3659	0.663
6000	1750	1000	400	4494	1302	970	384	5463	3960	0.725
6000	1750	1000	600	4402	1325	974	576	5376	4230	0.787
6000	1750	1000	800	4335	1359	976	784	5312	4545	0.856
6000	1750	1000	1000	4270	1355	975	893	5245	4701	0.896

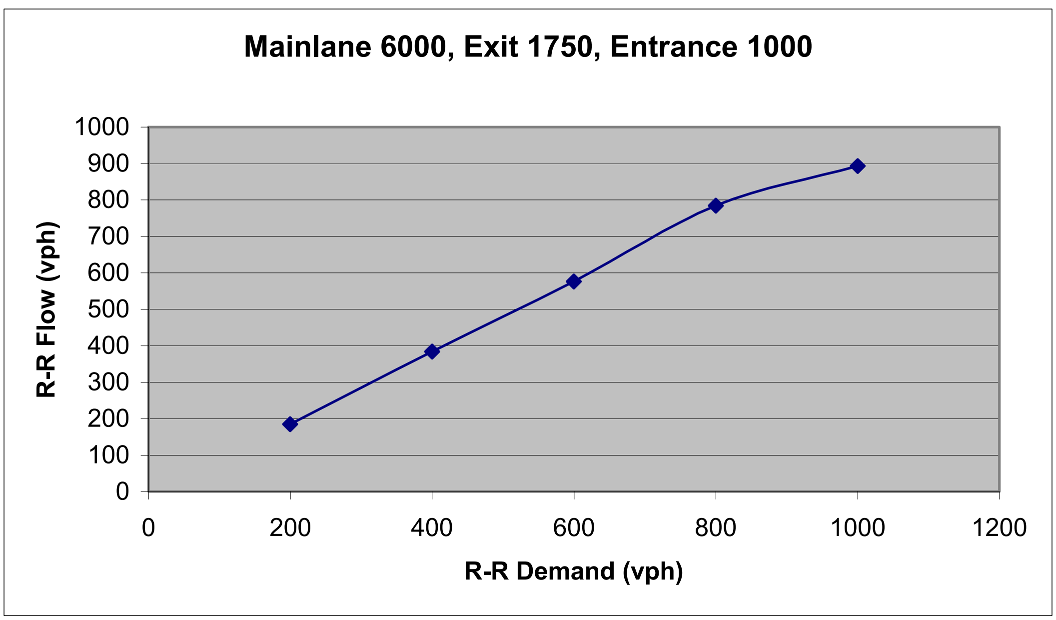

Figure B67 R-R Demand vs R-R Flow For Scenario 23

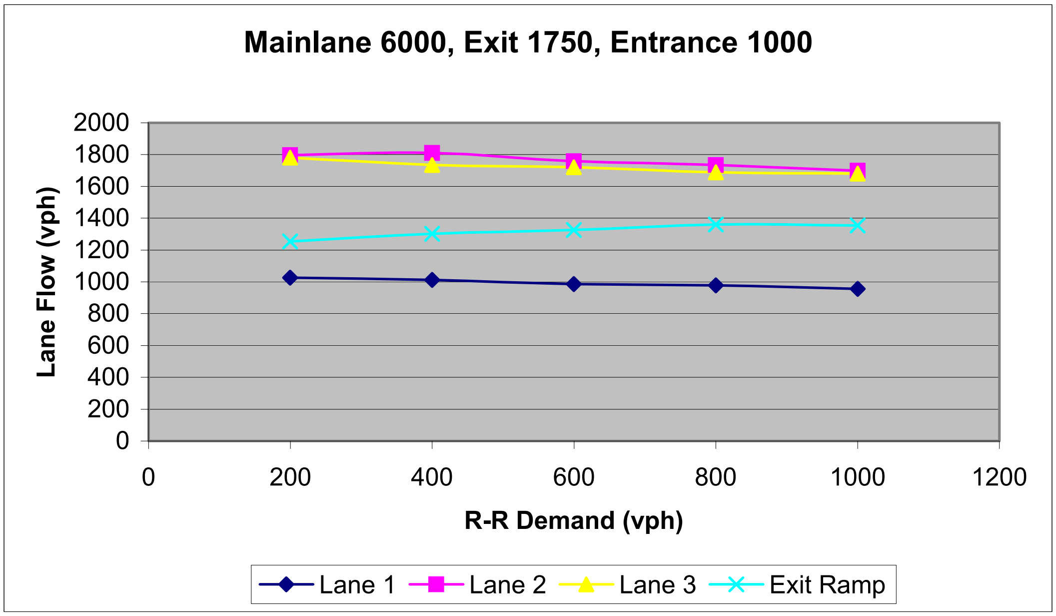

Figure B68 R-R Demand vs Lane Flow For Scenario 23

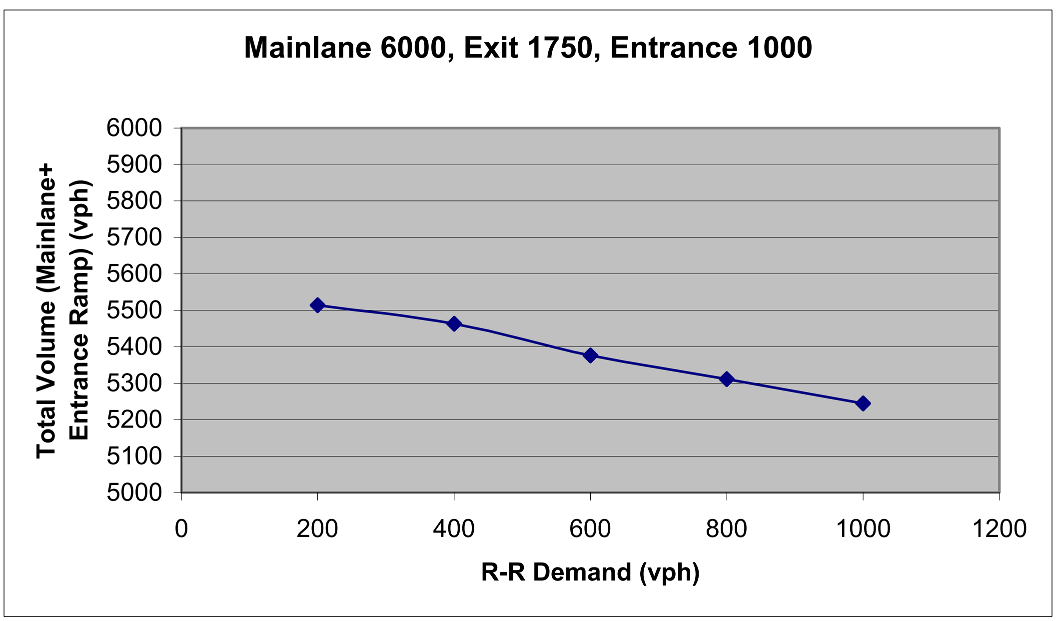

Figure B69 R-R Demand vs Total Volume (V) For Scenario 23

Scenarios 24:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	2000	1000	200	4464	1437	977	198	5441	3424	0.629
6000	2000	1000	400	4430	1454	974	394	5404	3764	0.696
6000	2000	1000	600	4341	1505	975	591	5316	4019	0.756
6000	2000	1000	800	4269	1527	969	780	5239	4302	0.821
6000	2000	1000	1000	4256	1553	980	904	5236	4511	0.861

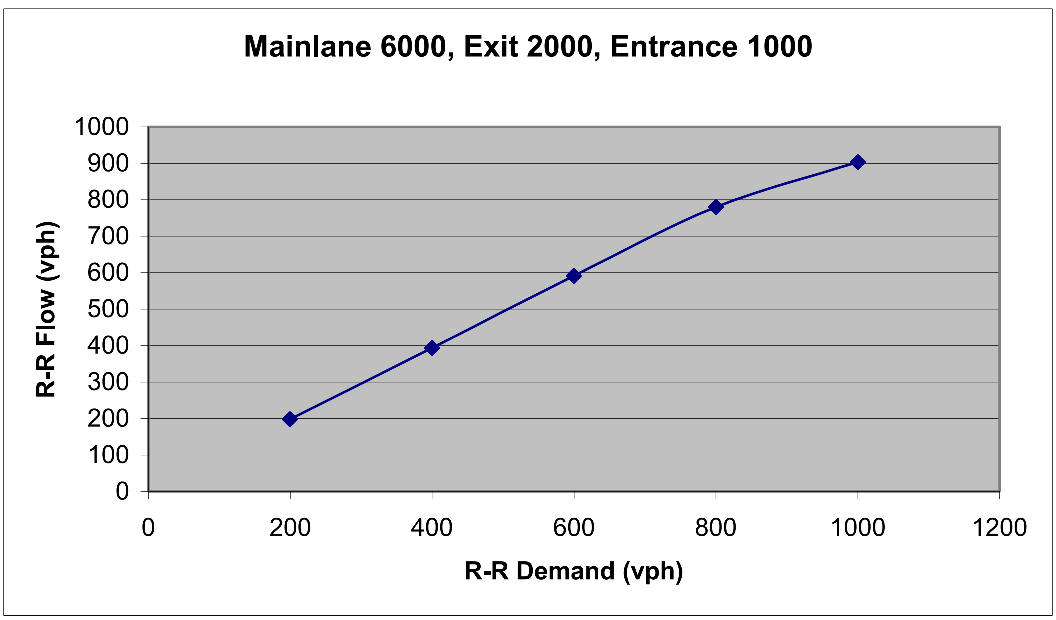

Figure B70 R-R Demand vs R-R Flow For Scenario 24

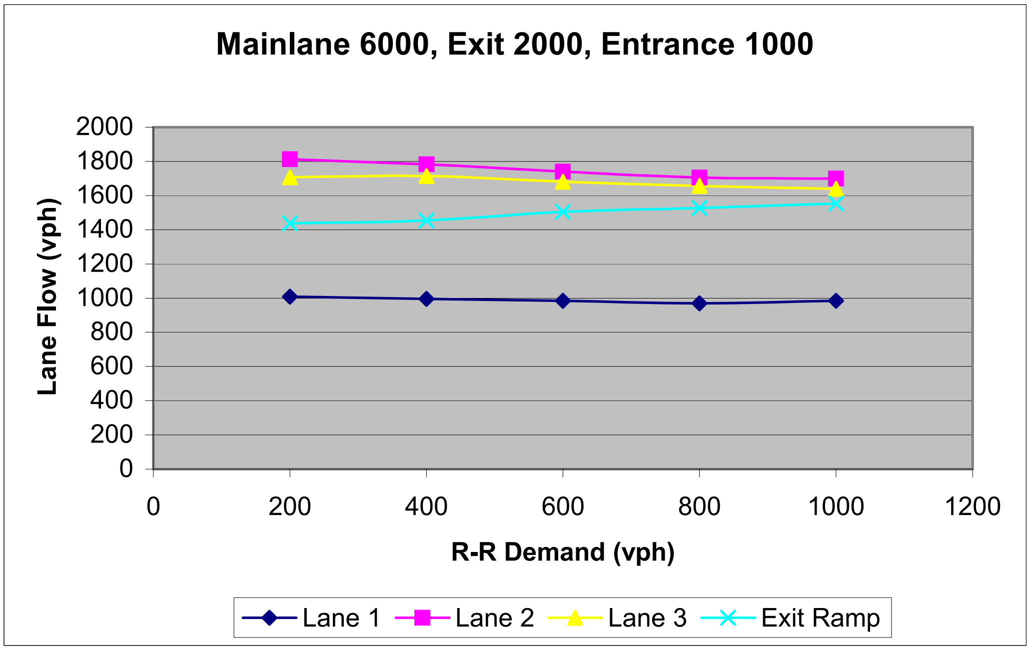

Figure B71 R-R Demand vs Lane Flow For Scenario 24

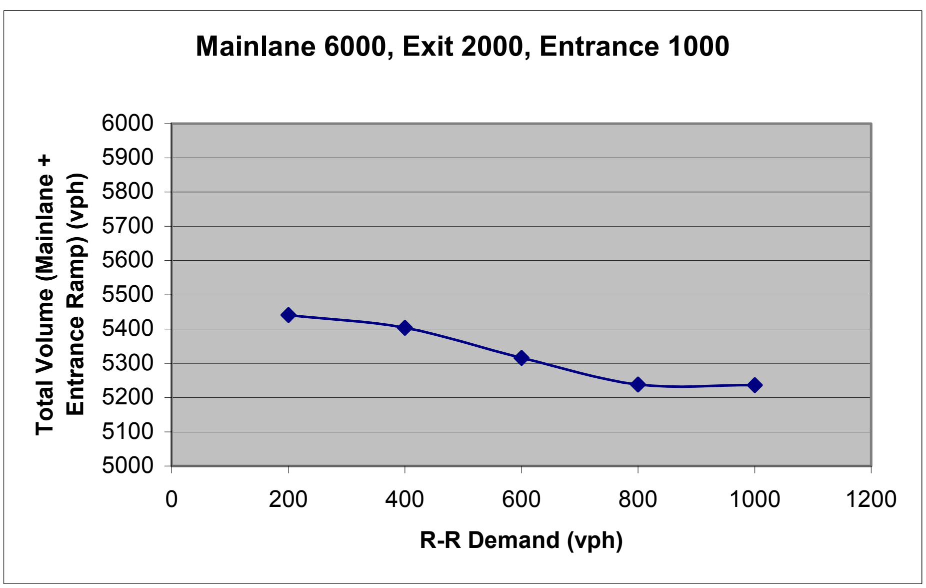

Figure B72 R-R Demand vs Total Volume (V) For Scenario 24

Scenarios 25:

- Total mainlane volume is 6500 vph
- Exit ramp volume is 1000 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6500	1000	1000	200	4776	707	974	187	5750	4443	0.773
6500	1000	1000	400	4675	759	976	392	5651	4701	0.832
6500	1000	1000	600	4539	820	988	595	5527	4908	0.888
6500	1000	1000	800	4507	875	979	786	5486	5205	0.949
6500	1000	1000	1000	4427	891	973	891	5401	5318	0.985

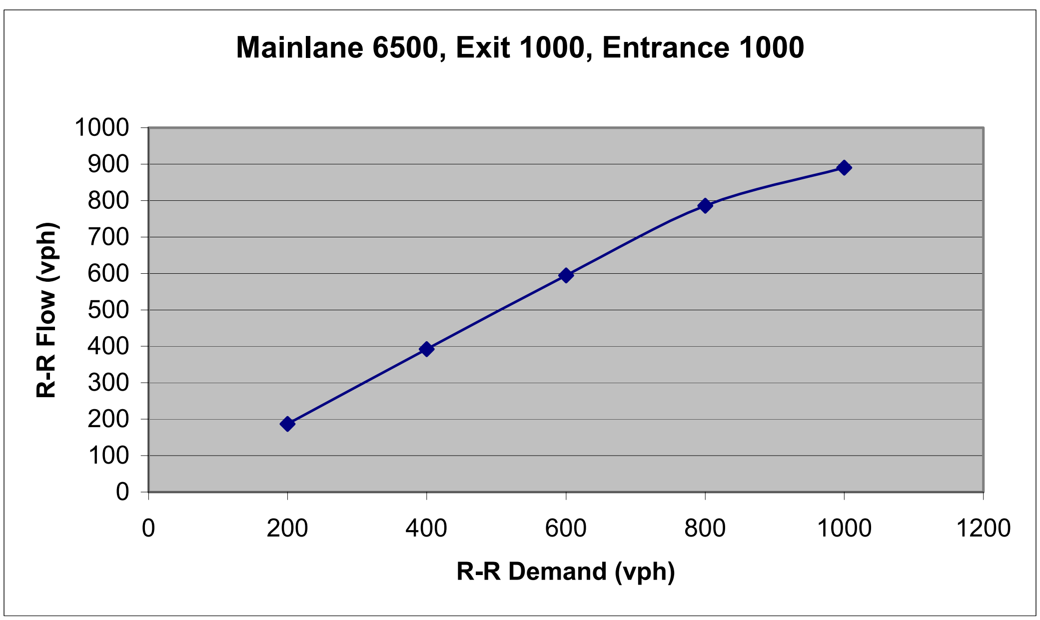

Figure B73 R-R Demand vs R-R Flow For Scenario 25

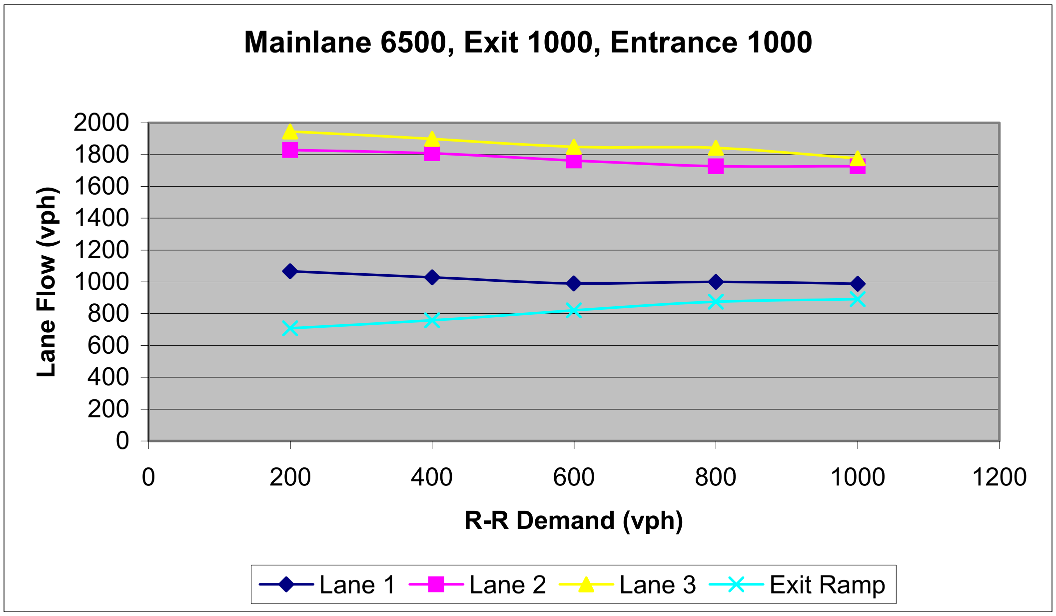

Figure B74 R-R Demand vs Lane Flow For Scenario 25

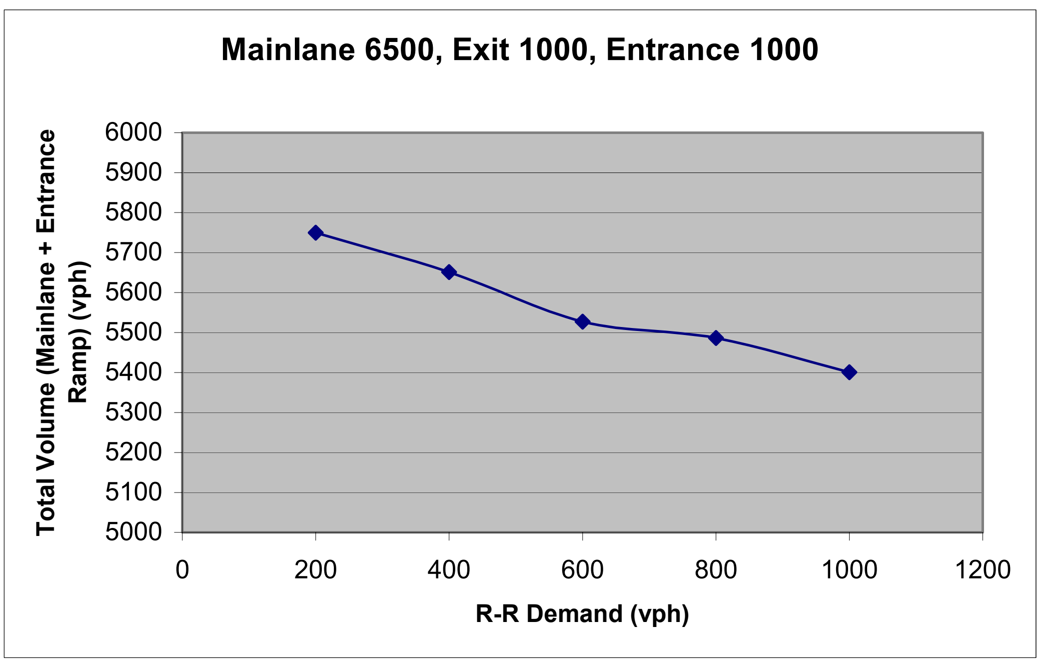

Figure B75 R-R Demand vs Total Volume (V) For Scenario 25

Scenarios 26:

