IMPACT OF ARTERIAL CHARACTERISTICS ON THE OPERATIONAL PERFORMANCE OF TRAFFIC ON URBAN ARTERIALS NEAR DIAMOND INTERCHANGES

A Dissertation

by

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ABSTRACT

Urban arterials near freeway interchanges are vital segments of road infrastructure. While freeways have high mobility and low access, urban arterials have medium mobility and medium access. Designing facilities that can efficiently connect these two types of roads is complex. There is an increasing need in the profession to have an in-depth understanding of the operational performance of urban arterials, while no clear and straightforward recommendations exist to enumerate the impact of various design and operational elements on the operational performance of traffic on urban arterials.

This dissertation suggests a step-by-step methodology to (a) comprehensively study traffic flow on urban arterials located in the vicinity of diamond interchanges, (b) quantify the impact of the urban street characteristics on lane changing, and subsequently, (c) study how lane change behavior impacts travel time.

This research studied two dependent variables, and, as a result, two final statistical models were built to demonstrate the relationship between the contributing factors and the dependent variables: travel time and lane changing.

PTV-VISSIM software was the main simulation tool used in this study. Each micro-simulation model was validated and calibrated for base conditions for each output: travel time (TT) and the number of lane changes (NLC). The author used factorial simulation and a hashing method to create all possible simulations by using the Python programming language. The final outputs were filtered to remove any redundant

scenarios. After compiling inputs and outputs in a filtered and adjusted database, the author used the R programming language to conduct linear regression analyses. The results show that the main contributing factors to the number of lane changes aside from the through, left, and right turning volumes are the number of driveways, distance of the right turning lane to the upstream intersection, and the number of median openings. The results indicated that adding each driveway contributed to 54 more lane changes, while each median opening added 79 more lane changes per hour. Also, placing the start of the right turning lane closer to the first signalized intersection lead to fewer lane changes on the segment.

The number of lane changes, the density of the driveways, segment length, and being located upstream to the freeway on-ramp contributed to higher travel times.
The findings of this research provide designers and urban planners a deeper insight into the impact of design attributes on the operational performance of urban arterials near interchanges.

DEDICATION

To my husband, Yashar Talebi, for being my best friend in the past thirteen years and supporting me in every walk of life.

To my parents, for all of their unconditional love and patience and teaching me to aim high.

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NOMENCLATURE

NLC	Number of Lane Changes
TT	Travel Time
NCHRP	National Cooperative Highway Research Program
HCM	Highway Capacity Manual
TRB	Transportation Research Board
VMT	Vehicle Miles Traveled
TTC	Time to Collision
PICUD	Potential Index for Collision with Urgent Deceleration

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CHAPTER I

INTRODUCTION

Urban arterials in the vicinity of diamond interchanges are key elements of the transportation network. At these locations, vehicles and drivers experience a transition between two significantly different driving environments and need to make a safe transition between the two.

To reach their destinations, vehicles originating from the freeway distribute across different lanes of urban arterials by conducting lane-change maneuvers. Lane-changing has been reported to account for 9% of all police-reported crashes (1) and contribute to a reduction in the capacity of freeways (2). Although there are numerous freeway studies, there are no studies or resources that quantify the interaction of vehicles and lane changes on urban arterials, and much less in the vicinity of diamond interchanges. To thoroughly understand these interactions, it is important to identify the factors that contribute to operational performance of traffic at urban arterials near interchanges.

There are various design and operational factors that may influence traffic operation on arterials. Some of these factors are:

- The driveway or the group of driveways that provide access to the land uses adjacent to the arterials,
- The distance between signalized intersections,
- Median type, and

- Traffic control type (stop controlled vs. yield controlled) of the right turning movements from the freeway exit ramp to the crossroad.

No clear and straightforward recommendations exist that enumerate the impact of various design elements on the operational performance of traffic on urban arterials. The author is aware of one freeway-based study that assessed lane changes with an approach similar to what is being proposed herein. The study that analyzed lane-changing maneuvers on freeways included a sensitivity analysis on the output of the simulation results to identify the contributing factors, but it did not quantify the impacts of the factors.

The present research provides a step-by-step methodology to (a) study traffic flow on urban arterials located in the vicinity of diamond interchanges, (b) to quantify the impact of the urban street characteristics on lane changing, and subsequently (c) to study how lane change maneuvers impact travel time.

This research provides details on systematically studying these facilities and presents a new performance measure to use when analyzing their performance. This study hypothesized that the number of lane changing maneuvers on an urban arterial impacts travel time and proved it through statistical analysis.

The results of this study will help transportation professionals identify the most important contributing factors and understand their impacts on arterial traffic flow near freeway interchanges. Designers can improve the operational performance of existing facilities by making the most impactful improvements and design new facilities more efficiently.

After the introduction (Chapter I), this dissertation includes a problem statement in Chapter II before explaining the research statement in Chapter III. Chapter IV is a review of the background literature and research related to this topic. In Chapter V, the research approach is explained in detail, including the steps taken to conduct the study and prepare the reader for the analysis section. Chapter VI presents the analysis steps and results of the study and a conclusion is presented in chapter VII.

CHAPTER II

PROBLEM STATEMENT

Lane changing maneuvers are difficult to measure but have a sizable impact on traffic flow. Previous research has not focused on studying this phenomenon for urban arterials just downstream or upstream of freeway exit and entry points. Vehicles that travel from the freeway off-ramps into the arterial traffic flow (see segments 2 and 4 in Figure 1) increase the number of lane changing maneuvers. A similar situation holds for vehicles that travel on the arterials (segments 1 and 3 in Figure 1) and change lanes to get to the freeway on-ramps.

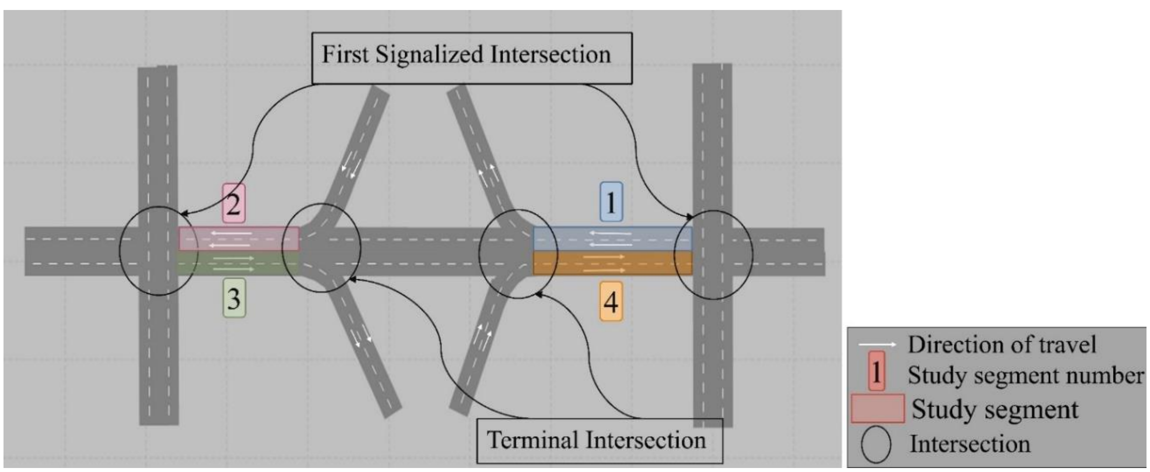

Figure 1 Generalized Schematic Illustration of Study Sites

Lane changing maneuvers contribute to increased crashes and decreased capacity on transportation networks. Various studies acknowledge that lane changing maneuvers impact freeway delays (3) while design and operational elements impact lane changing behavior on freeways (4). This research is an initial investigation into how design elements impact travel on arterials near freeway interchanges. In this study, “urban arterial” is defined to represent the segments of an urban arterial that are located immediately upstream or downstream of freeway entrance or exit points and extend to the first signalized intersection (segments 1 to 4 in Figure 1).

CHAPTER III

RESEARCH OBJECTIVES

The objective of this research is to investigate the impact of design and operational variables on lane-changing maneuvers, and subsequently study how these lane changes affect the operational performance of traffic — measured by travel time — on urban arterials near diamond interchanges. Therefore, this research has two goals. The first goal is to study how arterial characteristics influence lane changing. The second goal is to evaluate the impact of lane changing on travel time as a performance measure. To reach these objectives, ten study sites were selected and traffic data from each of these sites was collected. VISSIM was then used to model each study site. Then alternative scenarios were designed and simulated using a factorial design approach. Using the data obtained from the VISSIM models, statistical analysis was used to quantify the impacts of arterial characteristics on lane changing and travel time.

CHAPTER IV

BACKGROUND

Urban development near interchange locations usually occurs faster than in other parts of cities, and as a result, many businesses such as restaurants, shopping centers, retail stores, and hotels compete to locate themselves close to an interchange (5). Vehicles that travel from the freeway off-ramps into the traffic flow on the arterials (see segments 2 and 4 in Figure 1) increase the number of lane-changing maneuvers. Similar situations hold for vehicles that travel on the arterials (segments 1 and 3 in Figure 1) and change lanes to get on the freeway ramps.

One study found that lane changing maneuvers impact freeway delays (3), and design and operational elements impact lane changing behavior on freeways (4). The research conducted for this dissertation is an initial examination of how design elements may impact travel on arterials near freeway interchanges. In this study, “urban arterial” is defined to represent the segments of an urban arterial located immediately upstream or downstream of freeway entrance or exit points and extends to the first signalized intersection (segments 1 to 4 in Figure 1).

IV. 1 **Importance of Design on Access Management**

There are several important access management design considerations when building arterial roads. These include:

- How far the interchange is placed from the signalized intersections?

- How far should the first access point be permitted?
- How many access points should a shopping plaza have?
- Should the right turning movement at the start of the segment be stop-controlled or yield controlled?

Transportation professionals seek answers to these questions (5). Having research-based answers to questions of this type helps design and plan facilities that facilitate mobility and access on the urban infrastructure.