- Total mainlane volume is 6500 vph
- Exit ramp volume is 1500 vph
- R-R demand is from 200-1000
- Entrance ramp is from 1000 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6500	1500	1000	200	4649	1036	984	197	5634	4007	0.711
6500	1500	1000	400	4508	1078	980	410	5487	4249	0.774
6500	1500	1000	600	4498	1111	977	579	5475	4545	0.830
6500	1500	1000	800	4366	1165	975	785	5341	4770	0.893
6500	1500	1000	1000	4317	1208	985	920	5302	4950	0.934

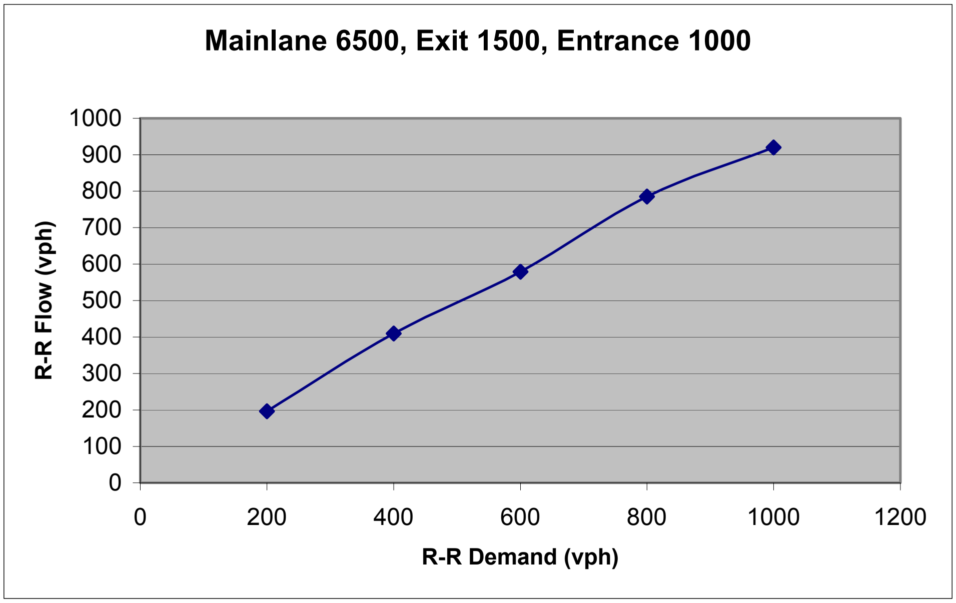

Figure B76 R-R Demand vs R-R Flow For Scenario 26

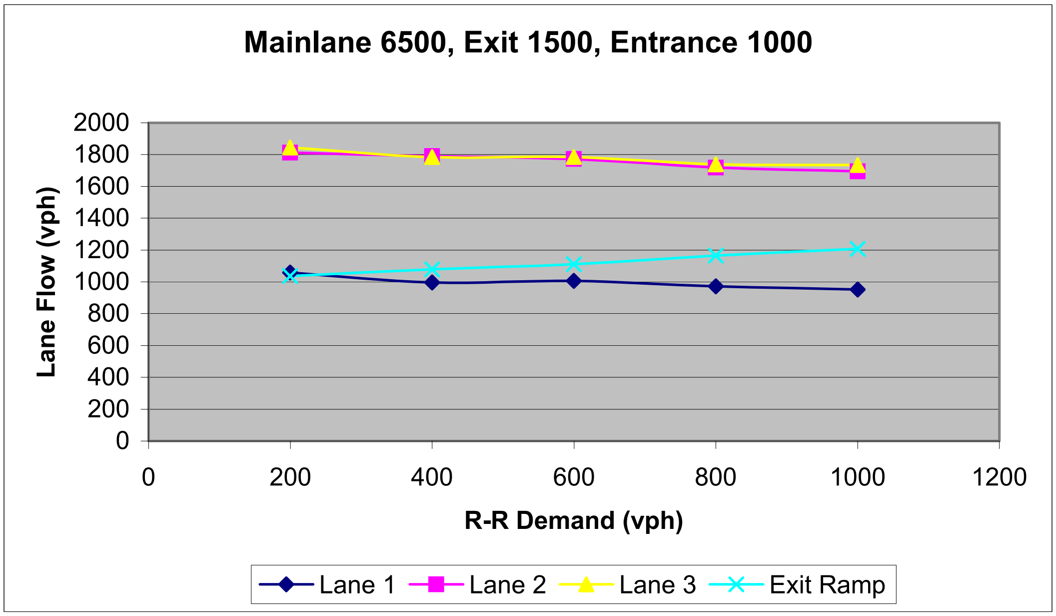

Figure B77 R-R Demand vs Lane Flow For Scenario 26

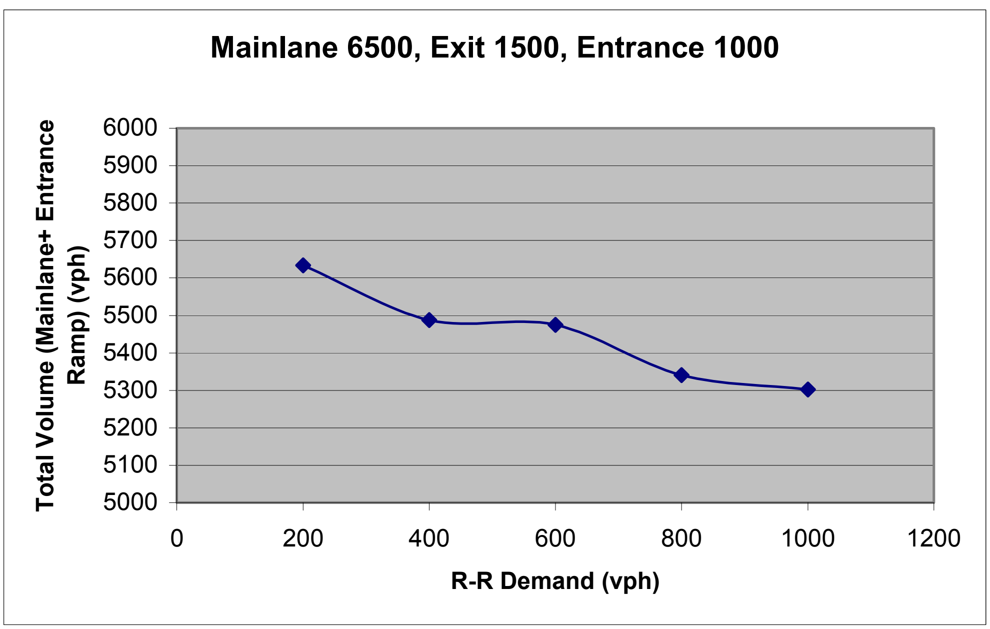

Figure B78R-R Demand vs Total Volume (V) For Scenario 26

APPENDIX C

SIMULATION RESULT FOR STAGE 2- MODEL 2

Appendix C includes simulation results of 16 scenarios in stage 2 model 2.

The inputs of each scenario are summarized in the following table:

Scenario	Entrance Ramp (vph)	Mainlane (vph)	Exit Ramp (veh)	R-R Volume
1	500	5000	800	100-500
2	500	5000	1200	100-500
3	500	5000	1600	100-500
4	500	5000	2000	100-500
5	500	5500	800	100-500
6	500	5500	1200	100-500
7	500	5500	1600	100-500
8	500	5500	2000	100-500
9	500	6000	800	100-500
10	500	6000	1200	100-500
11	500	6000	1600	100-500
12	500	6000	2000	100-500
13	500	6500	800	100-500
14	500	6500	1200	100-500
15	500	6500	1600	100-500
16	500	6500	2000	100-500

Scenarios 1:

- Total mainlane volume is 5000 vph
- Exit ramp volume is 800 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5000	800	500	100	4951	729	494	101	5445	4424	0.812
5000	800	500	300	4950	720	496	302	5446	4835	0.888
5000	800	500	500	4935	724	492	492	5427	5195	0.957

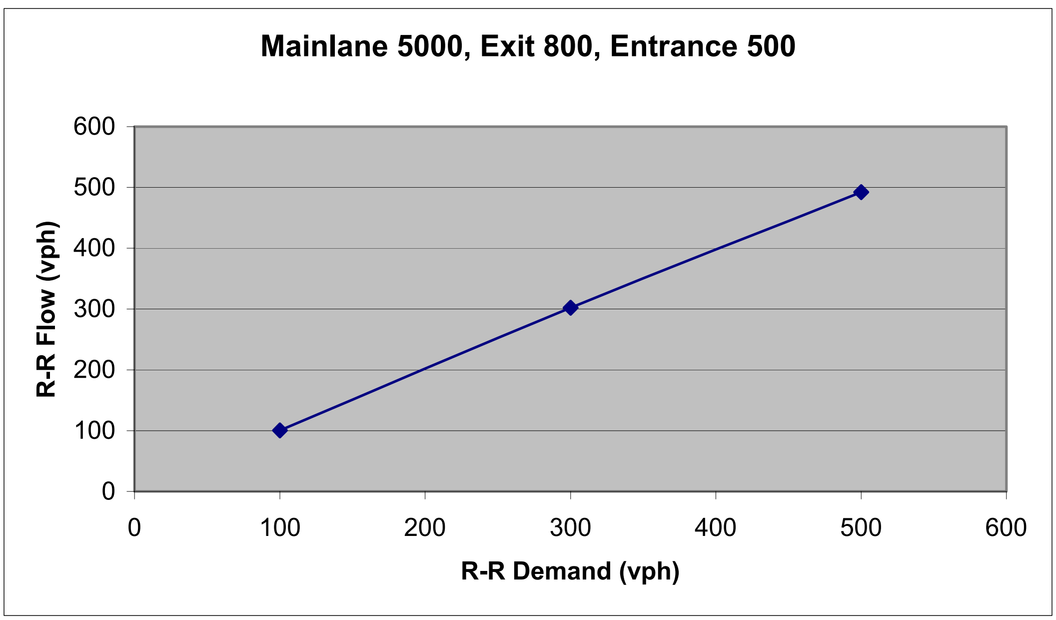

Figure C1 R-R Demand vs R-R Flow for Scenario 1

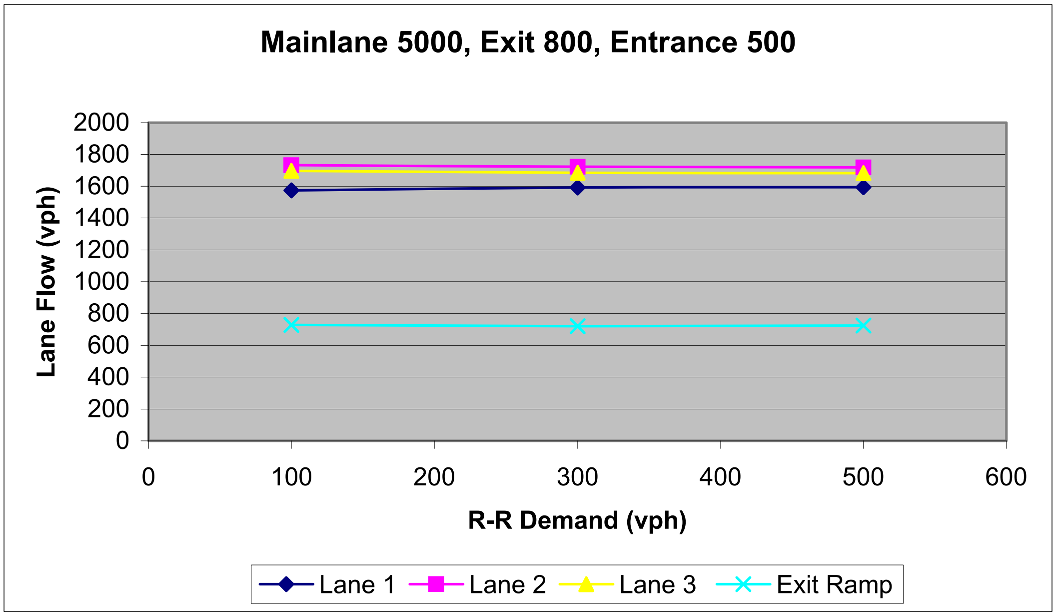

Figure C2 R-R Demand vs Lane Flow for Scenario 1

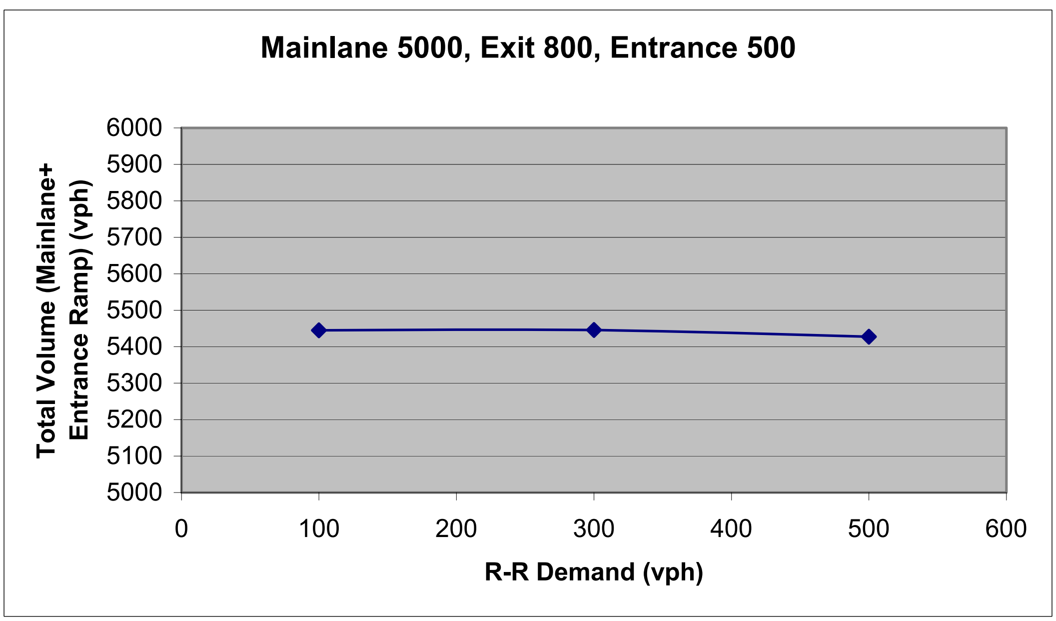

Figure C3 R-R Demand vs Total Volume (V) for Scenario 1

Scenarios 2:

- Total mainlane volume is 5000 vph
- Exit ramp volume is 1200 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5000	1200	500	100	4940	1081	498	100	5438	4059	0.746
5000	1200	500	300	4940	1075	498	302	5438	4469	0.822
5000	1200	500	500	4934	1087	493	493	5427	4833	0.891

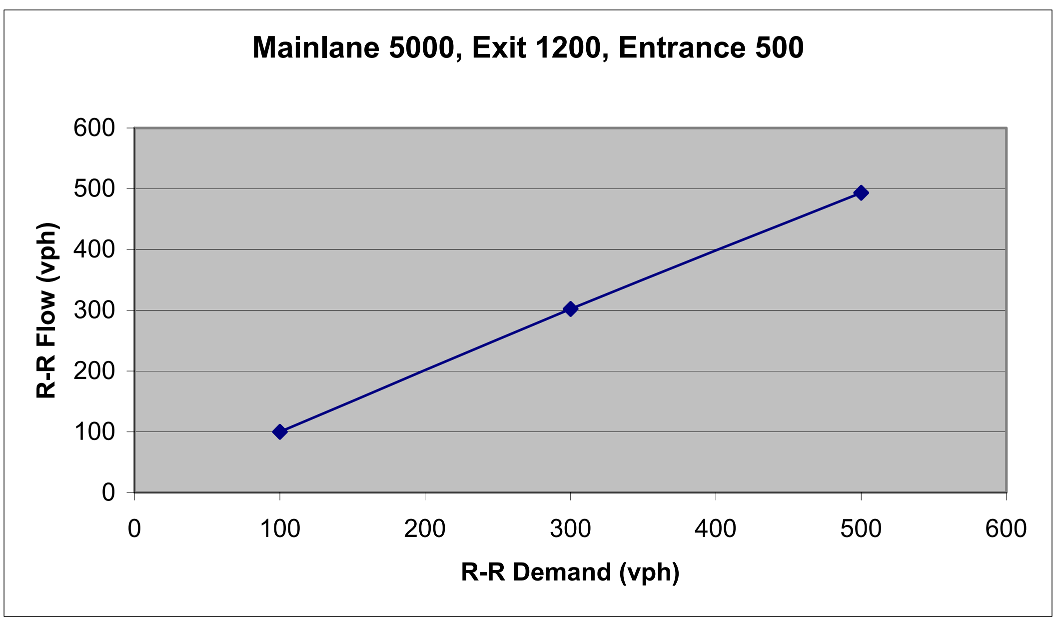

Figure C4 R-R Demand vs R-R Flow For Scenario 2

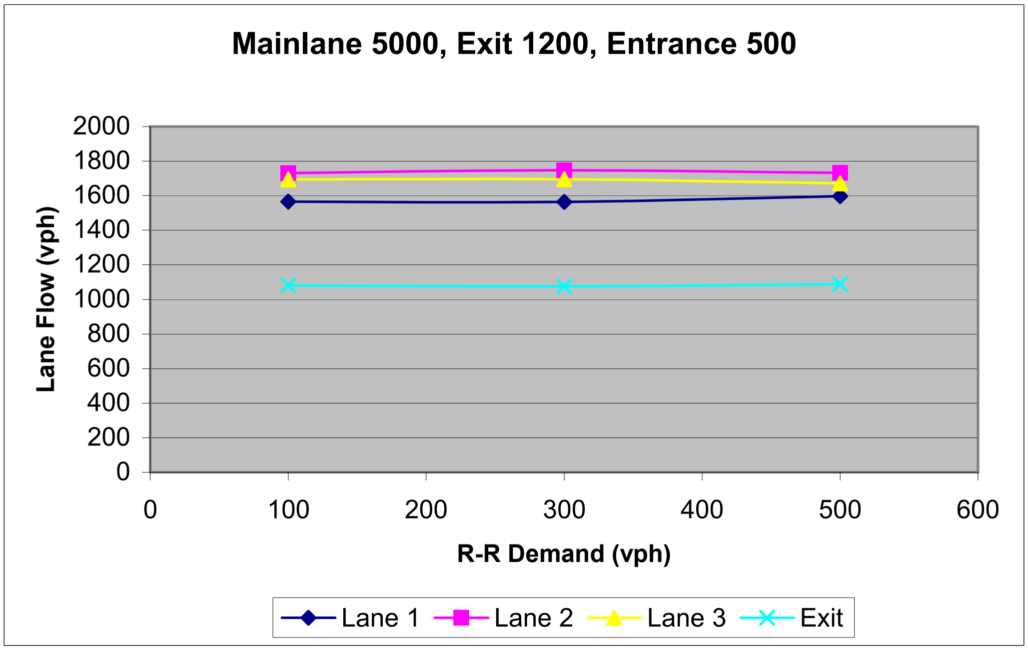

Figure C5 R-R Demand vs Lane Flow For Scenario 2

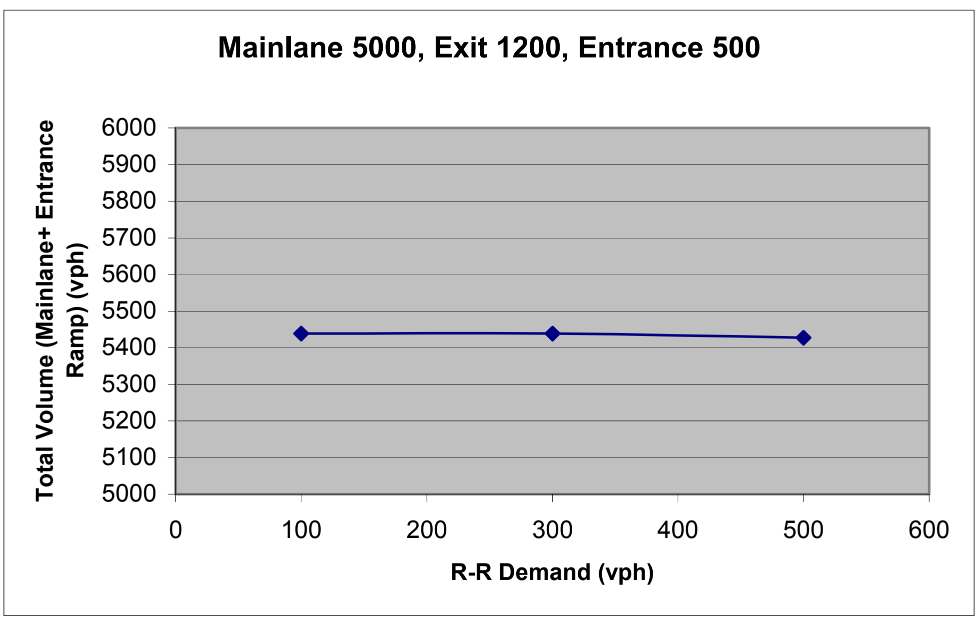

Figure C6 R-R Demand vs Total Volume (V) For Scenario 2

Scenarios 3:

- Total mainlane volume is 5000 vph
- Exit ramp volume is 1600 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system does not reach capacity yet.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5000	1600	500	100	4937	1460	495	95	5432	3667	0.675
5000	1600	500	300	4901	1460	492	294	5393	4030	0.747
5000	1600	500	500	4935	1457	493	493	5428	4463	0.822

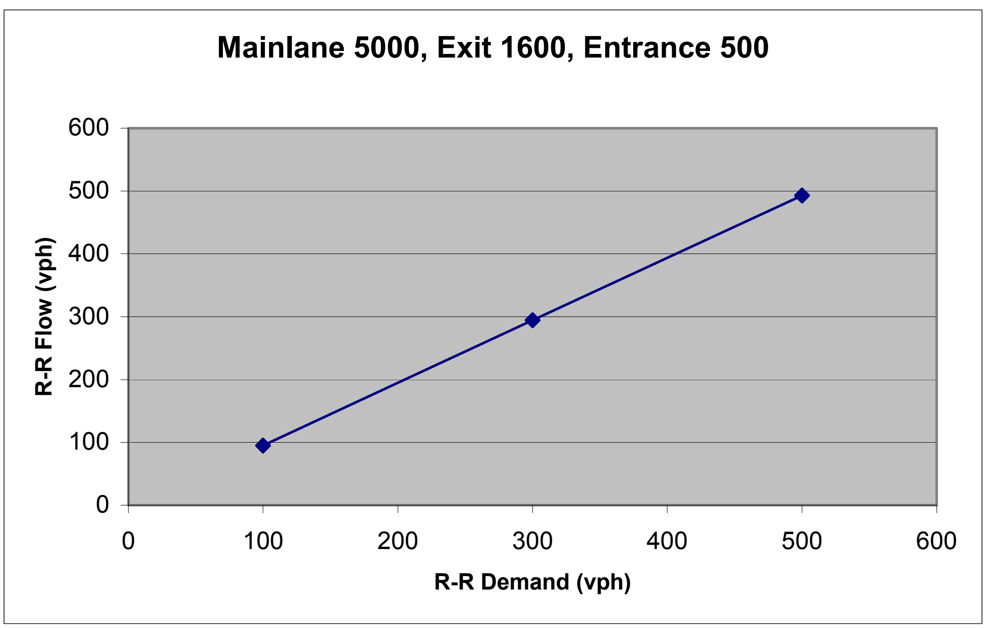

Figure C7 R-R Demand vs R-R Flow For Scenario 3

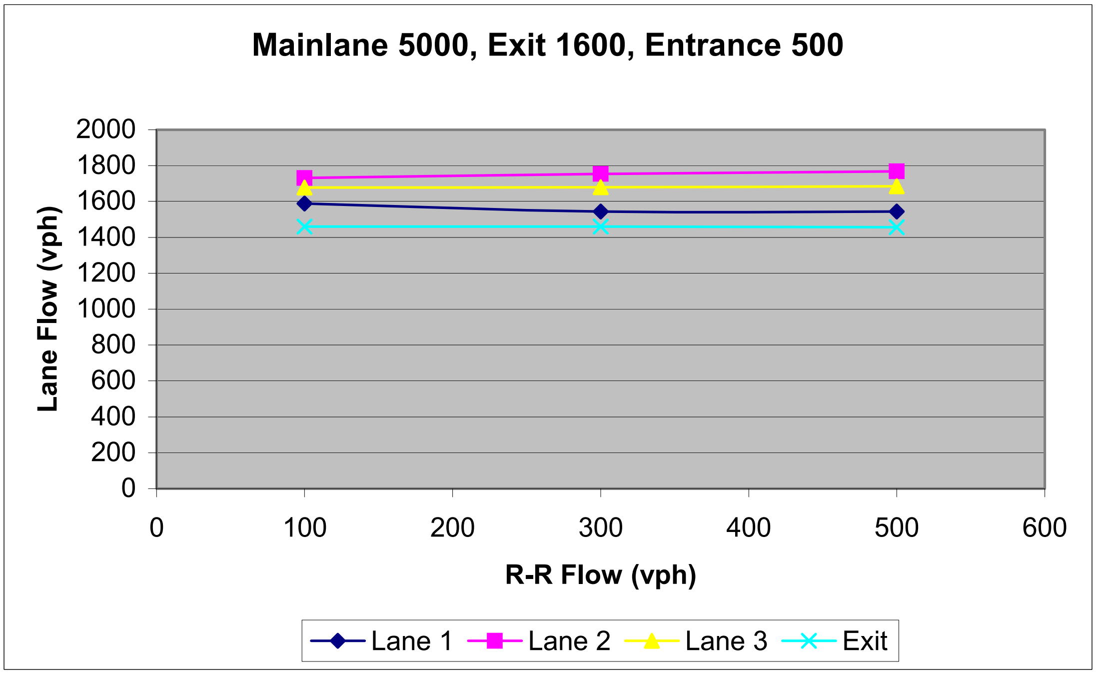

Figure C8 R-R Demand vs Lane Flow For Scenario 3

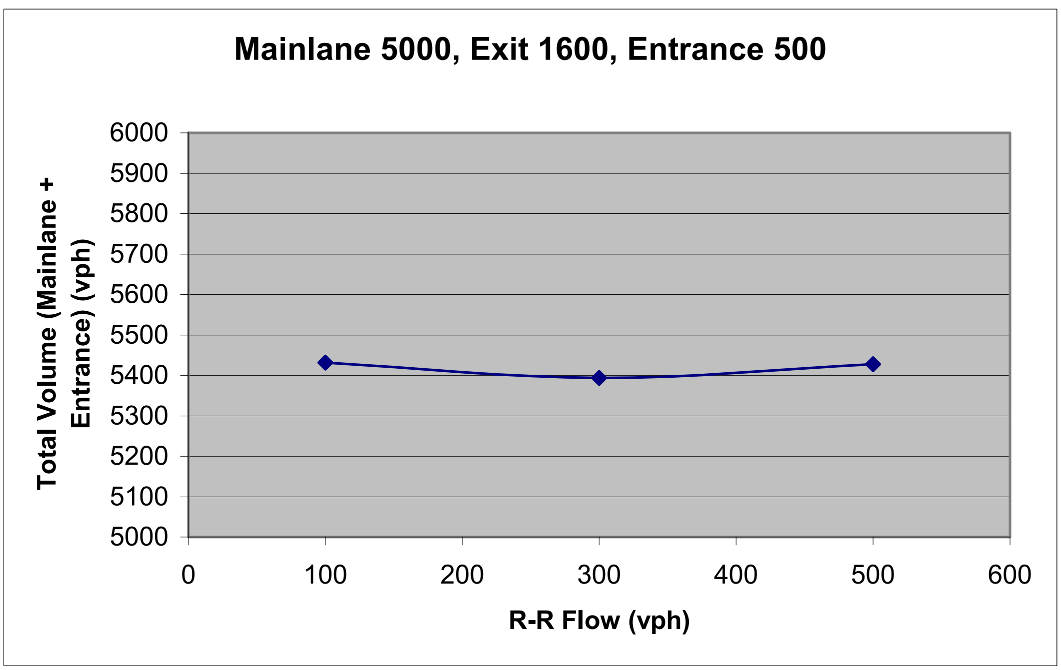

Figure C9 R-R Demand vs Total Volume (V) For Scenario 3

Scenarios 4:

- Total mainlane volume is 5000 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5000	2000	500	100	4836	1787	491	98	5327	3245	0.609
5000	2000	500	300	4840	1818	497	307	5337	3635	0.681
5000	2000	500	500	4781	1702	494	494	5276	4068	0.771

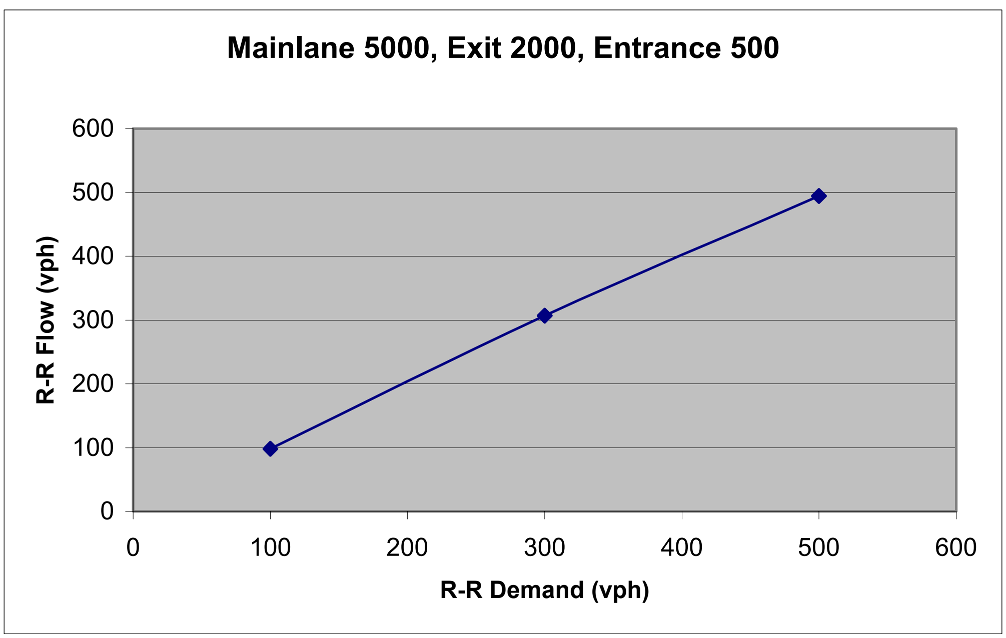

Figure C10 R-R Demand vs R-R Flow For Scenario 4

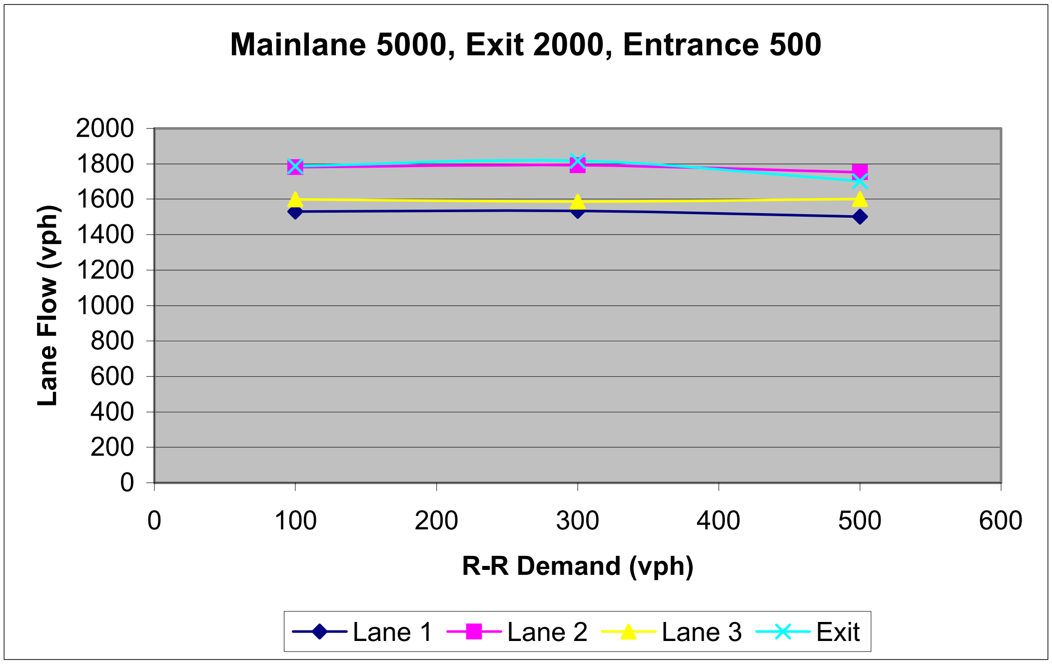

Figure C11 R-R Demand vs Lane Flow For Scenario 4

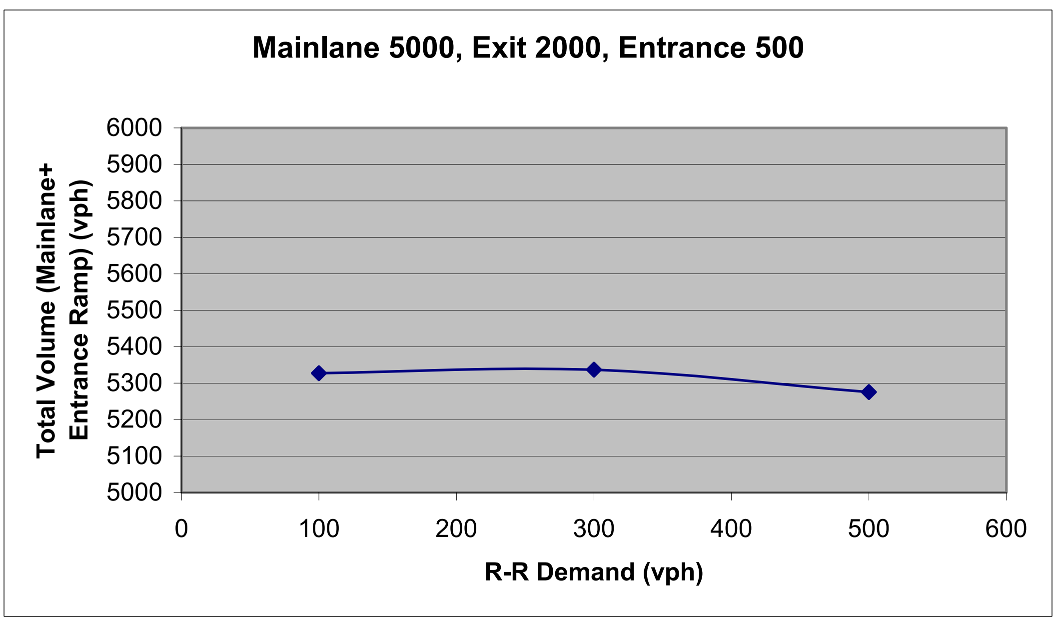

Figure C12 R-R Demand vs Total Volume (V) For Scenario 4

Scenarios 5:

- Total mainlane volume is 5500 vph
- Exit ramp volume is 800 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5500	800	500	100	5370	713	493	100	5863	4857	0.828
5500	800	500	300	5300	723	494	300	5793	5177	0.894
5500	800	500	500	5261	712	495	495	5755	5538	0.962