The early trend (1960's and 1970's) for studying different road features was to measure the cost based on injuries, deaths, or other losses. Example studies from this line of research considered the number of lanes (6), (7), lane width (7), surface type (8), cross slope (9), and surface friction (10). Cribbins et al. (11) evaluated the median width. Foley (12) investigated the frequency of median openings and the type of median barriers. Auxiliary lanes such as left-turn lanes and transition lanes (for speed adjustment) as well as shoulder and roadside characteristics, vertical alignment, traffic control devices have also been examined. Avelar et al. (13) categorized parameters that influenced arterial highway safety into the following seven categories:

- driveway spacing,
- proximity to interchange and intersection,
- traffic signal control,
- driveway type,
- roadway characteristics,
- land use and

- median type.

While the arterial highways in Avelar's study were located in rural locations, their results indicate that access management elements impact travel. Their results showed that segment length, Average Annual Daily Traffic (AADT), median type (Two Way Left Turn Lane- TWLTL) and having four lanes on the rural highway were the most significant factors contributing to highest number of crashes.

Urban roads account for roughly 25 to 27 percent of all roads, while they carry about two-thirds of vehicle miles traveled (VMT) in the U.S. (14)–(16). The Transportation Research Board (TRB) special report 260, "Strategic Highway Research," suggests that the VMT is expected to increase in the coming years (17). This prediction and similar predictions emphasize the importance of optimal management of the transportation network since there is little room to expand the physical infrastructure.

IV. 2 **Impact of Lane Changing on Traffic Flow**

Lane-changing impacts traffic flow in different ways. In some cases, vehicles adjust their speeds to seek an acceptable gap in the target lane, which causes disturbance in the traffic flow. In other cases, where there is adequate gap, lane change may takes place as a response to surrounding features, such as a stopped vehicle on the shoulder or presence of a driveway. Design and operational characteristics impact the location and frequency of lane changes. Some studies have investigated how lane change maneuvers impact traffic flow. These studies are reviewed next.

While changing their lanes, vehicles take up a space in their present lane and also seek an available space in the adjacent lane. This means that lane changing vehicles use more space than vehicles that don't change lanes. Jin incorporated kinematic wave theory to study the impacts of lane changing on traffic flow. He introduced a lane changing intensity factor and assumed that when a vehicle changes its lane, it needs to have two spaces dedicated to it (18). The fundamental equation of traffic flow has the form:

$$q = \rho v \tag{1}$$

Where:

q is the flow rate,

ρ is the density and

v is the speed.

In the presence of lane change maneuvers an intensity factor, ε, is introduced and the fundamental equation is written as:

$$q = (1 + \varepsilon)\rho V_{((1+\varepsilon)\rho)} \tag{2}$$

where $v = V(\rho)$. It can be seen that if ε exists, then lane changing impacts the traffic flow by increasing density.

In a study of single-point urban interchanges, Messer and Bonneson noted that the traffic flow on arterial cross-roads experiences turbulence due to the high volume of lane changes that occur on them (19). Although they reported the turbulence at arterial cross-roads, they did not quantify it. Numerous other studies have reported that lane changing can cause a reduction in capacity (2) and can have a direct influence on traffic safety (20).

From the first lane changing model introduced by Gipps (21) to the more recent ones like the MOBIL model (20), all models have studied lane changing in the context of the traveled way. These models regard lane changing as interactions among two or more vehicles and ignore the impact of design parameters on the recurrence and location of lane changing. As another example, Laval and Daganzo (22) proposed a hybrid approach (a combination of acceleration and kinematic wave model) to address the impact of lane changing on freeways where the studied lane changing maneuvers had the sole purpose of increasing speed. They calculated the number of lane changing maneuvers for each lane of the freeway and did not incorporate design and operational characteristics of the traveled way in their calculations.

Many researchers have studied the impact of lane changing on non-operational aspects of traffic. Some researchers used surrogate safety measures instead of crash data to evaluate the safety at different sites. As an example, Gettman (23) summarized surrogate safety measures.

More recently, Iliadi et al. (24) developed a crash prediction model for “type A” weaving sections (based on the HCM classification) on freeways. They developed a negative binomial regression model and showed that AADT (average annual daily traffic), length of weaving section, number of lanes and proportion of weaving vehicles impacted the number of crashes in weaving sections. Glad et al.(25), Golob et al.(26), Liu et al.(27), Park et al. (28), and other researchers have also studied the safety aspects of freeway weaving areas. Nevertheless, none of these studies has focused on the operational performance of traffic on arterials in the vicinity of interchanges. These studies also did not assess urban arterials in general or address the more detailed specifications of lane change maneuvers. Based on this review, it is clear that the lane changing on arterials connected to freeway ramp terminal intersection is an issue that should be examined.

IV. 3 Performance Measure Selection

A performance measure is a quantifiable indicator that measures the performance of a study object. Selecting a performance measure is a critical step in studying operational performance of a facility. In other words, performance measures quantify the performance of a system and should be selected such that they support the goals of the study. National Cooperative Highway Research Program (NCHRP) Synthesis 311 titled "Performance measures of operational effectiveness for highway segments and systems" lists performance measures that researchers commonly used for analysis (6). Based on the study, level of service (LOS) and traffic volume each account for 11% of performance measure chosen by researchers and engineers. VMT and travel time are also listed as other common performance measures accounting for 10%, and 8%, and of choice, respectively.

In this study, the objective was to evaluate the operational performance of urban arterials, with travel time being an important performance measure. On the other hand, lane changes contribute to delays and increase travel time. Based on this stipulation, the number of lane changes were studied first to find the relationship between the number of lane changes and design and operational factors. Then the number of lane changes were treated as an independent variable along with the original variables to model travel time. It is important to note that the HCM uses different performance measures to evaluate LOS on transportation facilities (29). LOS on urban streets (Chapter 15) is evaluated by calculating speed. At signalized intersections (chapter 16) and two way stop controlled intersections and T-intersections (Chapter 17), LOS is determined based on delay.

In this research, travel time and the number of lane changes were used to validate the simulation models and therefore the author used travel time estimation methods. In this research, probe vehicle data was used to collect actual travel time. The average travel time measurements from multiple probe vehicle runs were then compared to the model outputs as a method of validating the models.

Also, the number of lane changes was determined from the video files and compared to the number of lane changes extracted from the model outputs. The comparison for both travel time and the number of lane changes was done using the appropriate statistical methods and is explained in the model validation section in Chapter three.

IV. 4 **Variable Definitions**

Numerous variables can be used to describe to the performance of an urban arterial. The following paragraphs review and summarize studies that focus on the geometric and access related design parameters of urban arterials.

IV.4.1 ***Distance between the first signalized intersection and terminal intersection***

The first published literature on the impact of the length of a weaving section dates back to 1970 and was conducted by Cirilo (30). The study investigated the relationship between the number of merge and diverge movements to the number of crashes on urban freeways. She found that for the average daily traffic (ADT) values over 10,000 vehicles per day (vpd), the length of the weaving area had an increasing relationship with the number of crashes at the study sites. Cirillo also found that the percentage of merging and diverging vehicles contributed to the crash rate. The crash rate increased as the proportion of merging and diverging vehicles increased.

Another study indicates that signalized intersections located too close to the interchange can negatively impact the signal operation or cause excessive weaving maneuvers (5).

Layton (31) recommends placing the first intersection no closer than 1320 feet and the first access point (driveway or median) at least 750 feet away from the ramp entrance for two lane cross-roads. For four lane cross-roads, the minimum distance for the first major signalized intersection is 2640 feet, and the first access point is suggested to be placed at 750 feet while the suggested distance for the first median opening is 990 feet (31).

There are other recommendations by several other state agencies, but none of them specifies how the distance of the first intersection to the ramp location impacts the travel time on urban arterials near interchanges.

IV.4.2 ***Left turning vehicles***

Vehicles that enter the arterial from the freeway off ramp and intend to turn left at the next intersection can impede the through traffic on the arterials (25). Messer and Bonneson, who classified the arterial weaving maneuver types, acknowledged that left-turning vehicles have the most potential for impacting speed of the traffic flow due to increased number of deceleration, acceleration, and weaving periods (32). Table 1 shows Messer's and Bonneson's classification of an arterial weaving section. It can be seen from Table 1 that paths 1 and 2 are classified as weave type 1 which is the most difficult weaving type per their definition. The left-turning vehicles follow path 1 on the arterial segments studied in this research.

Table 1 Classification of Lane Change Maneuvers Present at an Urban Arterial near an Interchange. Adapted from Messer and Bonneson (32)

Path No.	**Maneuver (Entry to Exit)**	**Weave Type**	**Turns Made**	**Lane Changes**	
				2 Lane	**3 Lane**
1	Right Turn to Left Turn	1	2	1	2
2	Left Turn to Right Turn	1	2	1	2
3	Right Turn to Inside Through Lane	2	1	1	2
4	Left Turn to Outside Through Lane	2	1	1	2
5	Inside Through Lane to Right Turn	2	1	1	2
6	Outside Through Lane to Left Turn	2	1	1	2
7	Through to Adjacent Through Lane	3	0	1	1 or 2
Type 1: Maximum Difficulty, Type 3: Minimum Difficulty					

IV.4.3 ***Right turning vehicles***

The Green Book (Policy on Geometric Design of Highways and Streets) recommends providing right turning lanes to arterial streets to remove decelerating vehicles from the through lanes(33). It recommends different right turn lane lengths based on the design speed. Several studies have investigated the impact of right turning vehicles and the necessity of adding auxiliary right turning lanes at intersections. Alexander used regression to calculate delays experienced by through vehicles (34). He used 12 variables including grade, width, volume, percentage of right turning and left-turning traffic, v/c values and location (urban vs. rural) and concluded that the approach volume, right turning volume and average speed of non-delayed through vehicles are main factors that contribute to the total delay. The following equation shows Alexander's regression model.

$$Delay = -219.0 + 2.05X_5 + 0.37X_2 + 1.94X_6 \qquad (3)$$

where:

X_2= Approach Volume, vehicle per hour,

X_5= Number of right turning vehicles in approach direction per hour, and

X_6= average speed through the study area for a non-delayed through vehicles, feet per second.

In this study, the existence of a right turning lane and the distance of the beginning of the right turning lane from the terminal intersection were included as two of the independent variables to investigate their impact.

IV.4.4 *Access density*

Several researchers have studied the relationship between access density and travel time. Reily et al. studied the access density on rural and suburban highways and observed that for high volume freeways (>600 vph) introducing driveways, had an adverse impact on the through traffic speed (35). McShane conducted a similar study and observed drops in free flow speeds as number of driveways per mile increased (36). The Highway Capacity Manual, HCM, suggests a reduction in free flow speed on highways based on number of access points (29). Table 2 from the HCM shows reductions in free flow speed for different access density values.

Table 2 Reduction in Free Flow Speed on Highways Based on the Number of Access Points- Adapted from HCM (29)

Access Points/ Kilometer	Reduction if FFS (km/h)
0	0.0
6	4.0
12	8.0
18	12.0
24	16.0

Oregon DOT Highway design manual suggests allowing only right in – right out access to private properties along urban highways (37) but does not quantify the impact of access points on the safety or operation of traffic on the transportation network.

IV.4.5 *Median treatment*

The importance of median treatment on the operational performance of traffic has been well documented. Several guidelines suggest or require non-traversable medians on urban arterials. For example, ODOT Highway Design Manual (37) states that:

"Non-traversable medians are required on all new, multi-lane urban or rural expressways on new alignment. All other existing urban expressways should consider construction of a non-traversable median when projects are developed along these highways."

NCHRP Report 93 (38) suggests that when median openings are present, they should provide for a "natural" path for the vehicles that use the median to make turning movements. The guideline suggests using a bullet nose design for median openings since it minimized the encroachment on the adjacent lane. NCHRP Report 93 also suggests minimum distances between median openings based on arterial speed. These distances were calculated based on 10 mph difference between through and turning vehicles and deceleration rate of 8 ft/s^2. Table 3 lists absolute and desirable minimum distance values (in feet) for different arterial speeds.

Table 3 Minimum Distances between Median Openings- Adapted from NCHRP Report 93 (38)

Arterial Speed (mph)	**Minimum Distance (feet)**	
	Absolute Minimum (8.0 ft/s2 deceleration rate)	**Desirable Minimum (6.5 ft/s2 deceleration rate)**
25	140	390
30	190	370
35	240	460
40	300	530
45	360	670
50	430	780
55	510	910
25 feet should be added for every "stored" car in both options		

In this study medians were divided into three categories: all raised medians, raised medians with openings, and painted medians. Painted medians included two way left turning lanes and conventional painted medians.

IV.4.6 ***Corner clearance (first driveway)***

According to the AASHTO's "A Policy on Geometric Design of Highways and Streets" known as the green book (33) designers should refrain from placing driveways within the functional area of an at-grade intersection. The functional area of an intersection is the area around the intersection that includes interactions related to the intersection. The green book recommends including following areas in the upstream functional area of an intersection: 1) distance traveled during perception-reaction time, 2) deceleration distance to a stop, and 3) the amount of queuing at the intersection. The green book does not include recommendations for downstream functional areas of intersections but stopping sight distance is recommended by the access management manual (5) to determine the intersection downstream functional area.

The distance of the first driveway from the upstream intersection as well as the first driveway's volume were included as two of the independent variables.

IV. 5 **Data Collection**

Several different types of data were required to conduct this research including travel time, number of lane changes and all of the independent variables. Barceló et al mention over 15 sources for data collection such as infrared detectors, radar detectors, video cameras, probe vehicles, surveys, and mobile data (39). In this study, video cameras probe vehicles and Google Earth Pro® were used to collect different types of data. Data collection efforts are detailed in chapter V.

IV. 6 **Simulation**

After selecting performance measures and collecting data, the next step was to conduct the simulation. Simulation is considered an operation-research and management technique and is widely used in conducting research. Kaizer (40) defines simulation as following: "A Simulation is the imitation of a behavior of a system, entity, phenomenon or process through the exercise or use of a model." In transportation engineering simulation models are constructed by generating vehicles, assigning routes to vehicles (41)–(44), and assigning behaviors such as lane change preferences to the drivers (2), (45)–(47).

Simulation models rely on stochastic processes to generate random numbers and use them to construct the simulation models. Different random seeds generate different random numbers, and eventually, model outputs may differ from each other for different iterations.

Determining the number of runs is of significant importance, and it serves two main purposes:

- Making sure enough number of runs have been executed to reach confident results, and
- Avoiding too many runs that may result in wasting resources, if at all feasible.

The goal of determining the number of simulation runs is to ensure that the outputs of the study are consistent within the tolerable margins, and different iterations yield similar results (48).

There are two major approaches to determine the number of simulation runs (49), (50). The first set of techniques conduct a reasonable (within the study field) number of runs, calculate mean and standard deviation and use confidence interval and margin of error formulas to calculate the required number of simulation runs.

The second set of techniques start with very few simulation runs (as few as 2) and calculate mean, standard deviation, and the margin of error at each step and run more simulations until the desired margin of error is reached. In this study PTV-VISSIM software was used to perform 10 simulations and the first technique was used to determine the number of runs. The calculated number of required runs were very small (less than 4), therefore a minimum of 10 runs was chosen for all scenarios.

IV. 7 **Model Validation**

Simulation models are built to serve specific purposes and each model should be accurate within its purpose. Evaluating the accuracy of a model should be done with reference to the purpose of the model, and researchers emphasize that the purpose of the simulation should be known before a model is validated (51)–(53).

The first step to build each model is to build its conceptual model. The conceptual model "is the mathematical/logical/verbal representation" of the subject of the study (53). Sargent (52) defines the conceptual model validity as the process during which the modeler makes sure the assumptions and theories of the simulated models are correct, and the simulation model represents the subject of the study. The second step is to build a computerized model. A computerized model unifies different elements of the conceptual model into one model that can represent the study subject. To ensure a model's accuracy, researchers verify and validate their models.

Verification is checking if the conceptual model has been converted to the computerized model accurately (54). No model can reflect the actual situations with absolute accuracy because even the collected data and the mathematical models underlying the model may have inherent errors. In verifying the model, modelers ensure that the model is "sufficiently accurate" (51).

Many researchers propose that each stage of model building requires some form of verification or validation parallel to the model building task to ensure satisfactory outcome.

Different researchers classify validation tasks into different groups, but they all acknowledge 3 distinct categories: conceptual model validation, data validation, and operational validation. Conceptual validation step is conducted before simulation results comparison and is mainly conducted to ensure that the simulation is capable of supporting its intended uses (55). Data validation ensures that the quality and accuracy of input data are satisfactory, and operational validation compares the outputs of simulation to real world conditions to verify that the model can mimic the intended operation. Robinson (51) divides operational validation into "white-box" and "black-box" validation. White-box validation deals with the operational of small segments of the model, while black-box validation ensures that the whole model is representative of the real world with regard to the purpose of the study.

The operational validation techniques vary from asking other people their opinion about the model's accuracy and visually observing the model's operation to statistical methods and sensitivity analyses. In the context of this study, where the performance is "observable," a hypothesis test is suggested to compare the output of the model to the respective "observed" values.

Hypothesis tests usually start with stating the hypotheses, where:

H_0, or the null hypothesis, states that the model output represents the real world conditions, and

H_1, or the alternative hypothesis, states that the model is invalid and is not an "accurate" representation of the real world.

Researchers should choose validation techniques based on available data, the properties of their model, and purpose of the study. If there are multiple observations of a system, it is possible to use statistical validation approaches such as hypothesis testing. On the other hand, if there is only one observation of the system, confidence intervals are more commonly used (56).

IV.7.1 ***Independent samples t-test***

The independent samples t-test compares two sets of data points to determine if they are significantly different from each other. This method was first developed by William Gosset in 1908 and has been widely used ever since (57). The t-test is categorized as a hypothesis test where the null hypothesis is that the mean values of the two sets are not significantly different from each other.

The concept of the t-test is to answer the question whether the measured difference between two samples is smaller or larger than the standard error of the difference between the mean values of the two study samples (58):

$$t = \frac{\bar{X}_1 - \bar{X}_2}{S_{\bar{X}_1 - \bar{X}_2}} \tag{4}$$

where $\bar{X}_i$ is the mean for sample i,

and t is the test measure.

$S_{\bar{X}_1 - \bar{X}_2}$ is the standard error of the difference between the means of the two samples

$$S_{\bar{X}_1 - \bar{X}_2} = \sqrt{S_{\bar{X}_1}^2 + S_{\bar{X}_2}^2} \tag{5}$$

where $S_{\bar{X}_i}$ is the standard error of the mean for sample i, and

$$S_{\bar{X}_i} = \frac{\sigma}{\sqrt{n}} \tag{6}$$

where:

σ is the standard deviation of the sample and

n is the sample size.

In the present study, travel times have been measured multiple times by using a GPS enabled device in a pilot vehicle. This allowed the author to calculate the mean and standard deviations of the real measurements and compare them to the similar measures of model output data. The independent sample t-test was used to validate the models based on travel time. It should be noted that in conducting a t-test, it is assumed that the data points have been obtained from a normal distribution. To check the normality of the small sample sizes a Shapiro-Wilk test was conducted.