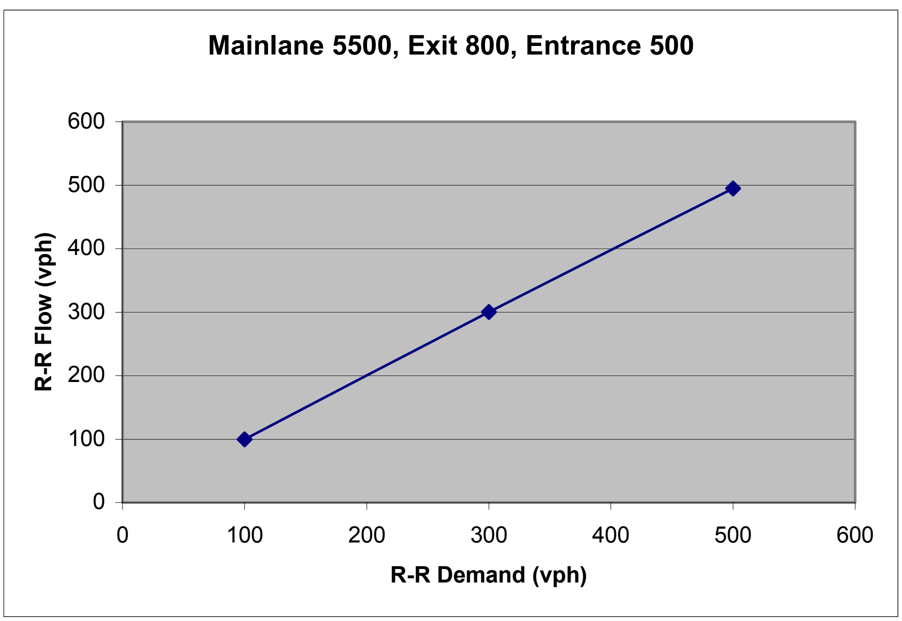

Figure C13 R-R Demand vs R-R Flow For Scenario 5

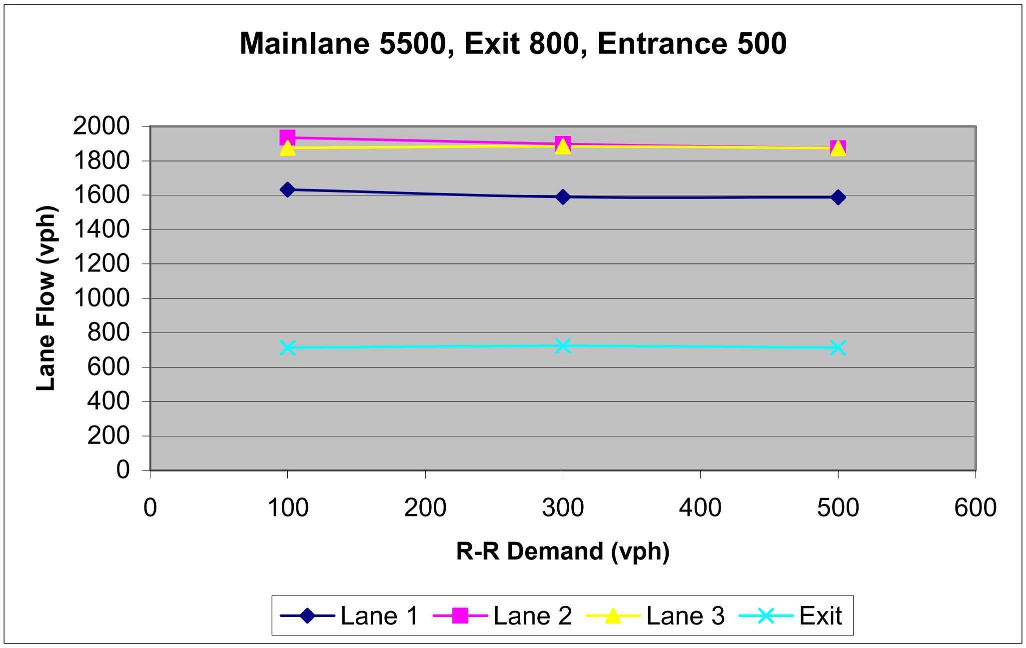

Figure C14 R-R Demand vs Lane Flow For Scenario 5

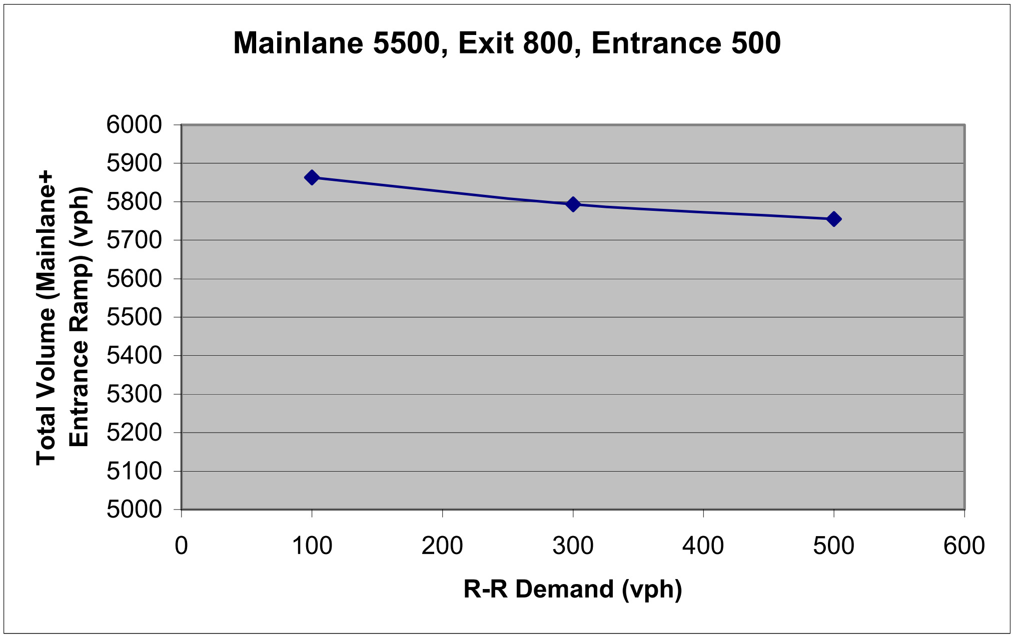

Figure C15 R-R Demand vs Total Volume (V) For Scenario 5

Scenarios 6:

- Total mainlane volume is 5500 vph
- Exit ramp volume is 1200 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5500	1200	500	100	5234	1054	494	103	5727	4386	0.766
5500	1200	500	300	5215	1045	489	289	5705	4749	0.832
5500	1200	500	500	5121	1047	495	495	5616	5064	0.902

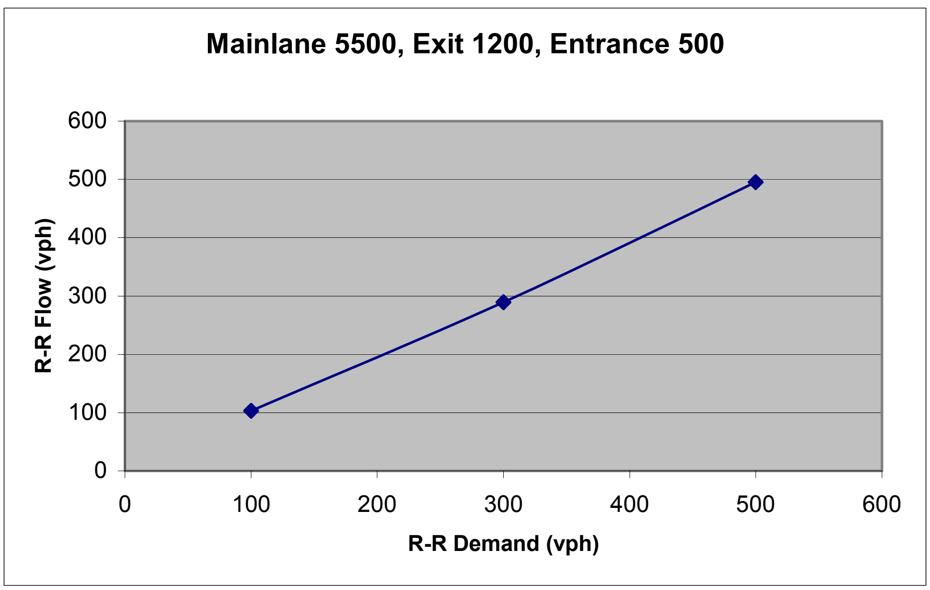

Figure C16 R-R Demand vs R-R Flow For Scenario 6

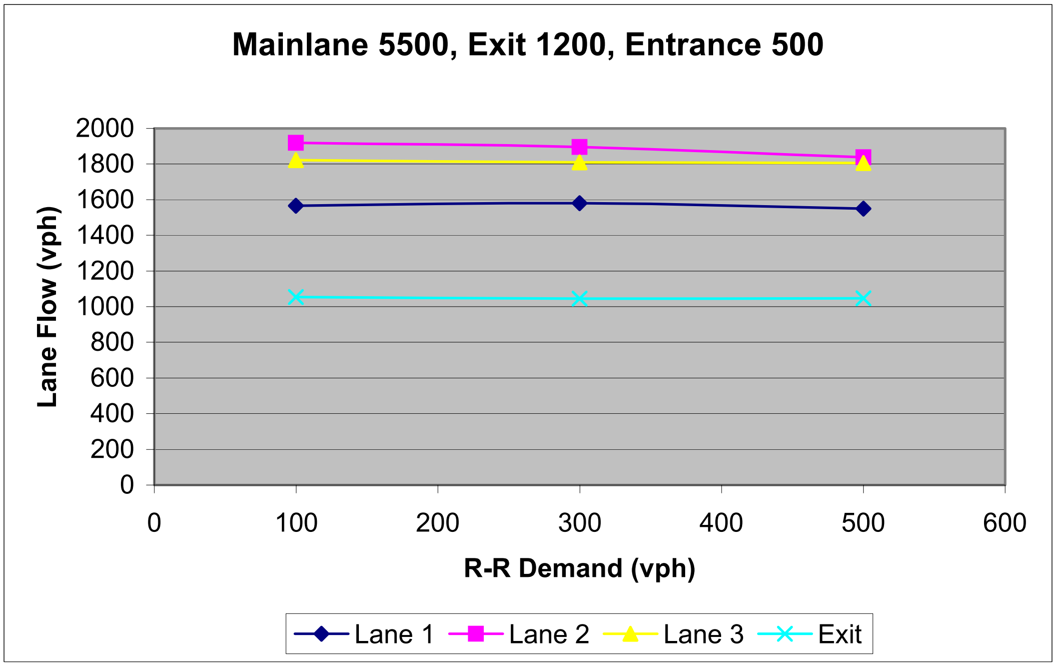

Figure C17 R-R Demand vs Lane Flow For Scenario 6

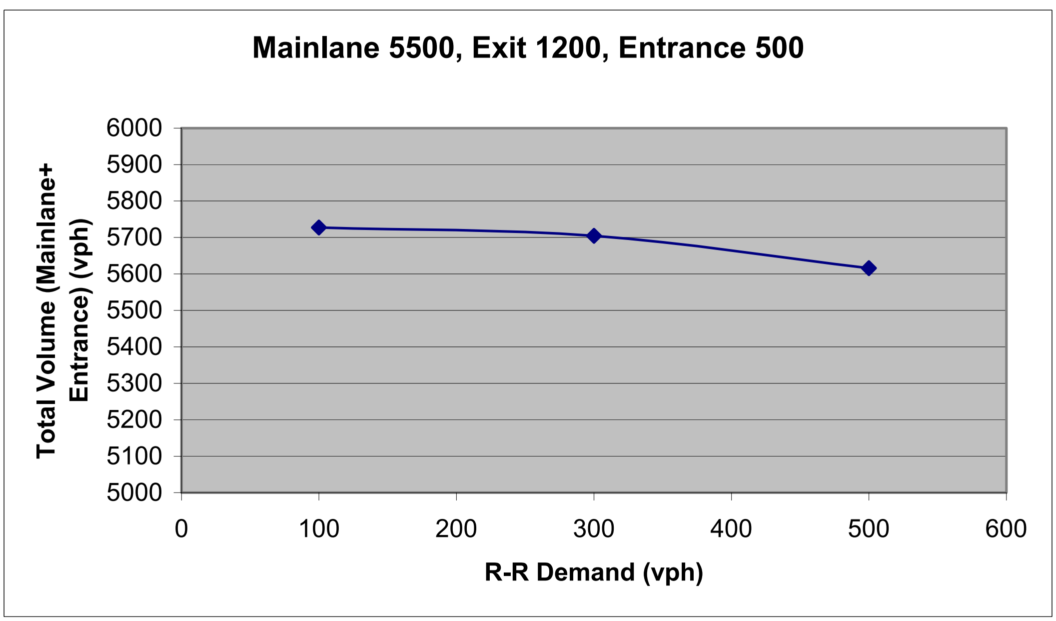

Figure C18 R-R Demand vs Total Volume (V) For Scenario 6

Scenarios 7:

- Total mainlane volume is 5500 vph
- Exit ramp volume is 1600 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5500	1600	500	100	5095	1381	494	114	5588	3942	0.705
5500	1600	500	300	5099	1357	493	299	5591	4340	0.776
5500	1600	500	500	5019	1397	496	496	5515	4615	0.837

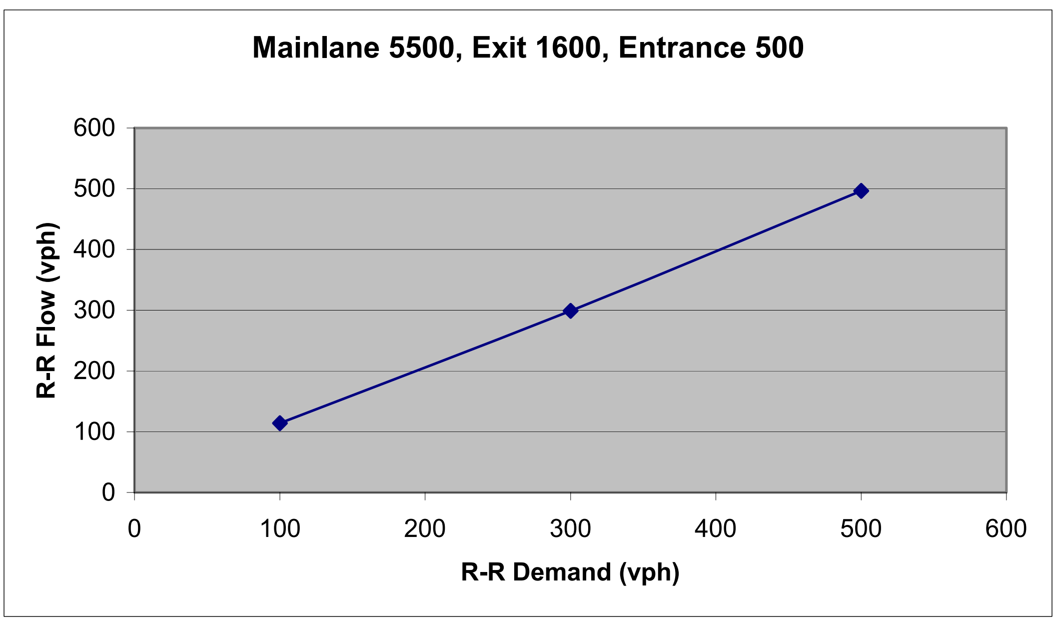

Figure C19 R-R Demand vs R-R Flow For Scenario 7

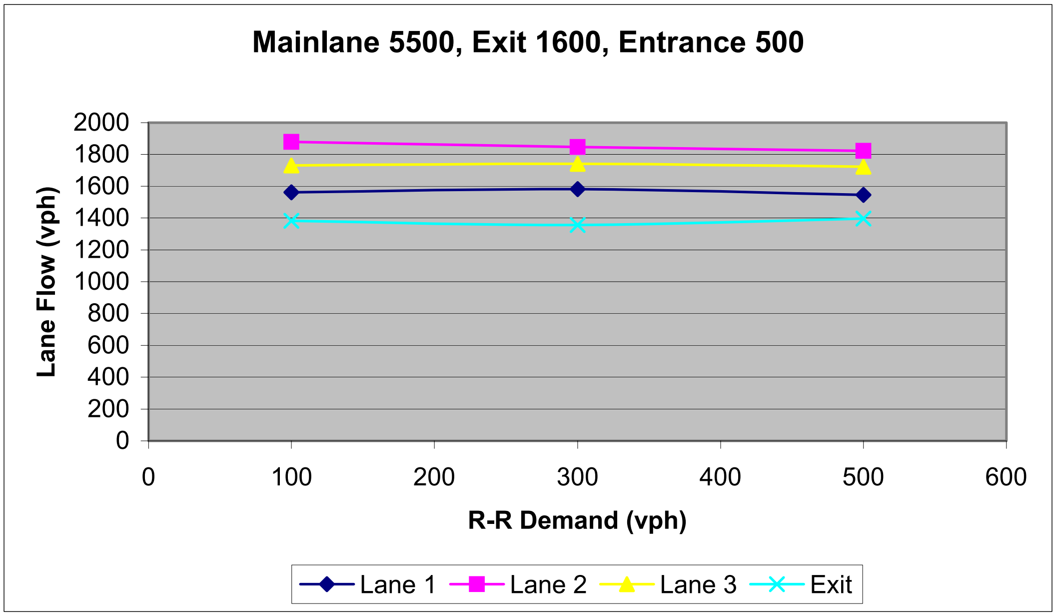

Figure C20 R-R Demand vs Lane Flow For Scenario 7

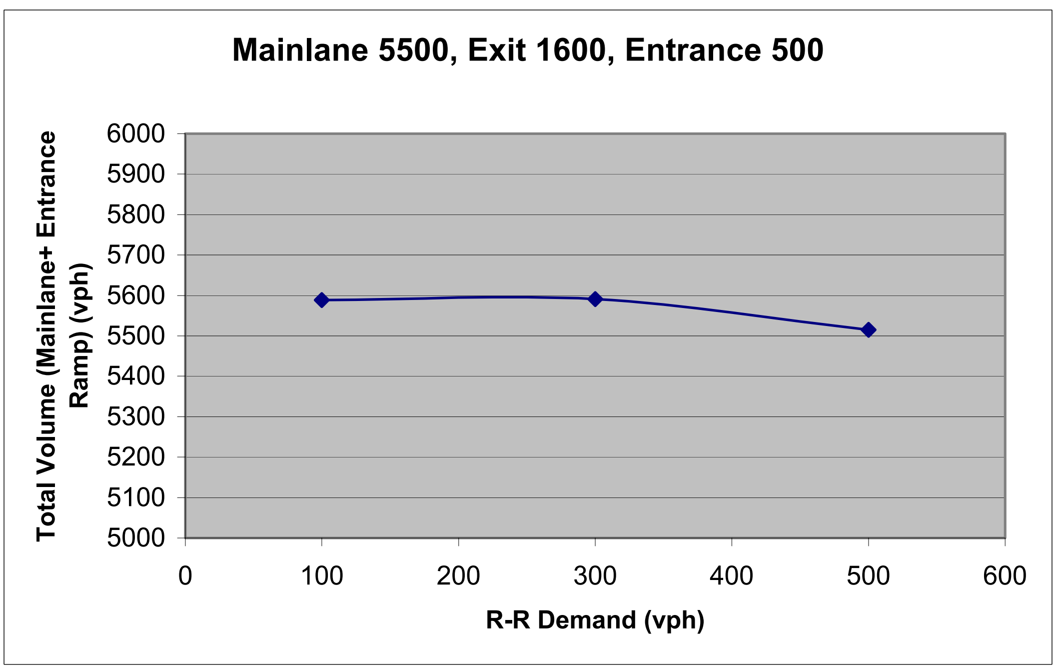

Figure C21 R-R Demand vs Total Volume (V) For Scenario 7

Scenarios 8:

- Total mainlane volume is 5500 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
5500	2000	500	100	4952	1686	497	102	5449	3469	0.637
5500	2000	500	300	4898	1672	493	300	5391	3825	0.710
5500	2000	500	500	4835	1692	492	492	5328	4128	0.775