IV.7.2 ***Confidence interval approach***

A confidence interval is a range of computed values (59) and is computed based on the mean of the measurements, level of confidence, sample size, and standard deviation of the population or the standard deviation of the sample. The confidence interval is written as [L,U], where L represents the lower bound, and U represents the upper bound:

$$Confidence\ interval = \left[\bar{x} - z_c \frac{\sigma}{\sqrt{n}}, \bar{x} + z_c \frac{\sigma}{\sqrt{n}}\right] \tag{7}$$

where $\bar{x}$ is the sample mean, z_c is the critical value for normal distribution for confidence interval c, σ is the standard deviation of the population, and n is the sample size. In cases where the population standard deviation is not known, it is estimated based on the sample standard deviation. In this case, the standard z distribution is replaced by Student t distribution:

$$Confidence\ interval = \left[\bar{x} - t_c \frac{s}{\sqrt{n}}, \bar{x} + t_c \frac{s}{\sqrt{n}}\right] \tag{8}$$

where s is the sample standard deviation and t_c is the critical value for the t distribution based on confidence interval c. The values for z and t can be calculated and are available in statistical tables. Different degrees of freedom yield different t values. Table 4 shows the z and t values for commonly used confidence levels.

Table 4 z and t Values for Commonly Used Confidence Levels

Confidence level c	**Normal z**	**Student t**			
		d.f.=5	**d.f.=10**	**d.f.=20**	**d.f.=30**
0.8	1.282	1.476	1.372	1.325	1.310
0.9	1.645	2.015	1.812	1.725	1.697
0.95	1.960	2.571	2.228	2.086	2.042
0.99	2.576	4.032	3.169	2.845	2.750

The number of lane changes is measured based on the recorded video files, and thus there is only one measurement of the number of lane changes per a given time range for each study site. Using a confidence interval approach, a margin of error for the output of the simulation is calculated, and the real world measurements are checked to be within the confidence interval.

If the confidence interval contains the number of lane changes, the model passes the validation check.

There are other methods like non-parametric statistical methods that are used when regular methods are not applicable (60). Among these methods, the Mann-Whitney U test (61), Kolmogorov Smirnov test (62), (63), and the Moses test (64) are often used. In this study the Mann-Whiney U-test was used in cases where the samples were not normally distributed.

IV. 8 **Data Analysis**

Regression analysis is one of the oldest analysis methods in modern mathematics, with first publications dating back to 1805 (65). Among different statistical data modeling techniques, multiple regression analysis is considered very powerful and is capable of conveying meaningful clues about the impact of each of the explanatory variables on the response variable when other variables are assumed constant. This technique helps determine the impact of each of the independent variables (such as median type, number of driveways, etc.) on the dependent variables (such as the travel time and the number of lane changes.)

Conventionally, a simple multiple regression analysis is used when there are multiple independent variables. The following section explains this concept further. Multiple linear regression models have the following functional form:

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_r X_r + \varepsilon \quad (9)$$

where X_i represents each independent variable and β_i quantifies the relationship between X_i and Y and ε is the random error which measures the difference between the actual response and modeled response.

Coefficients β_i are calculated in a way that the resulting equation yields the minimum sum of squares of the errors for each data point. After calculating the coefficients, the next step is deciding if the variable X_i shall be included in the final model. This is done by conducting a hypothesis test for each variable, the hypothesis being "if the coefficient for the variable X_i is not significantly different from zero." To test this hypothesis, researchers calculate a p-value, which is equal to the probability that the null hypothesis is true. A small p-value indicates that there is reasonable evidence in the data that supports the alternative hypothesis. The alternative hypothesis indicates that there is a relationship between the variable and the response when other variables are assumed constant. The choice of the boundary for the p-value is not based on calculation and is chosen by the researchers. Usually, a 0.05 value is chosen for the p-value, indicating that variables with p-values less than 0.05 should be considered statistically significant.

IV.8.1 ***Preliminary variable selection***

Often, not all independent variables collected in the study are strongly associated with the dependent variable. Removing the irrelevant variables from the final model is called variable selection or subset selection. After subset selection, the researchers fit the model using the least squared method using the smaller group of variables. Best subset selection and stepwise selection (including forward- and backward- selection and hybrid approaches) methods are relatively classic methods that are widely trusted and applied in practice. Stepwise selection methods use similar approaches to compare different models. In this study the forward selection stepwise method was used to select variables. Forward selection starts with only one constant and adds variables with lowest likelihood test p-value to the model. Methods such as Mallow's C_p, Akaike information criterion (AIC), the Bayesian information criterion (BIC), and adjusted R-squared are used to help limit the number of selected variables and choose the best combination of variables for the model. These methods introduce measures to enumerate how each regression model makes predictions. Residual Sum of Squares (RSS) in one of the components in all of these methods. The RSS is the sum of squared errors of the prediction. If a model predicts ŷ for each y in the data, and e_i= $\hat{y}_i$ -y_i, then $RSS = \sum e_i{}^2$. Total Sum of Squares (TSS) measures the total variance in the response and is calculated by the sum of squared differences each response value has from the mean of the responses. TSS is formulated as $\sum(y_i - \bar{y}^2)$. RSS measured the unexplained variability after performing the regression while TSS measures the variability in the response before the regression is done. TSS and RSS are both dependent on the values of y. for data with larges values in the response RSS and TSS can have larger values that data with smaller response values. The R squared measure, calculates the proportion of explained variability in the model and is shown as:

$$R\ squared = \frac{TSS-RSS}{TSS} \tag{10}$$

R squared does not depend on the scale of the response and has a value between 0 and 1 with larger values showing the higher proportion of response explained by the model. Therefore a model with larger R squared has a better performance.

The C_p estimate for a model with d predictors is given by equation 13:

$$C_p = \frac{1}{n}(RSS + 2d\hat{\sigma}^2) \tag{11}$$

where RSS is the residual sum of squares, $\hat{\sigma}^2$ is an estimate for variance of error ε for each response, and d is the number of predictors. In other words, C_p adds a penalty of $2d\hat{\sigma}^2$ to the training RSS because the training error tends to underestimate the test error. The AIC compares the quality of a set of statistical methods to each other. AIC is defined as:

$$AIC = \frac{1}{n\hat{\sigma}^2}(RSS + 2d\hat{\sigma}^2) \tag{12}$$

where n is the number of observations, and all other variables are defined previously. The AIC and the C_p are related with a factor of $\frac{1}{\hat{\sigma}^2}$; and because for each response $\hat{\sigma}^2$ are the same, the AIC and the C_p can be used interchangeably (66). The BIC Criterion is derived from a Bayesian point of view and is given by:

$$BIC = \frac{1}{n\hat{\sigma}^2}(RSS + \log(n)d\hat{\sigma}^2) \tag{13}$$

The BIC, like AIC and C_p, tends to provide smaller values for models with smaller test errors, so the models with smaller BICs are more appropriate to choose.

The adjusted R^2 is a variation of R squared that accounts for the selected number of variables in the model:

$$Adjusted\ R^2 = 1 - \frac{RSS/(n-d-1)}{TSS/(n-1)} \tag{14}$$

where RSS is the residual sum of squares,

TSS is the total sum of squares,

n is the number of observations, and

d is the number of selected variables.

Unlike C_p, AIC, and BIC, the larger Adjusted R^2 values indicate models with greater prediction power. In this study, Adjusted R^2, C_p, and BIC were calculated and models with that performed reasonably well in all criteria were selected.

IV.8.2 ***Final variable selection***

To ensure precision, researchers evaluate regression models with different techniques. Cross validation and validation set methods are two commonly used methods to conduct final variable selection. In the cross validation method, the full data set is divided into randomly chosen smaller subsets, and at each step one set is left out, and the regression model is built with the rest of the data. The final variables are calculated based on outputs of cross validation.

In the validation set method, researchers divide data into a training set and a test set. After building the model using the training set, the test set is used to conduct prediction and calculate errors. At this step, the best number of variables – based on the choice criteria – is chosen. After deciding on the number of variables, the model is reconstructed using the full data set. Using the full data set should result in better estimates of the coefficients.

Validation set models tend to be faster since they include fewer calculation steps. For this reason, the validation set method was used to build the final model in this study. Numerical results are represented in the results section.

CHAPTER V

RESEARCH APPROACH

V. 1 **Data Collection**

This study focused on urban arterials near diamond interchanges. The process to identify such locations started with studying freeway networks that connected to urban networks with a diamond interchange. Except for the State of Florida, no other state lists their freeway interchange inventory, but almost every state has an online mapping system with different layers for presenting different information. Visually studying the networks and using features like Google® street view and measuring the distances were used to identify diamond interchanges. The next step was to choose urban arterials near diamond interchanges for the study. In addition to being located in the vicinity of a diamond interchange, the study sites met the following criteria:

- Included diamond interchanges with and without frontage roads,
- Included segments without driveways as well as segments with a large number of driveways,
- Included segments with wide range of reasonable distances from the terminal intersection to the first intersection,
- Included sites from different geographic locations.

This process eliminated many of the study sites. For example, in the state of Florida, arterials near diamond interchanges had very short distances from the terminal intersection to the first signalized intersection. Since this was specific to the state of

Florida, all those locations were eliminated from the candidate sites' pool. In some other cases, only one of the first two intersections were signalized and the other one was either an all-way stop controlled intersection or the signalized intersection was located miles away. Those locations were also eliminated. Another issue was the cost of out-of-state data collection and the research team needed to identify several possible sites in a reasonable proximity to each other such that if one site had maintenance or roadwork, other sites were available for data collection. For example, state of Kansas had only two interchanges (250 miles apart) that were located on state highways and met the research criteria and those also were eliminated.

Despite having to eliminate many of study locations, the research team ensured that the number of study sites were large enough to include different design options and were located in different geographical locations. Finally, 10 locations were selected and used in this study. Table 5 shows the locations of study sites and Figure 2 shows the study site locations on a map.

Table 5 Study Sites' Names and Locations

Site Name	Location	Latitude and Longitude
7th St.	I-17 @ S 7th AV	33.42932, -112.08251
27th St.	Route 101 @ N 27th Ave	33.66873, -112.11724
Bell Rd.	I-17 @ Bell	33.63988, -112.11562
East Chase	I-30 @ East Chase Pkwy	32.760121, -97.167163
Grand Prairie	I-20 @ S Great SW Parkway	32.676409, -97.044319
Conway	I-40 @ US 64/Oak	35.09155, -92.42268
Rogers	I-49 @ US 71B/Walnut	36.33507, -94.18353
Cambridge	MN 65 @ MN 95 (1st Ave E)	45.572890, -93.210275
Forest Lake	US10 @ 80th St S	44.832687, -92.964584
Cottage Grove	I 35 @ W Broadway Ave	45.279444, -93.003447

Figure 2 Location of Study Sites on the U.S. Map

The data collection process included collecting volume, origin-destination, lane change and signal timing data from collected video and collecting travel time data using a GPS enabled device in a probe vehicle. Geometric design characteristics were collected using the video files as well as online mapping services such as google maps® and google earth pro®. After collecting video data, the research team used appropriate methods to reduce and convert the video data into suitable numerical forms. Video data collection was done by the research team as part of the NCHRP 07-23 project data collection step in 2014 and 2016. For each study site, approximately 8 hours of video data was collected. At each study site, four cameras were located to capture all movement volumes at the intersections and driveways. Locations of cameras are shown in figures 3-12. Travel time pilot runs were also done at the time of video data collection. Table 6 shows volume, signal distance, number of driveways, median type and the control type of the right turn movement for each of the study segments as collected in the data collection process. This set of data was later used to construct the base models and to generate alternative scenarios.

Travel times for the purpose of model validation were collected using a GPS enabled device with direction of each round of travel time runs was collected in a single text file. Each line of the text file included speed, acceleration (or deceleration), latitude, longitude, and time. Appendix A shows a sample code the author used to extract the travel time for each of the study segments at each study site.

Table 6 Base Conditions for All Study Segments

Study Site	Volume (vph)	Signal Distance (feet)	Number of Driveways	Median Type	Control Type of right Turn
Cambridge 1	1700	1300	3	Strategic Raised	Yield
Cambridge 2	1600	625	1	Continuous Raised	Yield
Cambridge 3	1600	625	1	Continuous Raised	Yield
Cambridge 4	1700	1300	4	Strategic Raised	Yield
Forest Lake 1	650	1245	2	Strategic Raised	Yield
Forest Lake 2	700	500	0	Continuous Raised	Yield
Forest Lake 3	700	500	0	Continuous Raised	Yield
Forest Lake 4	650	1245	1	Strategic Raised	Yield
Cottage Grove 1	1800	785	0	Continuous Raised	Yield
Cottage Grove 2	1350	470	0	Continuous Raised	Yield
Cottage Grove 3	1350	470	0	Continuous Raised	Yield
Cottage Grove 4	1800	785	2	Continuous Raised	Yield
Bell Rd. 1	3100	1650	8	Strategic Raised	Yield
Bell Rd. 2	3100	1150	5	Strategic Raised	Yield
Bell Rd. 3	3100	1150	3	Strategic Raised	Yield
Bell Rd. 4	3100	1650	5	Strategic Raised	Yield
Grand Prairie 1	1900	400	3	Continuous Raised	Yield
Grand Prairie 2	1550	830	5	Strategic Raised	Yield
Grand Prairie 3	1550	860	3	Strategic Raised	Yield

Table 6 Continued

Study Site	Volume (vph)	Signal Distance (feet)	Number of Driveways	Median Type	Control Type of right Turn
Grand Prairie 4	1900	400	2	Continuous Raised	Yield
7th Street 1	1300	1100	7	TWLT	Yield
7th Street 2	1750	850	2	TWLT	Yield
7th Street 3	1750	850	9	TWLT	Yield
7th Street 4	1300	1100	8	TWLT	Yield
East Chase 1	900	600	1	Continuous Raised	Yield
East Chase 2	1700	850	5	Strategic Raised	Yield
East Chase 3	1700	850	4	Strategic Raised	Yield
East Chase 4	900	600	1	Continuous Raised	Yield
Conway 1	1400	800	3	TWLT	Yield
Conway 2	1000	600	5	TWLT	Yield
Conway 3	1000	600	10	TWLT	Yield
Conway 4	1400	800	3	TWLT	Yield
Rogers 1	2400	430	1	TWLT	Yield
Rogers 2	2400	1800	11	TWLT	Yield
Rogers 3	2400	1800	5	TWLT	Yield
Rogers 4	2400	430	1	TWLT	Yield
27th St 1	1500	650	3	Strategic Raised	Yield
27th St 2	1650	700	2	Strategic Raised	Yield
27th St 3	1650	700	1	Strategic Raised	Yield
27th St 4	1500	650	1	Strategic Raised	Yield

V. 2 **Simulation Process**

For each of ten study sites a different VISSIM model was constructed using collected data in the data collection step. Figures 3- 12 illustrate each of the base networks along with the location of cameras used for data collection. Each study site included four study segments and these segments were numbered such that oddnumbered segments were located upstream to the freeway on-ramp and even-numbered segments were located downstream to the freeway off-ramp.