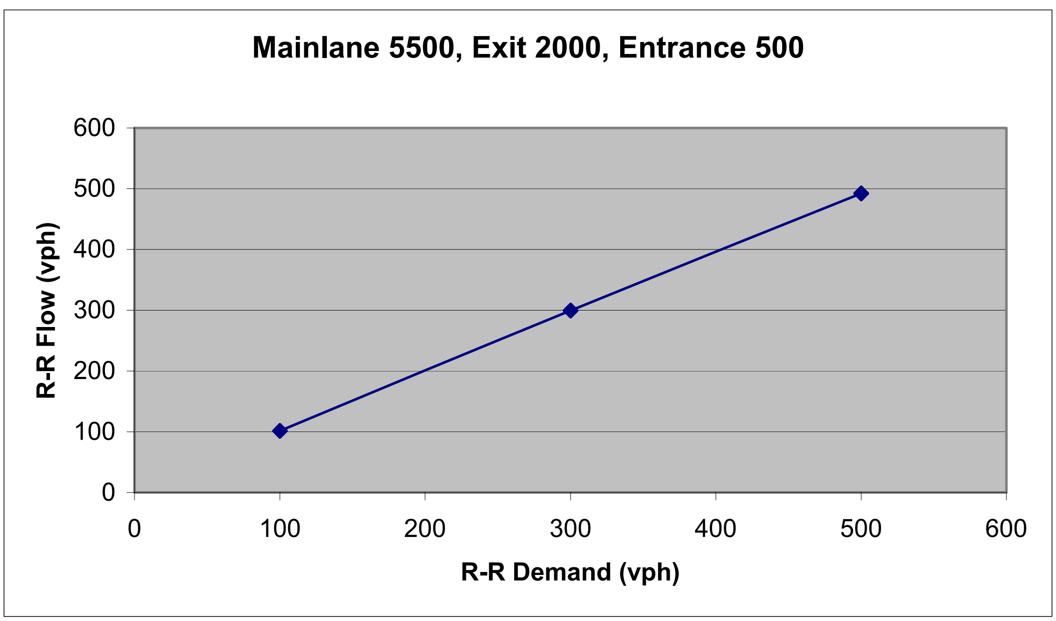

Figure C22 R-R Demand vs R-R Flow For Scenario 8

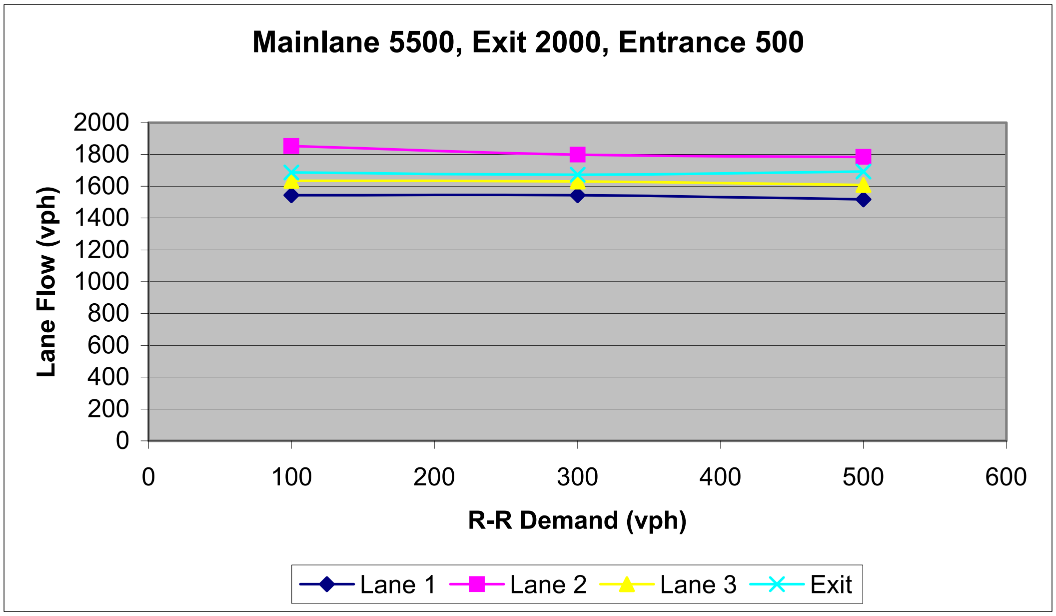

Figure C23 R-R Demand vs Lane Flow For Scenario 8

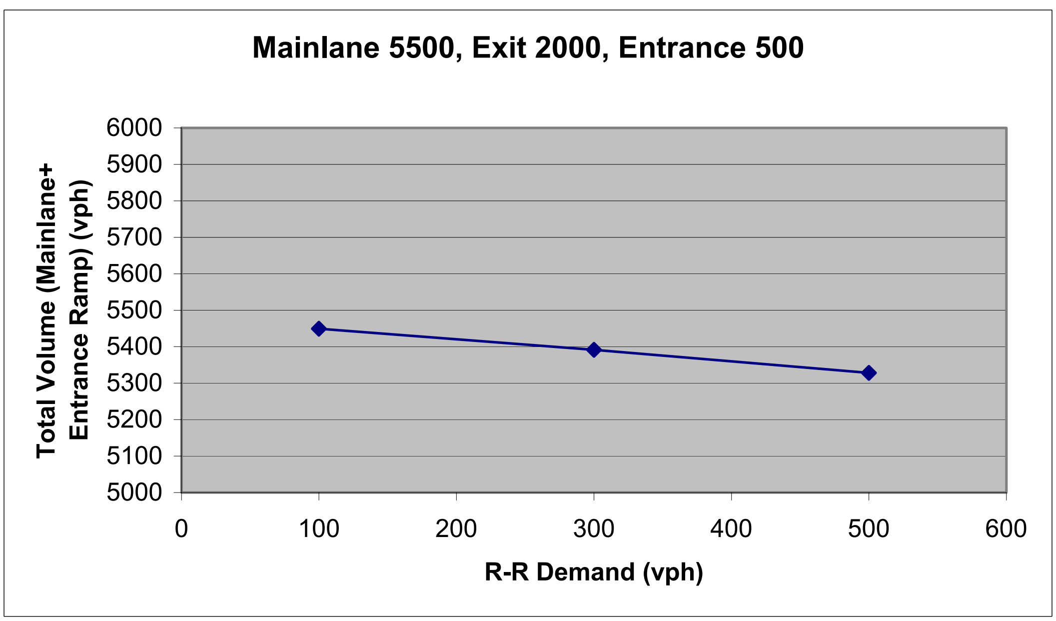

Figure C24 R-R Demand vs Total Volume (V) For Scenario 8

Scenarios 9:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 800 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	800	500	100	5407	670	495	102	5902	4941	0.837
6000	800	500	200	5414	698	494	211	5908	5139	0.870
6000	800	500	300	5343	678	491	295	5834	5256	0.901
6000	800	500	400	5299	681	492	389	5790	5397	0.932
6000	800	500	500	5262	686	491	491	5753	5558	0.966

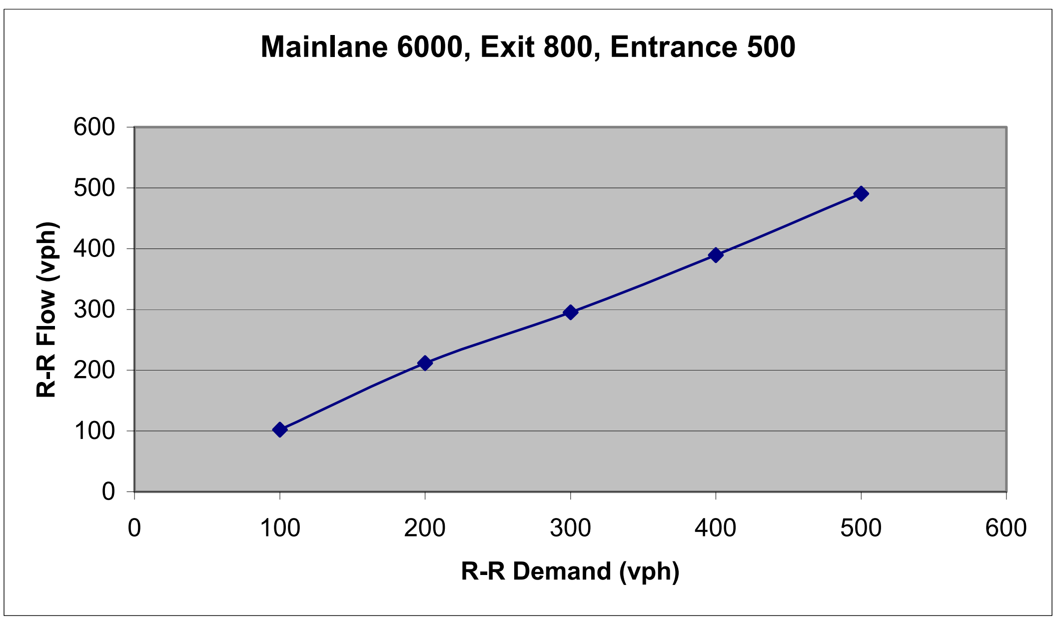

Figure C25 R-R Demand vs R-R Flow For Scenario 9

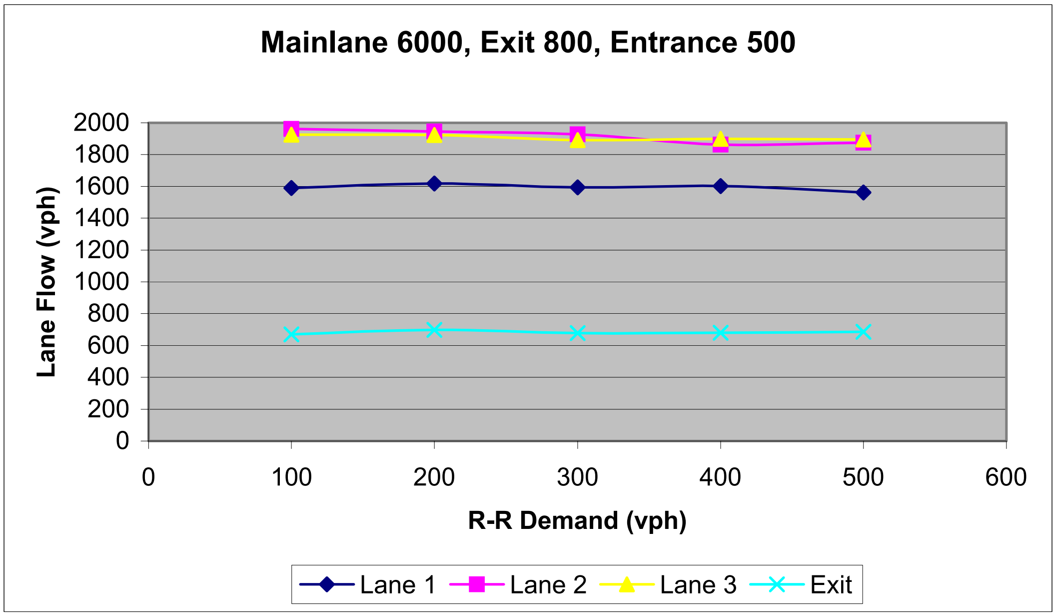

Figure C26 R-R Demand vs Lane Flow For Scenario 9

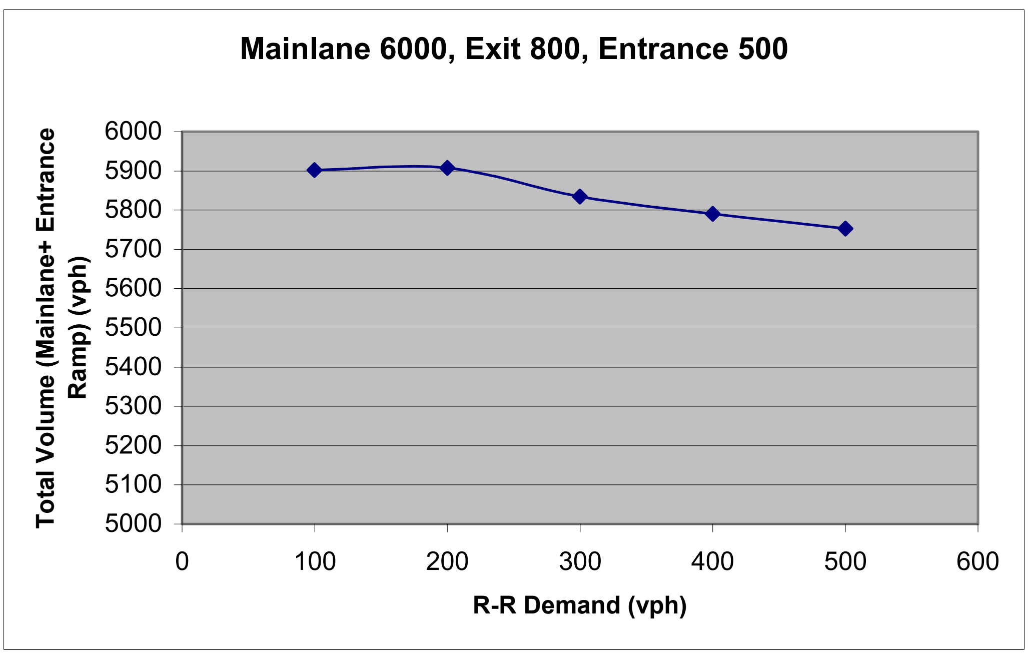

Figure C27 R-R Demand vs Total Volume (V) For Scenario 9

Scenarios 10:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 1200 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	1200	500	100	5293	983	493	92	5786	4494	0.777
6000	1200	500	200	5250	987	498	195	5748	4654	0.810
6000	1200	500	300	5238	1010	490	301	5728	4829	0.843
6000	1200	500	400	5163	1001	493	394	5657	4950	0.875
6000	1200	500	500	5130	1003	493	493	5623	5113	0.909

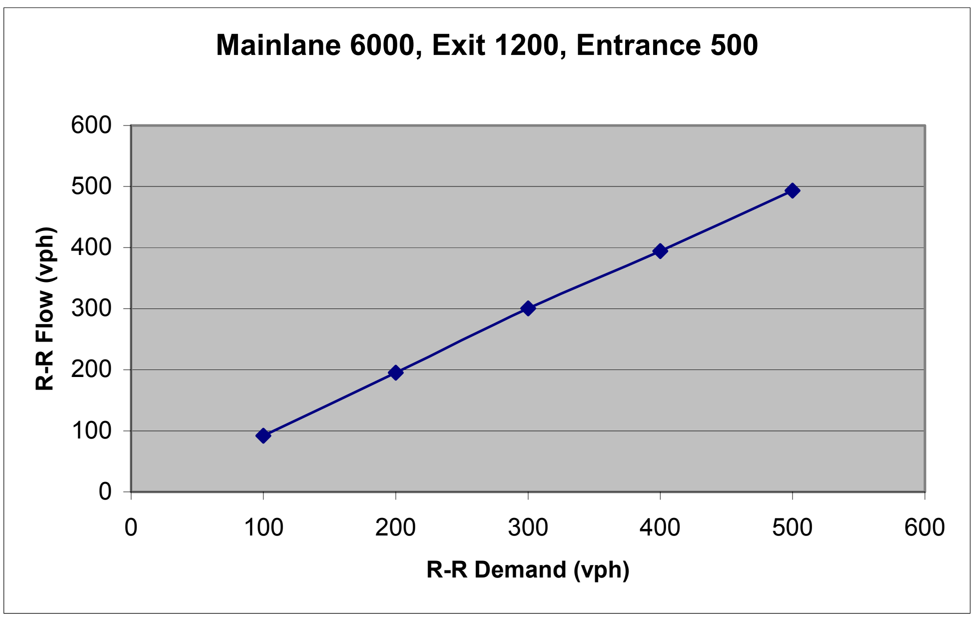

Figure C28 R-R Demand vs R-R Flow For Scenario 10

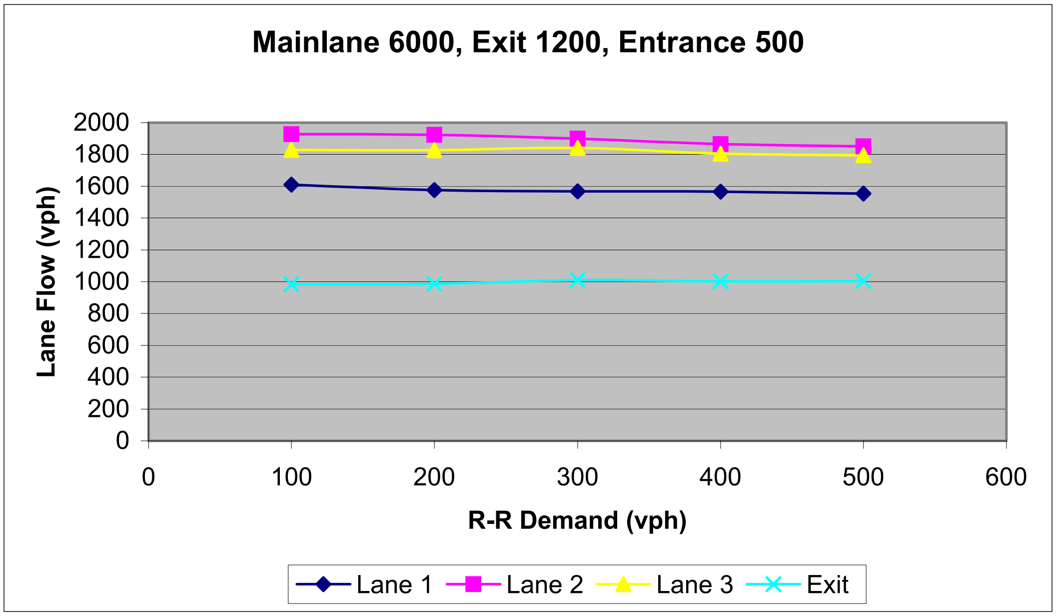

Figure C29 R-R Demand vs Lane Flow For Scenario 10

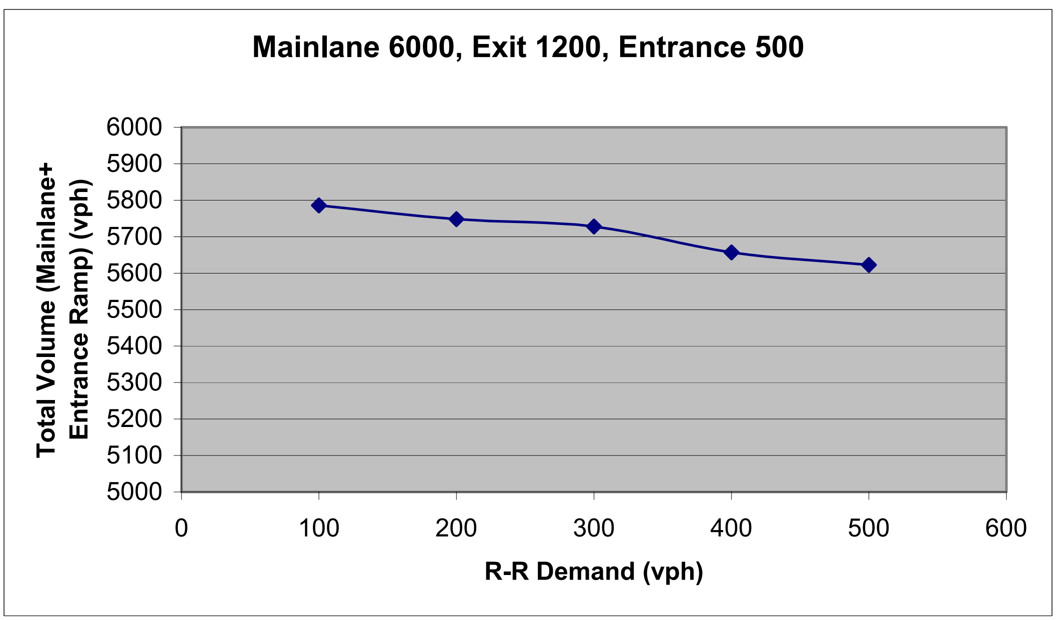

Figure C30 R-R Demand vs Total Volume (V) For Scenario 10

Scenarios 11:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 1600 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	1600	500	100	5150	1275	494	104	5644	4082	0.723
6000	1600	500	200	5124	1276	495	199	5619	4246	0.756
6000	1600	500	300	5068	1260	499	302	5567	4412	0.793
6000	1600	500	400	5053	1309	493	399	5547	4542	0.819
6000	1600	500	500	4995	1283	494	494	5489	4700	0.856