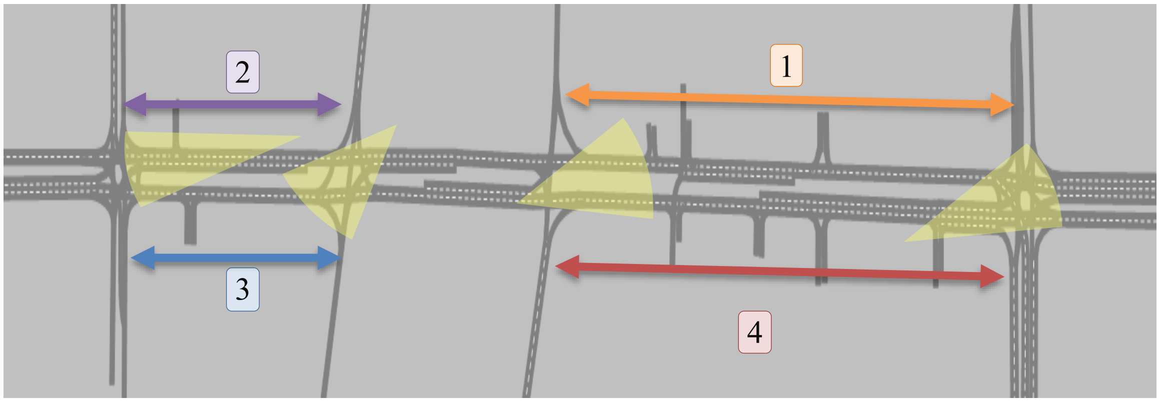

Figure 3 Cambridge, MN Site Base Network and Camera Locations

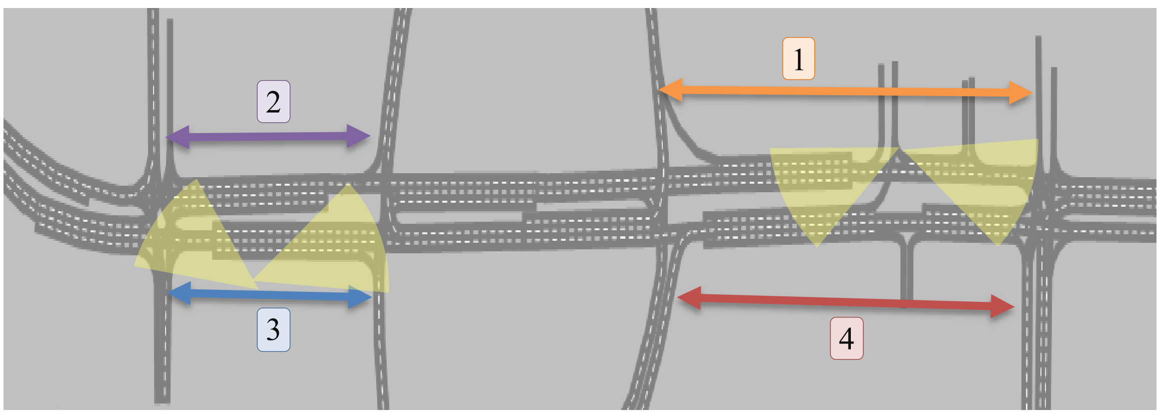

Figure 4 Forest Lake, MN Site Base Network and Camera Locations

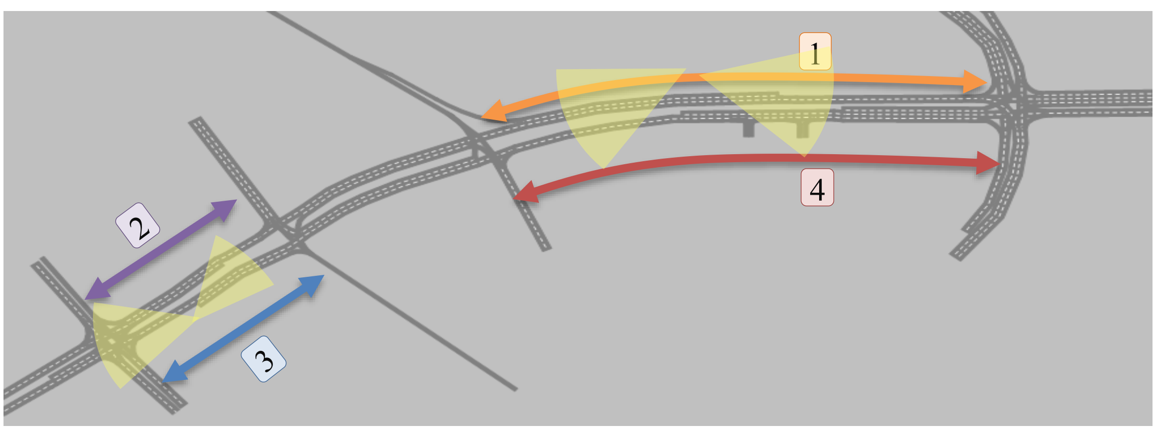

Figure 5 Cottage Grove, MN Site Base Network and Camera Locations

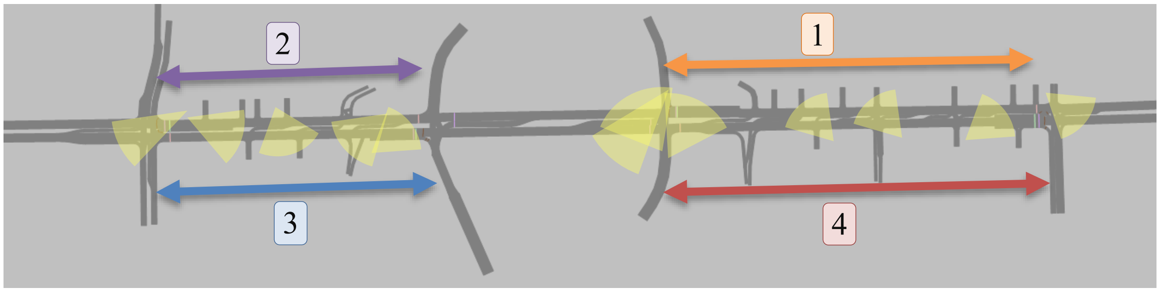

Figure 6 Bell Road, AZ Site Base Network and Camera Locations

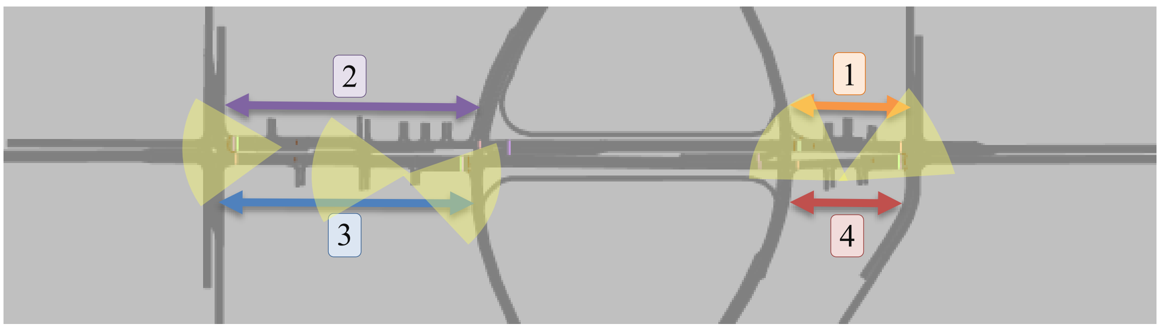

Figure 7 Grand Prairie, TX Site Base Network and Camera Locations

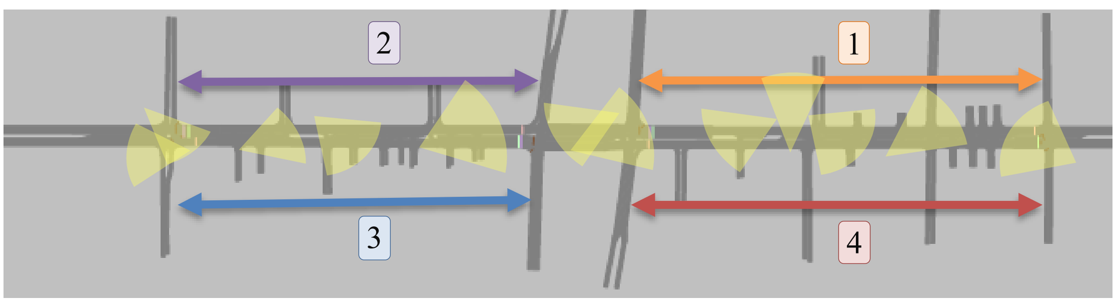

Figure 8 7th Street, AZ Site Base Network and Camera Locations

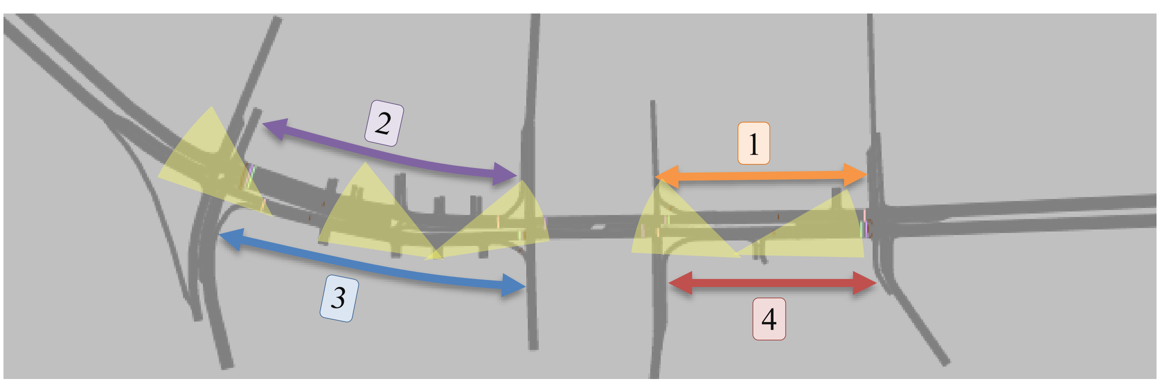

Figure 9 East Chase, TX Site Base Network and Camera Locations

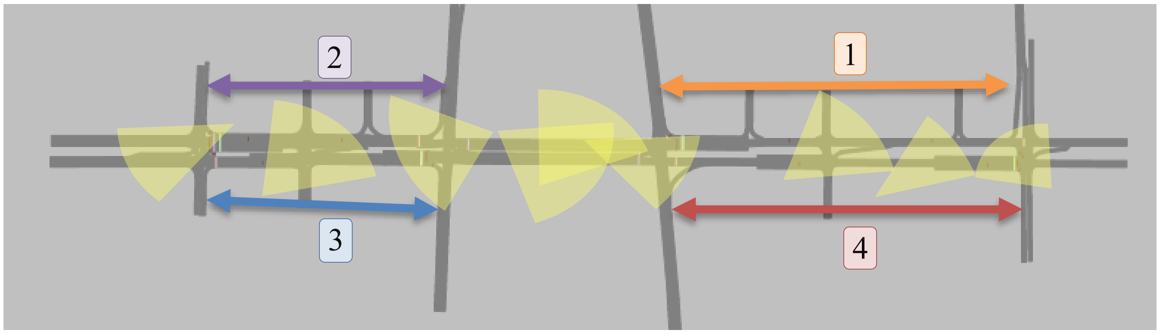

Figure 10 27th St, AZ Site Base Network and Camera Locations

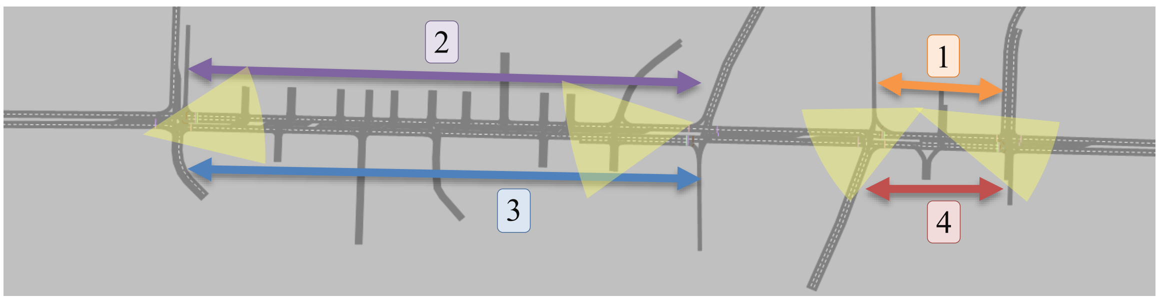

Figure 11 Rogers, AR Site Base Network and Camera Locations

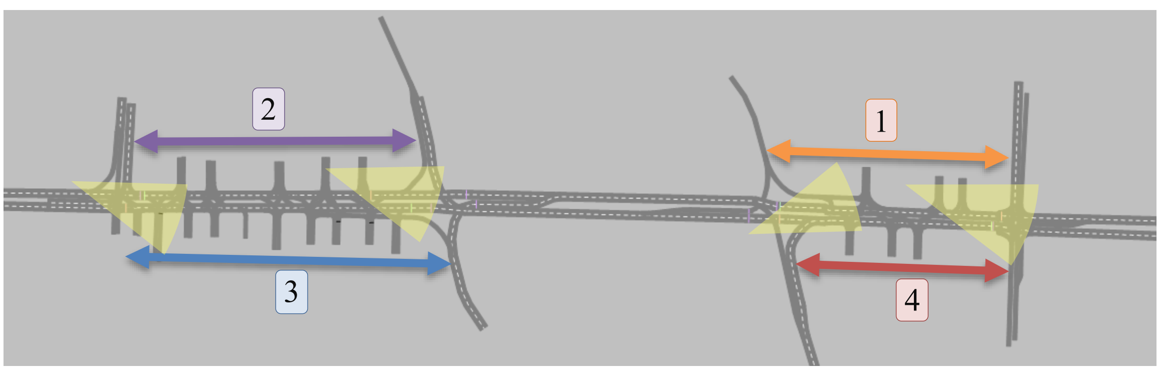

Figure 12 Conway, AR Site Base Network and Camera Locations

V.2.1 ***Model validation***

The base simulation models were built in PTV VISSIM 11 software (20), which is a part of the PTV suite. Each model consisted of a street network that included four consecutive signalized intersections located on a major arterial. The two middle signalized intersections served the on- and off-ramps of a diamond interchange (See figure 1). The research team reduced the volume and signal timing information from the approximately eight-hour-long video files recorded at the study location. The authors used Google Earth® and the available video files to obtain geometric design characteristics such as the number of lanes, lane widths, driveways, and channelization data. To reduce video data at each intersection and at driveways, UTC ULTRA boards from JAMAR technologies Inc. were used. JAMAR boards help reduce intersection video data by movement. The team collected the data in one-minute intervals to precisely determine the peak hour. After collecting the video data, an Excel® spreadsheet was created to include all movements in the network. The movement values that represented volumes entering the network were added to determine the peak hour. For movements that had continuous flow, the videos were played in actual speed while for driveways that did not have continuous flow, the videos were sped up and the time at which each car passed the driveway was recorded. These volumes were later added to the right time interval

A GPS enabled device in a probe vehicle was used to collect travel time data. The probe vehicle traveled along the study network in each direction several times logging location, time, speed, and acceleration every second. The study segments were the segments of the arterial located upstream and downstream of the ramps. Thus, the study network consisted of four study segments that had different traffic, geometric, and signal timing characteristics. Figure 1 illustrates the study segments and their assigned number in this study. Lane change measurements were obtained from the recorded video files. There was only one real-world lane change measurement available for each of the study segments during the study interval.