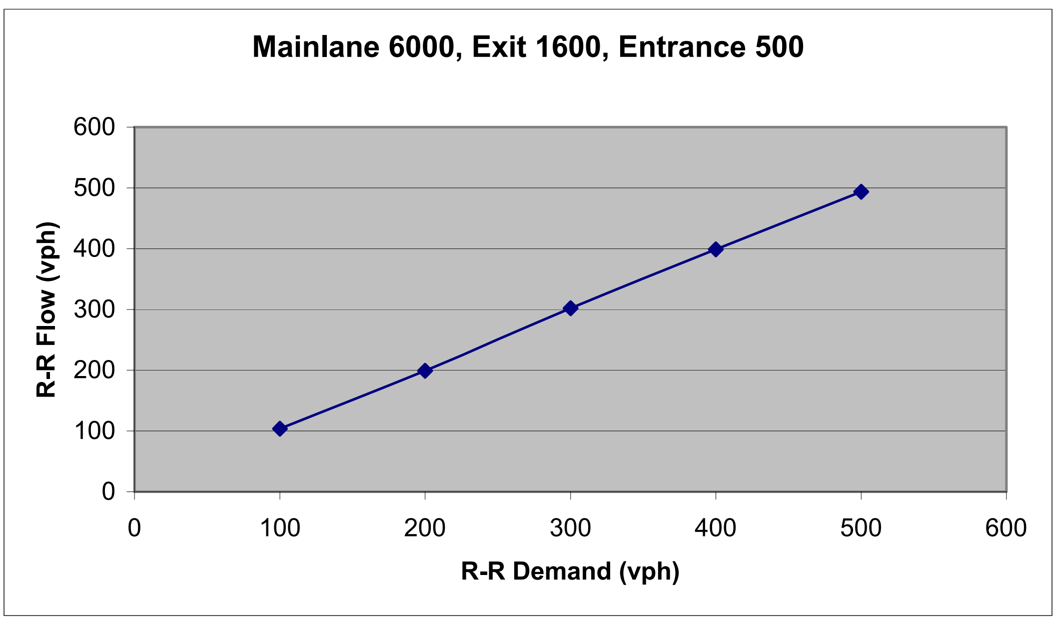

Figure C31 R-R Demand vs R-R Flow For Scenario 11

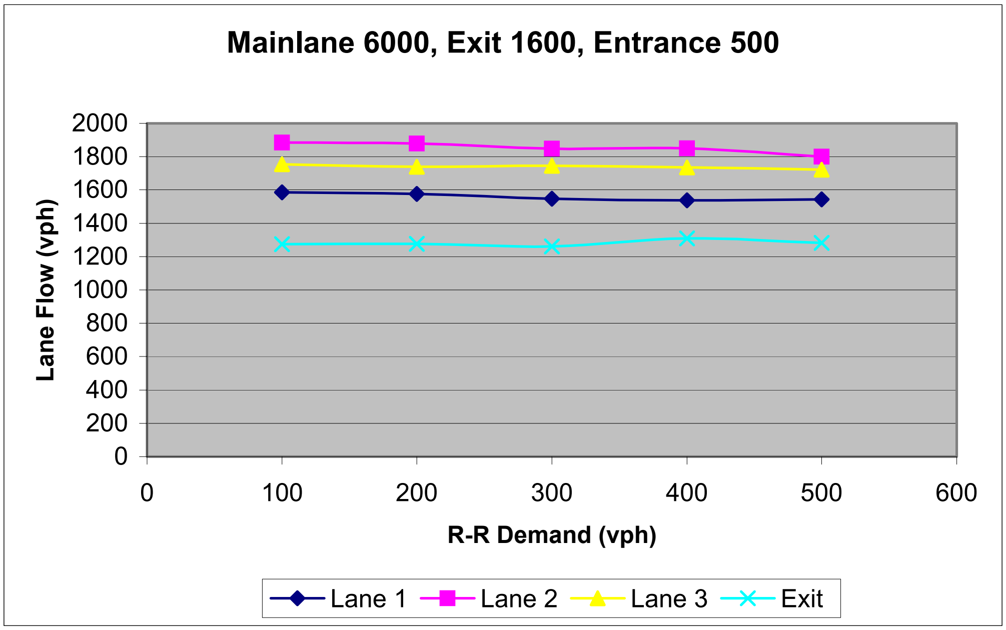

Figure C32 R-R Demand vs Lane Flow For Scenario 11

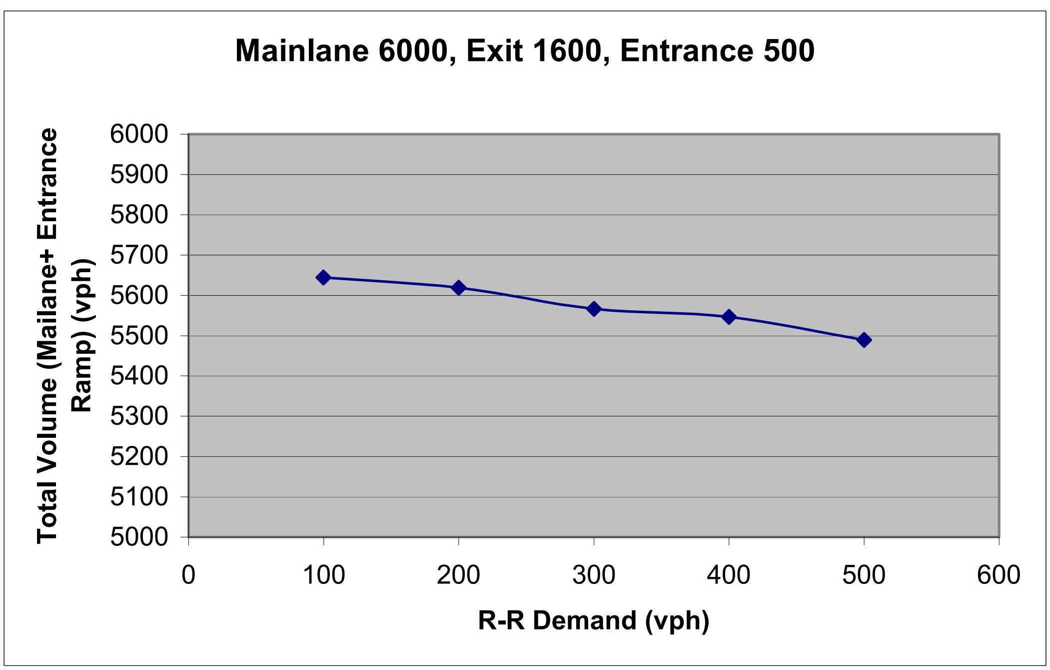

Figure C33 R-R Demand vs Total Volume (V) For Scenario 11

Scenarios 12:

- Total mainlane volume is 6000 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6000	2000	500	100	4999	1559	497	98	5496	3637	0.662
6000	2000	500	200	4995	1586	494	200	5489	3809	0.694
6000	2000	500	300	4973	1572	495	295	5468	3992	0.730
6000	2000	500	400	4907	1574	494	403	5401	4139	0.766
6000	2000	500	500	4896	1571	492	492	5388	4310	0.800

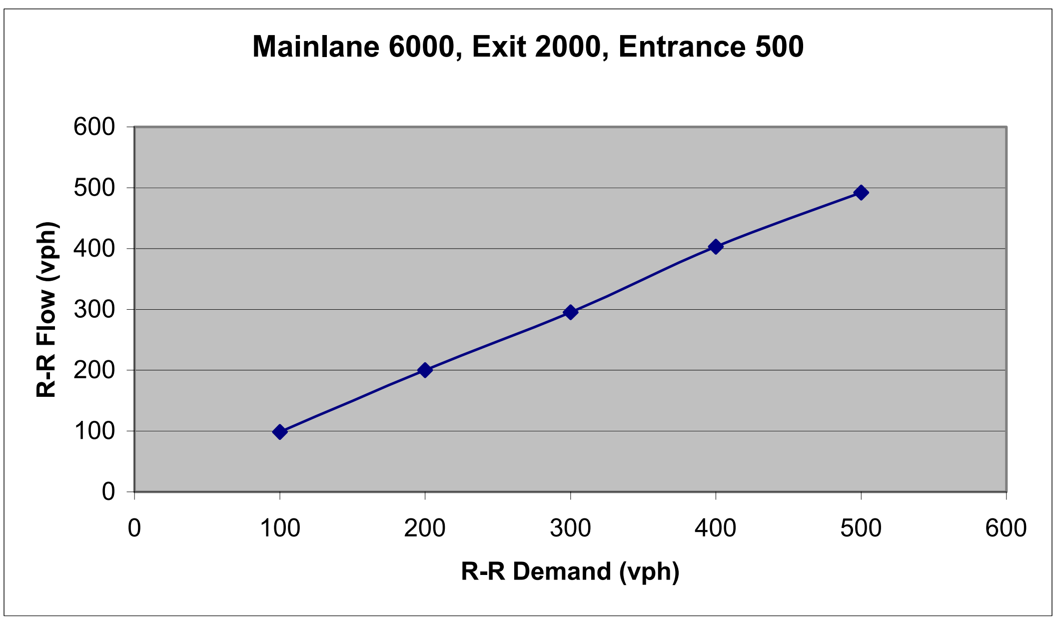

Figure C34 R-R Demand vs R-R Flow For Scenario 12

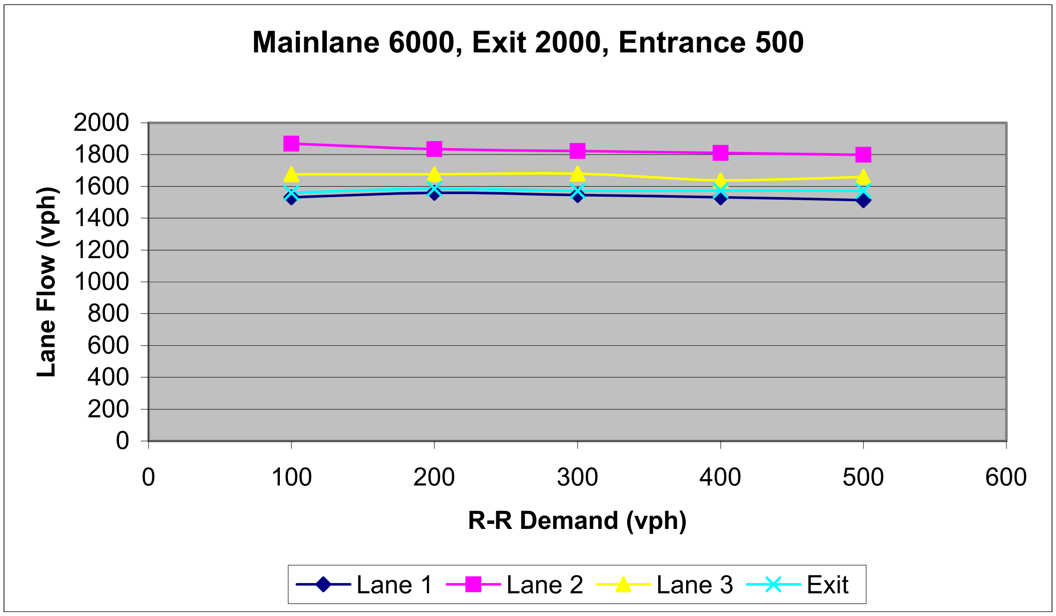

Figure C35 R-R Demand vs Lane Flow For Scenario 12

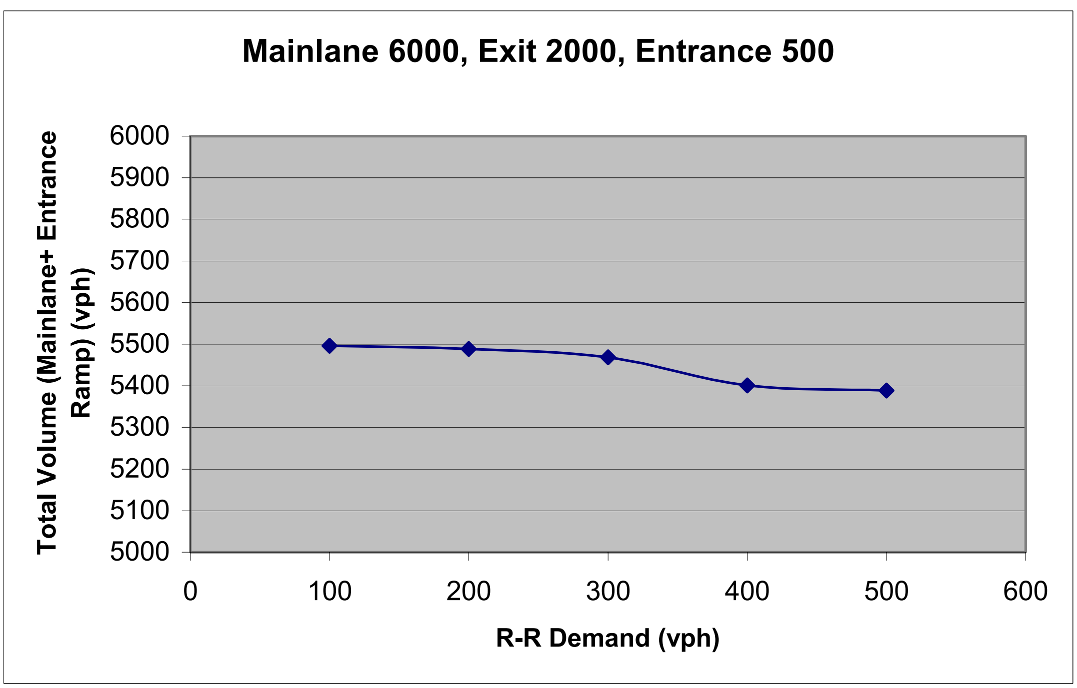

Figure C36 R-R Demand vs Total Volume (V) For Scenario 12

Scenarios 13:

- Total mainlane volume is 6500 vph
- Exit ramp volume is 800 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6500	800	500	100	5454	629	496	102	5950	5030	0.845
6500	800	500	300	5325	664	496	294	5821	5249	0.902
6500	800	500	500	5238	685	492	492	5730	5538	0.966