V.2.1.1 Validating travel times

In VISSIM, travel time measurement is done by adding travel time measurement segments to the network. These segments were added to VISSIM models and their respective coordinates were marked on Google Earth to obtain their latitudes and longitudes. The latitudes and longitudes were used to clean the probe vehicle GPS log files and obtain actual travel time values. Since there were several probe vehicle runs for each segments, a code in r language was used to perform this task.

The next step was to extract travel time values from the VISSIM simulation runs. Each of the ten simulation runs for each scenario produced several hundred counts of travel time for each segment (depending on the number of vehicles traversing each travel time measurement segment). VISSIM was set up to record and report the average of travel times for every 15 minutes of simulation run time. This produced 40 measurements (each being the average of simulated vehicle travel times) over ten simulation runs for each of the four segments at each site.

There were slight differences between the traveled path lengths in VISSIM software and measured travel lengths in log files. This difference stemmed from the fact that the real world travel time measurements came from a GPS enabled device that only logged location data every second. Figure 13 illustrates six different pilot vehicle runs for Cambridge segment 2. Points with the same color belong to the same pilot run.

Since VISSIM measured the travel times for each vehicle between the two specific points- yellow dashed lines in Figure 13, the author needed to calculate the travel time and the traveled distance for points where their longitude and latitude fell between the respective longitudes and Latitudes of the VISSIM model's travel time measurement boundaries. In Figure 13, the top section shows the mapped data from actual vehicle runs, the bottom section shows the VISSIM model with travel time measurement segment, and the yellow dashed lines project the start and the end of the travel time measurement segment on the Google Earth® map for Cambridge Segment 2. The traveled distances between the first and last points of each pilot run were shorter than the VISSIM distance. The author adjusted travel time measurements to account for this difference. Table 7 summarizes travel time measurements and simulation run results for Cambridge, MN. This process was repeated for all study sites to validate the models.

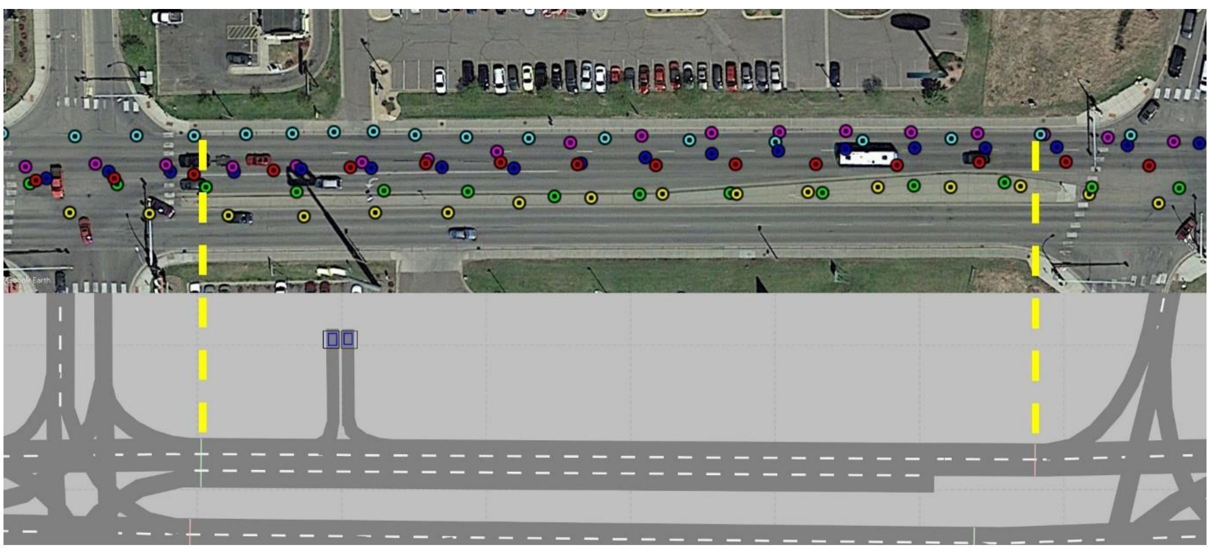

Figure 13 Mapped Pilot Run Points vs. VISSIM Travel Time Segment Source: Google Earth Pro

Table 7 Descriptive Statistics of Measured and Simulated Travel Times for Cambridge, MN

	N	Min (sec.)	Max (sec.)	Mean (sec.)	Standard Deviation (sec.)
Segment 1 Measured	16	24.60	55.02	32.65	9.91
Segment 1 Simulated	40	27.68	29.83	28.79	0.49
Segment 2 Measured	16	11.71	15.51	13.27	1.16
Segment 2 Simulated	40	12.62	14.79	13.48	0.56
Segment 3 Measured	16	11.11	36.06	14.05	5.97
Segment 3 Simulated	40	12.32	15.03	13.22	0.57
Segment 4 Measured	16	19.33	63.13	33.40	14.47
Segment 4 Simulated	40	28.18	31.66	29.84	0.84

A t-test was used to compare means of the two sets of data (measured versus simulated.) The t-test assumes that the data points belong to a normally distributed population. To check the normality of the samples with a small size, the author conducted a Shapiro-Wilk test.

The Null Hypothesis (H0) in the Shapiro-Wilk test assumes that the sample came from a normal distribution. With p-values in all four tests were smaller than the significance level of 0.05, and this indicated that the null hypothesis could be rejected and the data were not distributed with a normal distribution. Table 8 shows the p-value and Shapiro-Wilk test results summary for the travel time measurements. The normality test and the t-test were conducted for all travel time measurements at all study sites. The w statistic in Shapiro-Wilk test is the measure of normality and is defined as:

$$W = \frac{(\sum_{i=1}^{n} a_i x_i)^2}{\sum_{i=1}^{2}(x_i - \bar{x})^2} \qquad (15)$$

where:

x_i is the i^{th} smallest value of x and a_i is the Shapiro-Wilk constant. If W is smaller than a critical value, the null hypothesis is rejected and therefore we can conclude that the dataset does not come from a normally distributed dataset which was the case in this data set. The critical W values can be calculated and are also available in tables. The critical W value for a sample of size 16 and alpha=0.05 16 is 0.887 (67).

Table 8 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for Cambridge, MN

Measurement	w-value	p-value
Segment 1	0.77372	0.001245
Segment 2	0.88302	0.04326
Segment 3	0.43326	$6.231e^{-07}$
Segment 4	0.80215	0.002909

Figure 14 illustrates box plots of measured and simulated travel times for each of four study segments in Cambridge, MN. The figures show that measured travel times had more variation than the simulation results.

If travel time data were not normally distributed then the data were transformed and the transformed data were checked for normal distribution. The measured travel time was transformed by taking its square root, log, reciprocal, and second power. If none of these resulted in a normal distribution, a non-parametric method was used to compare the two sets of data. Results for other study sites are summarized in Appendix B.