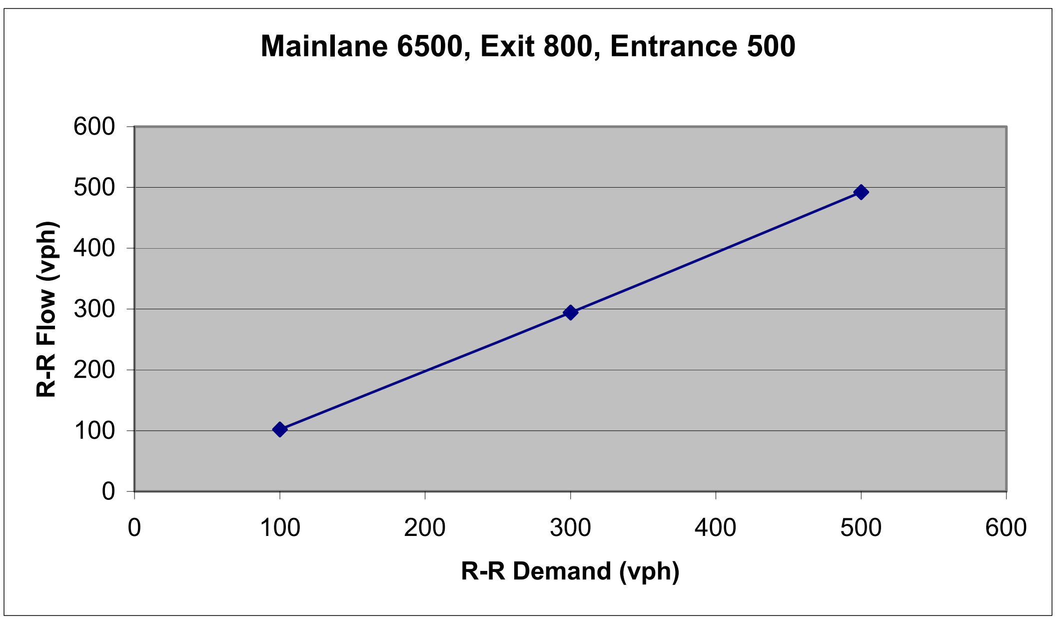

Figure C37 R-R Demand vs R-R Flow For Scenario 13

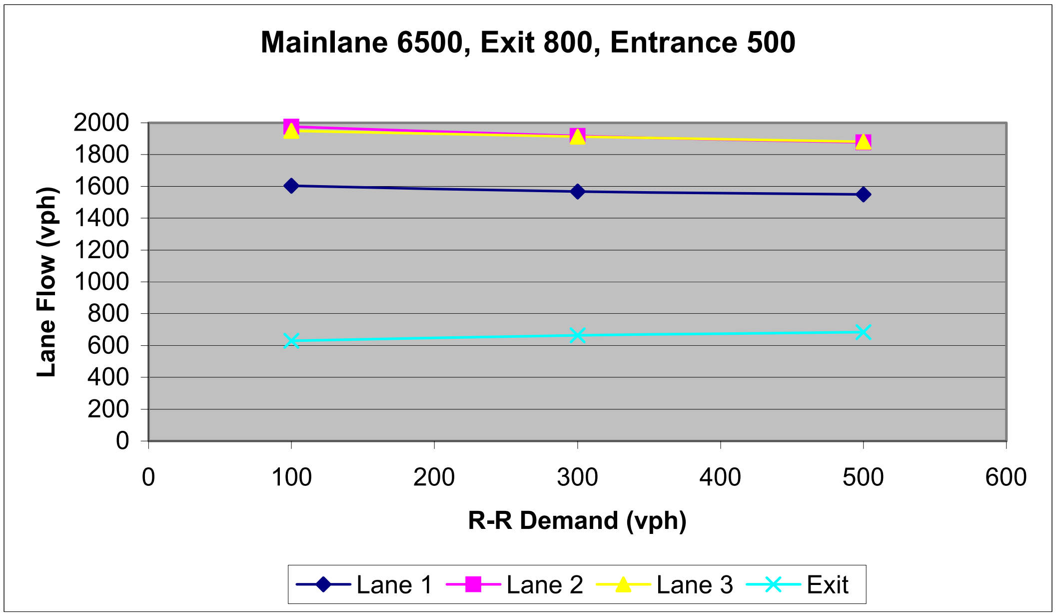

Figure C38 R-R Demand vs Lane Flow For Scenario 13

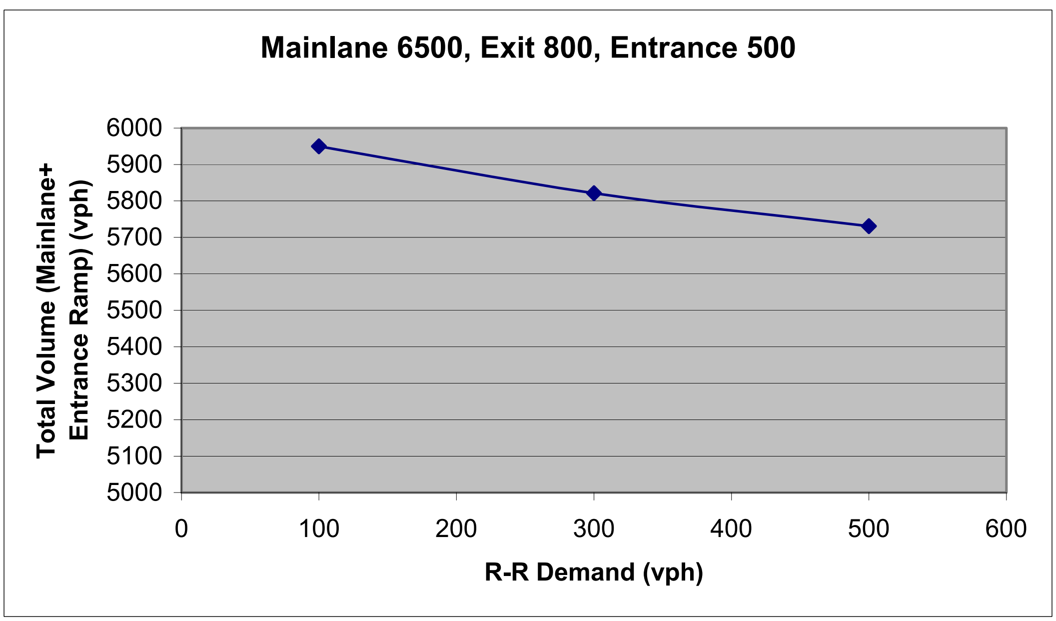

Figure C39 R-R Demand vs Total Volume (V) For Scenario 13

Scenarios 14:

- Total mainlane volume is 6500 vph
- Exit ramp volume is 1200 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6500	1200	500	100	5305	980	493	99	5798	4524	0.780
6500	1200	500	300	5249	953	495	300	5744	4896	0.852
6500	1200	500	500	5121	966	492	492	5613	5138	0.915

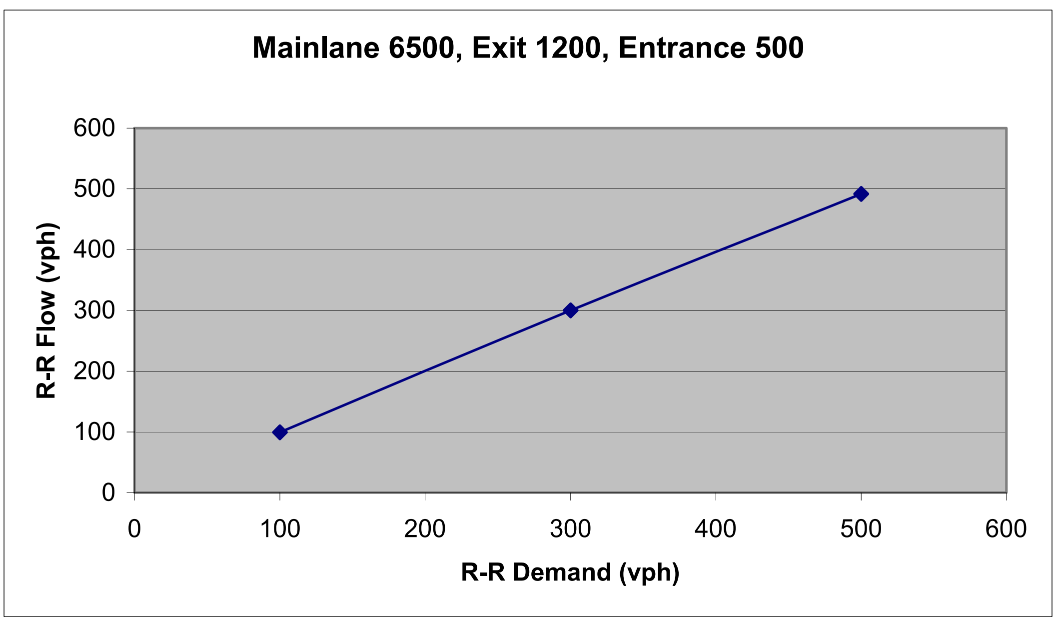

Figure C40 R-R Demand vs R-R Flow For Scenario 14

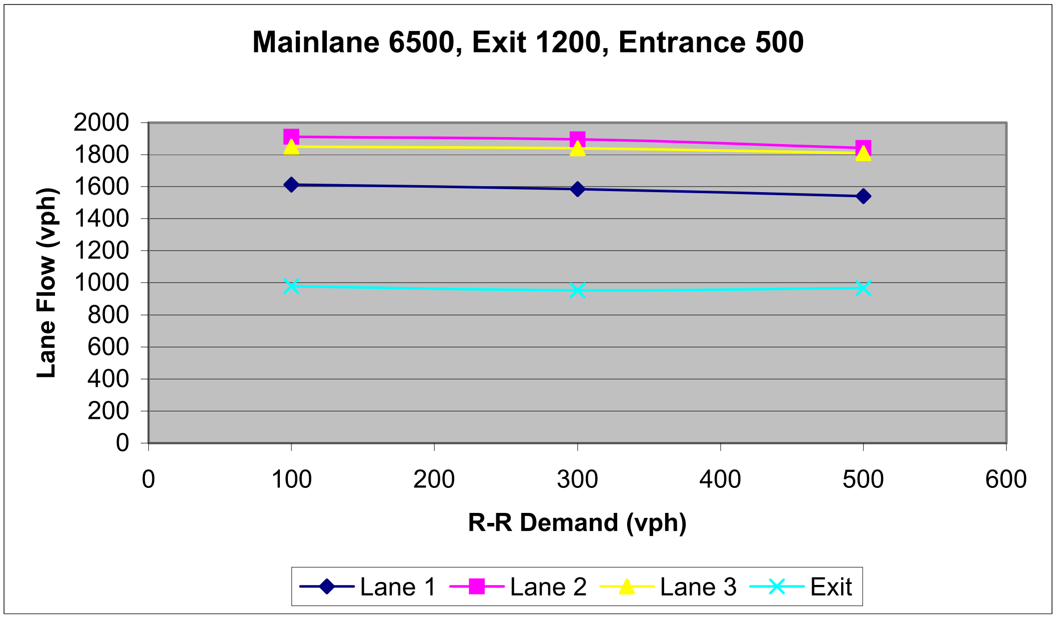

Figure C41 R-R Demand vs Lane Flow For Scenario 14

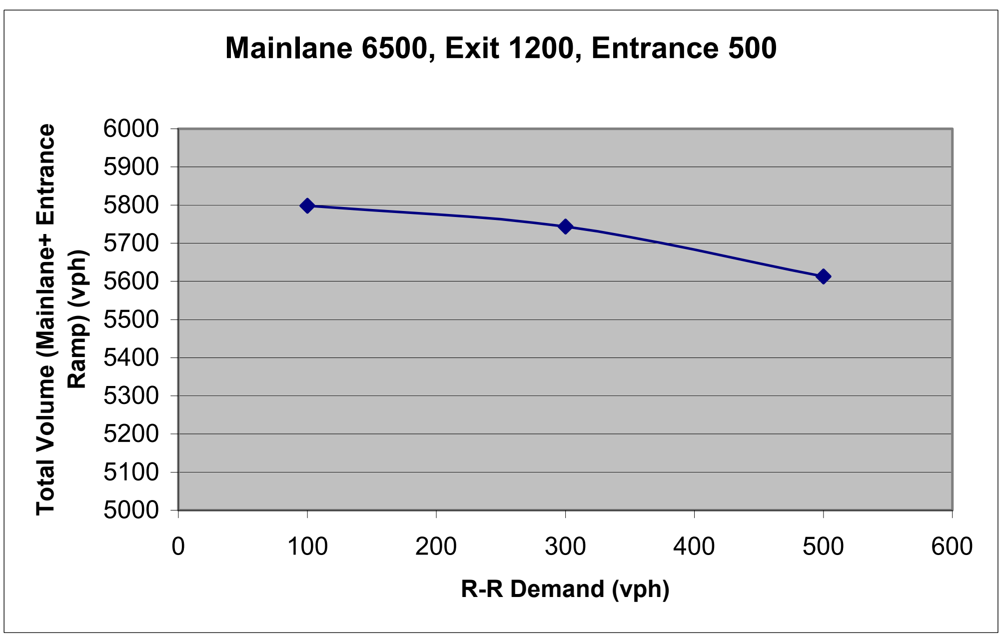

Figure C42 R-R Demand vs Total Volume (V) For Scenario 14

Scenarios 15:

- Total mainlane volume is 6500 vph
- Exit ramp volume is 1400 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6500	1600	500	100	5231	1190	492	105	5724	4252	0.743
6500	1600	500	300	5100	1213	493	297	5593	4482	0.801
6500	1600	500	500	5038	1240	491	491	5529	4779	0.864

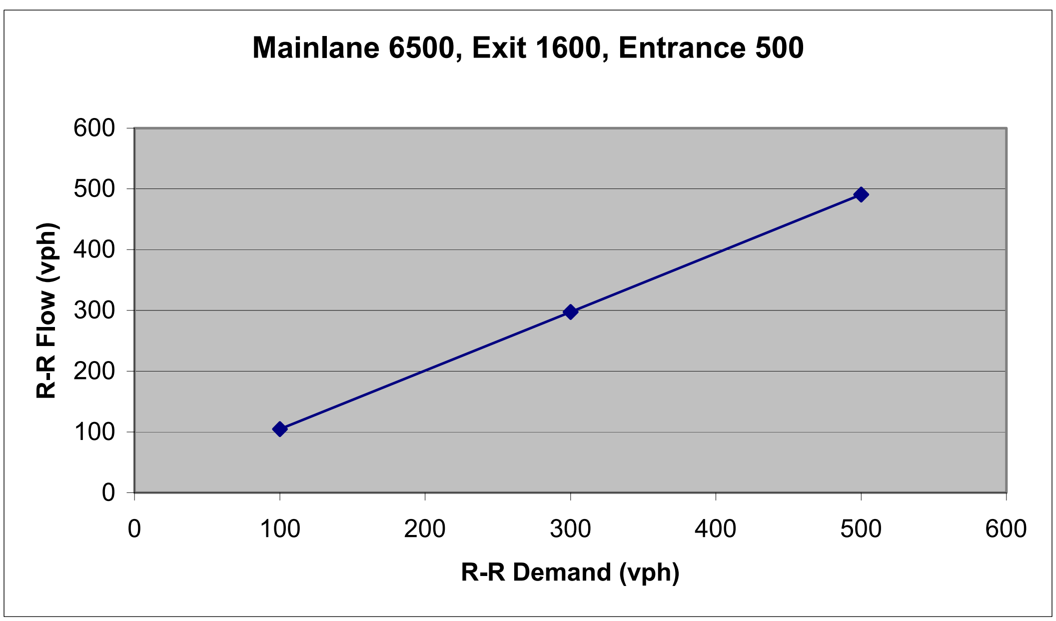

Figure C43 R-R Demand vs R-R Flow For Scenario 15

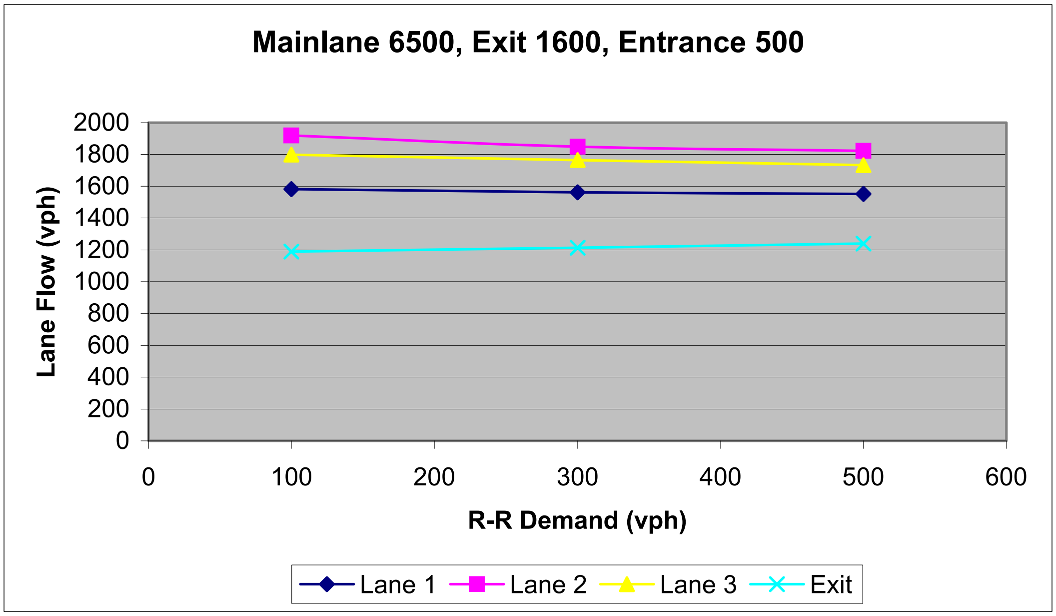

Figure C44 R-R Demand vs Lane Flow For Scenario 15

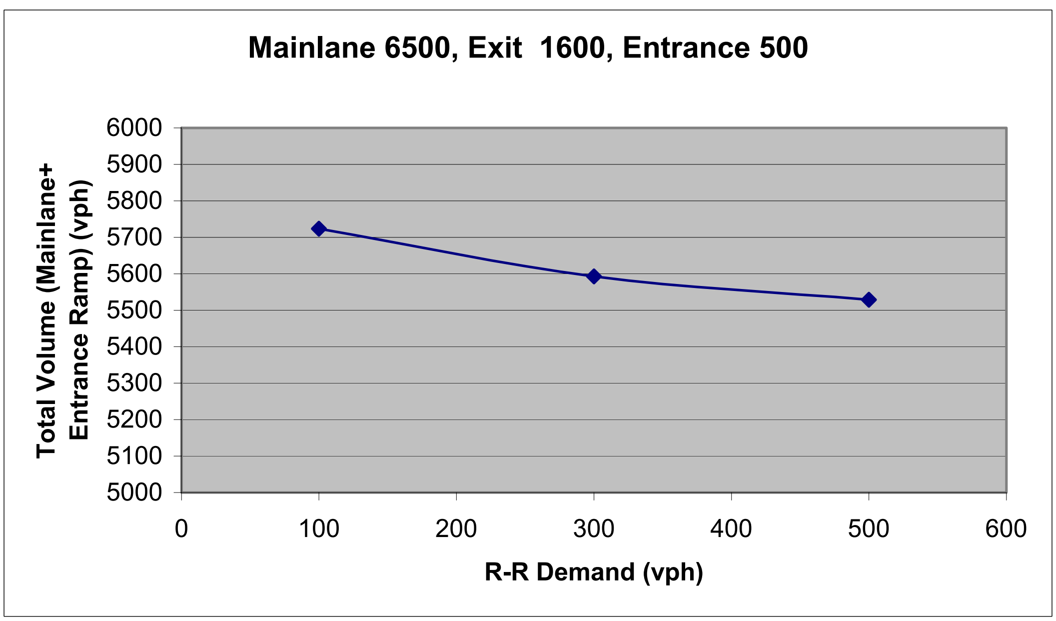

Figure C45 R-R Demand vs Total Volume (V) For Scenario 15

Scenarios 16:

- Total mainlane volume is 6500 vph
- Exit ramp volume is 2000 vph
- R-R demand is from 100-500
- Entrance ramp is from 500 vph

At this scenario, system is at capacity.

Input Volume (Demand)				Actual Volume (Real)						
Mainlane	Exit	Entrance	R-R	Mainlane	Exit	Entrance	R-R	V	Vw	Vr
6500	2000	500	100	5054	1447	494	105	5548	3817	0.688
6500	2000	500	300	5022	1486	494	306	5516	4148	0.752
6500	2000	500	500	4917	1491	494	494	5411	4414	0.816

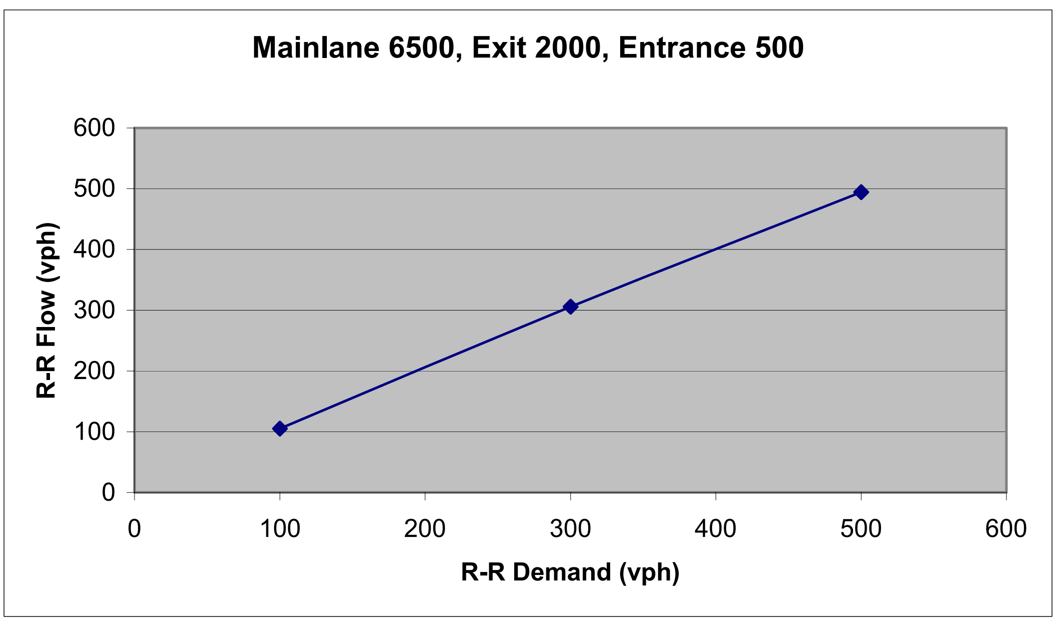

Figure C46 R-R Demand vs R-R Flow For Scenario 16

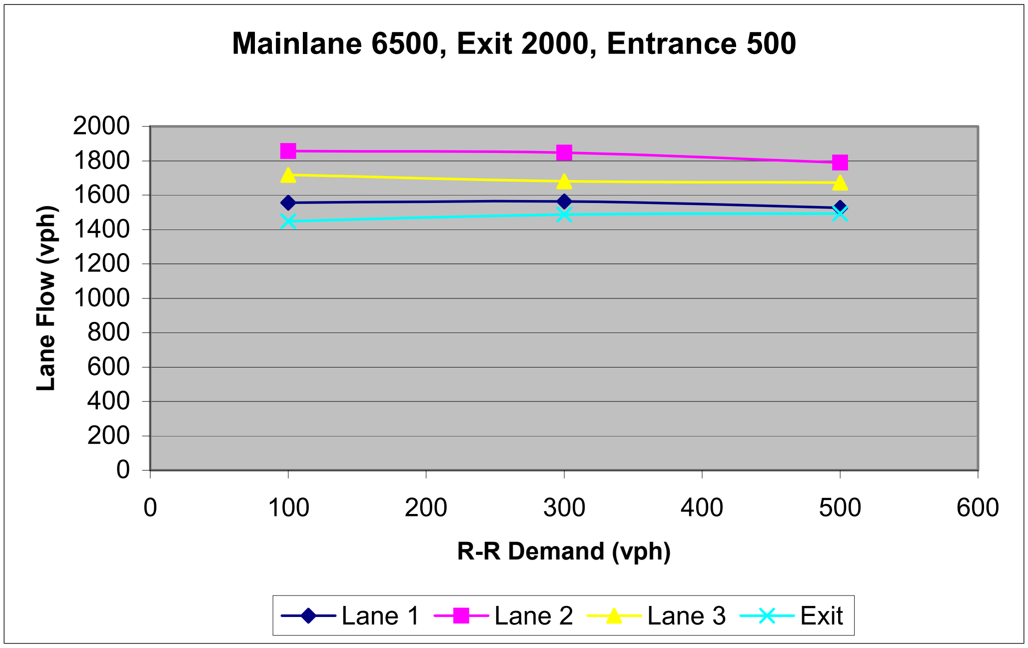

Figure C47 R-R Demand vs Lane Flow For Scenario 16

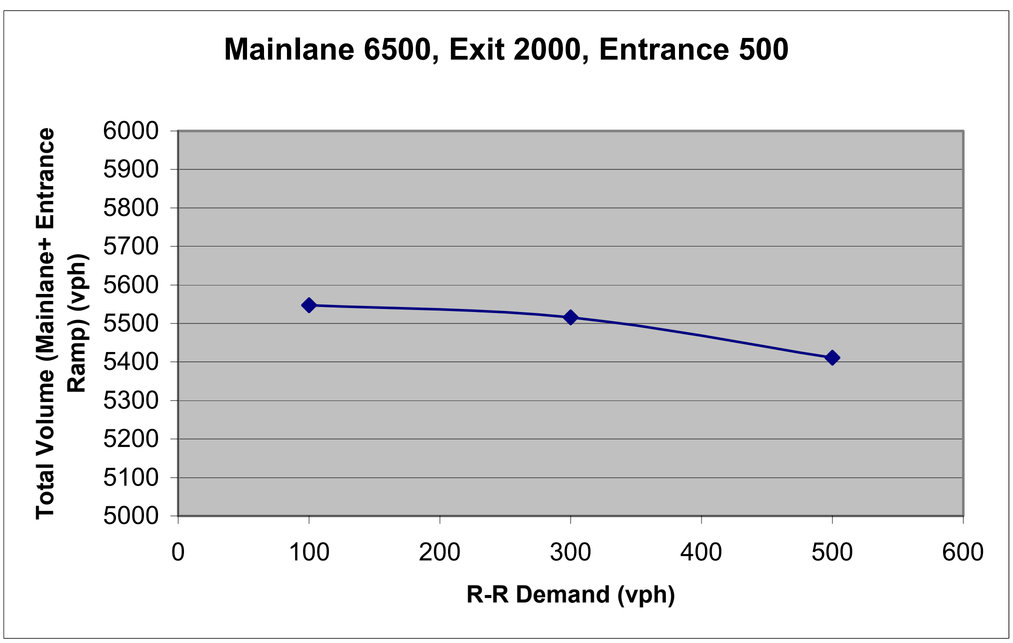

Figure C48 R-R Demand vs Total Volume (V) For Scenario 16

APPENDIX D

SPSS OUTPUT FOR STEPWISE ANALYSIS

Regression

Variables Entered/Removed(a)			
Model	**Variables Entered**	**Variables Removed**	**Method**
1	Mainlane	.	Forward (Criterion: Probability-of-F-to-enter <= .050)
2	Exit	.	Forward (Criterion: Probability-of-F-to-enter <= .050)
3	RR	.	Forward (Criterion: Probability-of-F-to-enter <= .050)
a Dependent Variable: Capacity			

Model Summary				
Model	**R**	**R Square**	**Adjusted R Square**	**Std. Error of the Estimate**
1	.813(a)	.662	.659	105.48133
2	.951(b)	.904	.903	56.26827
3	.990(c)	.980	.980	25.59056
a Predictors: (Constant), Mainlane				
b Predictors: (Constant), Mainlane, Exit				
c Predictors: (Constant), Mainlane, Exit, RR				

ANOVA(d)						
Model		**Sum of Squares**	**df**	**Mean Square**	**F**	**Sig.**
1	**Regression**	3068128.460	1	3068128.460	275.754	.000(a)
	Residual	1568809.903	141	11126.311		
	Total	4636938.364	142			
2	**Regression**	4193681.776	2	2096840.888	662.275	.000(b)
	Residual	443256.588	140	3166.118		
	Total	4636938.364	142			
3	**Regression**	4545910.468	3	1515303.489	2313.875	.000(c)
	Residual	91027.895	139	654.877		
	Total	4636938.364	142			
a Predictors: (Constant), Mainlane						
b Predictors: (Constant), Mainlane, Exit						
c Predictors: (Constant), Mainlane, Exit, RR						
d Dependent Variable: Capacity						

Coefficients(a)

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	3522.211	119.505		29.473	.000
	Mainlane	.422	.025	.813	16.606	.000
2	(Constant)	4306.190	76.111		56.578	.000
	Mainlane	.325	.015	.627	22.422	.000
	Exit	-.285	.015	-.527	-18.855	.000
3	(Constant)	5113.520	49.092		104.163	.000
	Mainlane	.187	.009	.361	21.085	.000
	Exit	-.317	.007	-.585	-45.182	.000
	RR	-.262	.011	-.373	-23.192	.000
a Dependent Variable: Capacity						

Excluded Variables(d)

Model		Beta In	t	Sig.	Partial Correlation	Collinearity Statistics
						Tolerance
1	Exit	-.527(a)	-18.855	.000	-.847	.874
	RR	-.231(a)	-3.717	.000	-.300	.568
	WeavingVolume	.163(a)	3.353	.001	.273	.947
2	RR	-.373(b)	-23.192	.000	-.891	.547
	WeavingVolume	-.404(b)	-23.180	.000	-.891	.465
3	WeavingVolume	12.880(c)	1.303	.195	.110	1.44E-006
a Predictors in the Model: (Constant), Mainlane						
b Predictors in the Model: (Constant), Mainlane, Exit						
c Predictors in the Model: (Constant), Mainlane, Exit, RR						
d Dependent Variable: Capacity						

APPENDIX E

DENSITY FOR ALL RUNNING SCENARIOS

Input Volume (Demand)						Density			
Mainlane	Exit	Entrance	R-R	V	Capacity	Lane 1	Lane 2	Lane 3	Weaving Section
3500	1000	1000	200	4453	No	26	28	34	30
3500	1000	1000	400	4449	No	26	29	34	30
3500	1000	1000	600	4455	No	25	29	35	30
3500	1000	1000	800	4443	No	25	29	35	30
3500	1000	1000	1000	4453	No	25	30	36	30
3500	1500	1000	200	4452	No	26	27	38	31
3500	1500	1000	400	4457	No	25	27	38	30
3500	1500	1000	600	4465	No	25	28	39	30
3500	1500	1000	800	4456	No	25	28	39	31
3500	1500	1000	1000	4445	No	24	29	40	31
3500	2000	1000	200	4450	No	26	26	45	32
3500	2000	1000	400	4440	No	24	26	44	31
3500	2000	1000	600	4452	No	27	28	49	35
3500	2000	1000	800	4449	No	27	30	52	36
3500	2000	1000	1000	4440	No	24	27	47	33
4000	1000	1000	200	4963	No	30	33	39	34
4000	1000	1000	400	4938	No	30	33	39	34
4000	1000	1000	600	4947	No	30	33	39	34
4000	1000	1000	800	4950	No	29	33	40	34
4000	1000	1000	1000	4938	No	29	34	40	34
4000	1500	1000	200	4950	No	30	32	42	34
4000	1500	1000	400	4933	No	29	32	42	34
4000	1500	1000	600	4927	No	29	32	42	34
4000	1500	1000	800	4937	No	30	32	43	35
4000	1500	1000	1000	4926	No	30	34	45	36
4000	2000	1000	200	4925	No	32	32	54	39
4000	2000	1000	400	4960	No	32	32	52	39
4000	2000	1000	600	4946	No	31	33	52	39
4000	2000	1000	800	4918	No	30	33	49	37
4000	2000	1000	1000	4887	No	31	33	49	38
4500	1000	1000	200	5441	No	35	37	43	39
4500	1000	1000	400	5401	No	34	37	43	38
4500	1000	1000	600	5398	No	36	39	44	40
4500	1000	1000	800	5420	No	37	40	45	41

4500	1000	1000	1000	5354	No	37	40	44	40
4500	1500	1000	200	5424	No	36	38	47	40
4500	1500	1000	400	5420	No	36	38	47	40
4500	1500	1000	600	5364	Yes	36	38	46	40
4500	1500	1000	800	5302	Yes	37	39	49	41
4500	1500	1000	1000	5334	Yes	36	39	48	41
4500	2000	1000	200	5276	Yes	36	36	53	42
4500	2000	1000	400	5212	Yes	36	37	53	42
4500	2000	1000	600	5194	Yes	36	37	53	42
4500	2000	1000	800	5164	Yes	36	37	54	42
4500	2000	1000	1000	5138	Yes	36	37	52	41
4800	1500	1000	200	5516	Yes	39	39	48	42
4800	1500	1000	400	5475	Yes	38	40	48	42
4800	1500	1000	600	5379	Yes	38	39	48	42
4800	1500	1000	800	5300	Yes	36	39	48	41
4800	1500	1000	1000	5273	Yes	37	39	47	41
4800	2000	1000	200	5343	Yes	37	37	52	42
4800	2000	1000	400	5266	Yes	37	38	53	43
4800	2000	1000	600	5236	Yes	36	37	53	42
4800	2000	1000	800	5219	Yes	36	38	52	42
4800	2000	1000	1000	5156	Yes	36	38	52	42
5000	800	1000	200	5733	Yes	40	42	46	43
5000	800	1000	400	5655	Yes	40	42	46	42
5000	800	1000	600	5558	Yes	39	41	45	42
5000	800	1000	800	5524	Yes	39	42	45	42
5000	1000	1000	200	5651	Yes	40	41	46	42
5000	1000	1000	400	5634	Yes	39	41	46	42
5000	1000	1000	600	5528	Yes	39	41	46	42
5000	1000	1000	800	5448	Yes	38	41	45	41
5000	1000	1000	1000	5368	Yes	38	40	45	41
5000	1500	1000	200	5542	Yes	39	39	48	42
5000	1500	1000	400	5463	Yes	38	39	48	42
5000	1500	1000	600	5448	Yes	38	40	48	42
5000	1500	1000	800	5376	Yes	37	39	48	41
5000	1500	1000	1000	5264	Yes	37	39	47	41
5500	1000	1000	200	5680	Yes	40	41	46	42

5500	1000	1000	400	5624	Yes	39	41	46	42
5500	1000	1000	600	5503	Yes	39	41	46	42
5500	1000	1000	800	5444	Yes	38	41	45	41
5500	1000	1000	1000	5399	Yes	38	41	45	41
5500	1200	1000	200	5631	Yes	39	41	46	42
5500	1200	1000	400	5548	Yes	39	40	47	42
5500	1200	1000	600	5479	Yes	38	40	46	42
5500	1200	1000	800	5402	Yes	38	40	46	41
5500	1200	1000	1000	5351	Yes	38	40	45	41
5500	1500	1000	200	5553	Yes	39	40	47	42
5500	1500	1000	400	5483	Yes	38	40	48	42
5500	1500	1000	600	5437	Yes	38	40	47	42
5500	1500	1000	800	5361	Yes	37	39	47	41
5500	1500	1000	1000	5274	Yes	37	39	47	41
5500	2000	1000	200	5419	Yes	37	38	50	42
5500	2000	1000	400	5320	Yes	37	38	50	42
5500	2000	1000	600	5302	Yes	37	38	51	42
5500	2000	1000	800	5246	Yes	36	38	51	42
5500	2000	1000	1000	5211	Yes	36	38	50	41
6000	800	1000	200	5784	Yes	40	42	46	42
6000	800	1000	400	5692	Yes	40	42	46	42
6000	800	1000	600	5554	Yes	39	41	45	42
6000	800	1000	800	5507	Yes	39	41	45	42
6000	1000	1000	100	5783	Yes	40	42	46	43
6000	1000	1000	200	5739	Yes	40	42	46	42
6000	1000	1000	300	5682	Yes	39	41	46	42
6000	1000	1000	400	5637	Yes	39	41	46	42
6000	1000	1000	500	5595	Yes	39	41	46	42
6000	1000	1000	600	5521	Yes	39	41	46	42
6000	1000	1000	700	5518	Yes	39	41	45	42
6000	1000	1000	800	5449	Yes	38	41	45	41
6000	1000	1000	900	5434	Yes	38	41	45	41
6000	1000	1000	1000	5373	Yes	38	41	45	41
6000	1200	1000	200	5669	Yes	40	41	46	42
6000	1200	1000	400	5587	Yes	39	41	46	42
6000	1200	1000	600	5505	Yes	38	41	46	42

6000	1200	1000	800	5422	Yes	38	41	46	42
6000	1200	1000	1000	5359	Yes	38	40	46	41
6000	1500	1000	200	5590	Yes	39	40	47	42
6000	1500	1000	400	5489	Yes	38	40	47	42
6000	1500	1000	600	5453	Yes	38	40	47	42
6000	1500	1000	800	5361	Yes	37	40	47	41
6000	1500	1000	1000	5298	Yes	37	40	47	41
6000	1750	1000	200	5514	Yes	39	39	48	42
6000	1750	1000	400	5463	Yes	38	39	48	42
6000	1750	1000	600	5376	Yes	37	39	48	42
6000	1750	1000	800	5312	Yes	37	39	48	41
6000	1750	1000	1000	5245	Yes	37	39	48	41
6000	2000	1000	200	5441	Yes	38	39	49	42
6000	2000	1000	400	5404	Yes	38	39	49	42
6000	2000	1000	600	5316	Yes	37	38	50	42
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6500	1000	1000	400	5651	Yes	39	42	46	42
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6500	1000	1000	800	5486	Yes	38	41	45	42
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6500	1500	1000	200	5634	Yes	39	40	47	42
6500	1500	1000	400	5487	Yes	38	40	47	42
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6500	1500	1000	1000	5302	Yes	37	39	47	41
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5000	800	500	300	5446	No	35	37	41	38
5000	800	500	500	5427	No	34	37	42	38
5000	1200	500	100	5438	No	34	36	43	38
5000	1200	500	300	5438	No	35	37	44	39
5000	1200	500	500	5427	No	35	37	44	39
5000	1600	500	100	5432	No	35	36	46	39
5000	1600	500	300	5393	No	36	38	48	41
5000	1600	500	500	5428	No	36	38	49	41

5000	2000	500	100	5327	Yes	38	38	55	44
5000	2000	500	300	5337	Yes	38	39	56	44
5000	2000	500	500	5276	Yes	37	38	54	43
5500	800	500	100	5863	Yes	41	42	46	43
5500	800	500	300	5793	Yes	41	43	46	43
5500	800	500	500	5755	Yes	40	43	46	43
5500	1200	500	100	5727	Yes	40	41	48	43
5500	1200	500	300	5705	Yes	40	42	48	43
5500	1200	500	500	5616	Yes	40	42	47	43
5500	1600	500	100	5588	Yes	40	40	50	43
5500	1600	500	300	5591	Yes	39	40	50	43
5500	1600	500	500	5515	Yes	39	40	50	43
5500	2000	500	100	5449	Yes	39	39	53	44
5500	2000	500	300	5391	Yes	38	39	53	43
5500	2000	500	500	5328	Yes	38	38	53	43
6000	800	500	100	5902	Yes	42	43	47	44
6000	800	500	200	5908	Yes	42	43	47	44
6000	800	500	300	5834	Yes	41	43	46	44
6000	800	500	400	5790	Yes	41	43	47	44
6000	800	500	500	5753	Yes	40	43	46	43
6000	1200	500	100	5786	Yes	41	42	47	43
6000	1200	500	200	5748	Yes	41	42	47	43
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6000	1200	500	400	5657	Yes	40	42	48	43
6000	1200	500	500	5623	Yes	40	42	47	43
6000	1600	500	100	5644	Yes	40	41	49	43
6000	1600	500	200	5619	Yes	40	41	49	43
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6000	1600	500	400	5547	Yes	39	41	49	43
6000	1600	500	500	5489	Yes	39	40	49	42
6000	2000	500	100	5496	Yes	39	39	52	43
6000	2000	500	200	5489	Yes	39	40	52	44
6000	2000	500	300	5468	Yes	39	39	52	43
6000	2000	500	400	5401	Yes	38	39	52	43
6000	2000	500	500	5388	Yes	38	39	52	43
6500	800	500	100	5950	Yes	42	44	47	44

6500	800	500	300	5821	Yes	41	43	47	44
6500	800	500	500	5730	Yes	41	43	46	43
6500	1200	500	100	5798	Yes	41	42	48	44
6500	1200	500	300	5744	Yes	41	42	47	43
6500	1200	500	500	5613	Yes	40	41	47	43
6500	1600	500	100	5724	Yes	40	42	48	43
6500	1600	500	300	5593	Yes	39	41	48	43
6500	1600	500	500	5529	Yes	39	41	49	43
6500	2000	500	100	5548	Yes	39	40	50	43
6500	2000	500	300	5516	Yes	39	40	50	43
6500	2000	500	500	5411	Yes	38	39	51	43

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