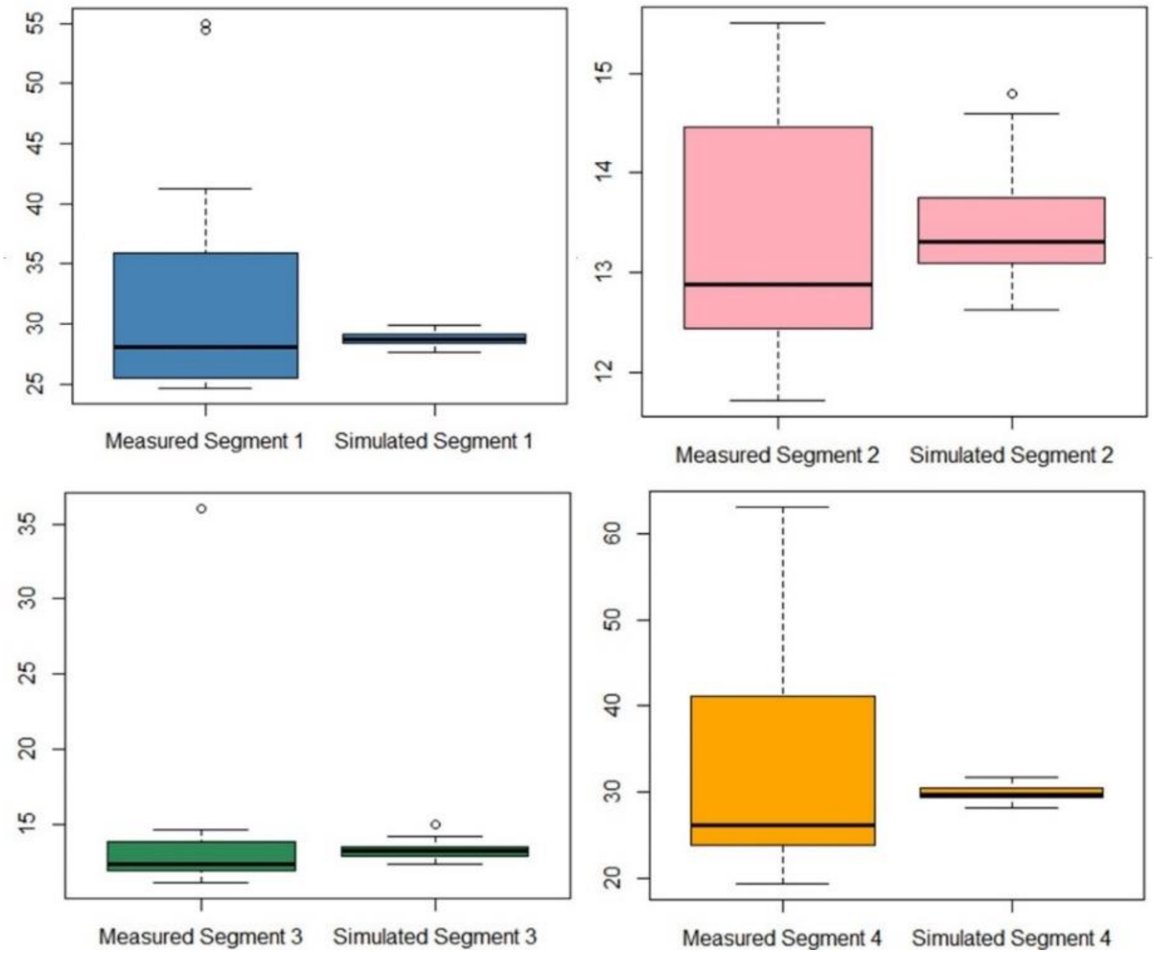

Figure 14 Box Plots of Measured and Simulated Travel Times for Cambridge, MN

Wilcoxon rank-sum test (also known as the Mann Whitney U-test) is a non-parametric test and is an alternative to the two-sample t-test for non-normally distributed data. The null hypothesis in Wilcoxon rank-sum test is that the two independent sets came from populations with the same distribution. In the data, all tests had p-values over 0.05, and the null hypothesis could not be rejected (see Table 9.)

Table 9 Wilcoxon Rank Sum Test Result Summary for Travel Times for Cambridge, MN

Measurement vs. Simulation	W	p-value
Segment 1	118	0.6689
Segment 2	89	0.1486
Segment 3	88	0.1381
Segment 4	99	0.2871

V.2.1.2 Validating number of lane changes

The number of lane changes made by vehicles was measured based on the 1-hour long recorded video files and there was only one measurement of the number of lane changes for each study segment.

To validate the number of lane changes, it was assumed that the data from video files were the ground-truth-value of the lane changes ($\mu 0$) and a confidence interval testing was conducted to see if the $\mu 0$ value is in the 99% confidence interval of simulation results.

Using a confidence interval approach, a margin of error for the outputs of the simulation were calculated and the real-world measurements were examined to be within the confidence interval for each Segment. Table 10 shows descriptions of simulated values for Number of Lane Changes (NLC) in Cambridge, MN study site, and the criteria used for confidence interval testing. The results indicated that the model could produce a reasonable number of lane changes. This process was repeated for all study segments at all study sites. The results are presented in Appendix B.

Table 10 Confidence Interval Testing for Simulated Number of Lane Changes (Cambridge, MN)

	Segment 1	Segment 2	Segment 3	Segment 4
Min	119	219	79	220
Max	147	257	111	263
Mean	129.9	239.8	94.7	241.3
Standard Deviation	10.70	12.50	9.53	12.81
Lower bound	118.31	226.26	84.37	227.43
Upper bound	141.49	253.34	105.03	255.17
Measured NLC	122	228	88	228
Valid?	Yes	Yes	Yes	Yes

V.2.2 ***Developing scenarios for study***

As stated previously, the goal of this research was to provide guidance on how to improve the performance of urban arterials near diamond interchanges. Treatments such as changing the median type, adding, removing, or relocating driveways and changing the controller settings to permit or block right turns on red at the terminal intersections are relatively fast and easy to implement. These treatments along with volume were adjusted to perform a regression analysis to study their impact on the operational performance of the arterials.

Although volume was not an access management variable, it was essential to study arterial under different volume conditions. Capacities for each study segment were calculated using the Highway Capacity Manual’s procedure (29).

For driveways, in addition to the present setting of the driveways, scenarios with one driveway for the entire segment and no driveways were created. In both driveway alternative options, the driveway traffic flow was diverted to the appropriate destination.

For median treatment option, where the median was not raised, a raised median alternative scenario was built and studied. This was done by modifying the OD matrices and changing link settings to block vehicles from using the median. In addition, the traffic control type of the right turning movement was modified in downstream segments to account for stop controlled vs yield controlled traffic. Table 11 summarizes different scenario options.

Table 11 Scenario Management Options

Volume	Driveways	Median	Traffic Control Type of Right Turn
Off Peak	All driveways	As is	Stop controlled
Peak	No driveways	All raised	Yield controlled
0.75xCapacity	One driveway for each Plaza		
Capacity			

Not all study sites had the potential to create all 48 possible scenarios. For example, if a study site had raised median, changing median options was dropped from the scenario production process. Table 12 shows scenario options for all of study segments.

Table 12 Scenario Options for All Study Segments

Study Site	Number of Driveways	Median Type	Traffic Control Type of Right Turning Movement	Volume (vph)
Cambridge 1	3,1,0	Strategic Raised, Continuous Raised	Yield	1550, 1700, 3900, 5200
Cambridge 2	1,0	Continuous Raised	Yield, Stop	1400, 1600, 3400, 4700
Cambridge 3	1,0	Continuous Raised	Yield	1400, 1600, 3400, 4700
Cambridge 4	4,1,0	Strategic Raised, Continuous Raised	Yield, Stop	1550, 1700, 3900, 5200

Table 12 Continued

Study Site	Number of Driveways	Median Type	Traffic Control Type of Right Turning Movement	Volume (vph)
Forest Lake 1	2, 1, 0	Strategic Raised, Continuous Raised	Yield	600, 650, 1300, 1700
Forest Lake 2	0	Continuous Raised	Yield, Stop	650, 700, 1850, 2500
Forest Lake 3	0	Continuous Raised	Yield	650, 700, 1850, 2500
Forest Lake 4	1, 0	Strategic Raised, Continuous Raised	Yield, Stop	600, 650, 1300, 1700
Cottage Grove 1	0	Continuous Raised	Yield	1150, 1800, 2700, 3700
Cottage Grove 2	0	Continuous Raised	Yield, Stop	1200, 1350, 2150, 2850
Cottage Grove 3	0	Continuous Raised	Yield	1200, 1350, 2150, 2850
Cottage Grove 4	2, 1, 0	Continuous Raised	Yield, Stop	1150, 1800, 2700, 3700
Bell Rd. 1	8, 1, 0	Strategic Raised, Continuous Raised	Yield	2000, 2700, 3100, 3900
Bell Rd. 2	5, 1, 0	Strategic Raised, Continuous Raised	Yield, Stop	1800, 2500, 3100, 3900
Bell Rd. 3	3, 1, 0	Strategic Raised, Continuous Raised	Yield	1800, 2500, 3100, 3900
Bell Rd. 4	5, 1, 0	Strategic Raised, Continuous Raised	Yield, Stop	2000, 2700, 3100, 3900
Grand Prairie 1	3, 1, 0	Continuous Raised	Yield	900, 1500, 1900, 2700
Grand Prairie 2	5, 1, 0	Strategic Raised, Continuous Raised	Yield, Stop	700, 1300, 1700, 2350
Grand Prairie 3	3, 1, 0	Strategic Raised, Continuous Raised	Yield	700, 1550, 1700, 2350
Grand Prairie 4	2, 1, 0	Continuous Raised	Yield Stop	900, 1550, 1900, 2700
7th Street 1	7, 1, 0	TWLT, Continuous Raised	Yield	1300, 2000, 2300, 2500
7th Street 2	2, 1, 0	TWLT, Continuous Raised	Yield, Stop	1750, 2450, 2750, 2950
7th Street 3	9, 1, 0	TWLT, Continuous Raised	Yield	1750, 2450, 2750, 2950
7th Street 4	8, 1, 0	TWLT, Continuous Raised	Yield, Stop	1300, 2000, 2300, 2500
East Chase 1	1, 0	Continuous Raised	Yield	900, 1100, 1200, 1400
East Chase 2	5, 1, 0	Strategic Raised, Continuous Raised	Yield, Stop	1200, 1500, 1700, 2100

Table 12 Continued

Study Site	Number of Driveways	Median Type	Traffic Control Type of Right Turning Movement	Volume (vph)
East Chase 3	4, 1, 0	Strategic Raised, Continuous Raised	Yield	1200, 1500, 1700, 2100
East Chase 4	1, 0	Continuous Raised	Yield, Stop	900, 1100, 1200, 1400
Conway 1	3,1,0	TWLT, Continuous Raised	Yield	1000, 1400, 1800, 3600
Conway 2	5, 1, 0	TWLT, Continuous Raised	Yield, Stop	800, 1000, 1600, 3200
Conway 3	10, 1, 0	TWLT, Continuous Raised	Yield	800, 1000, 1600, 3200
Conway 4	3, 1, 0	TWLT, Continuous Raised	Yield, Stop	1000, 1400, 1800, 3600
Rogers 1	1, 0	TWLT, Continuous Raised	Yield	1800, 2200, 2400
Rogers 2	11, 1, 0	TWLT, Continuous Raised	Yield, Stop	1800, 2200, 2400
Rogers 3	5, 1, 0	TWLT, Continuous Raised	Yield	1800, 2200, 2400
Rogers 4	1, 0	TWLT, Continuous Raised	Yield, Stop	1800, 2200, 2400
27th St 1	3, 1, 0	Strategic Raised, Continuous Raised	Yield	800, 1100, 1500, 1900
27th St 2	2, 1, 0	Strategic Raised, Continuous Raised	Yield, Stop	850, 1150, 1650, 2050
27th St 3	1, 0	Strategic Raised, Continuous Raised	Yield	850, 1150, 1650, 2050
27th St 4	1, 0	Strategic Raised, Continuous Raised	Yield, Stop	800, 1100, 1500, 1900

V.2.3 ***Numbering scenarios***

Working with several variables and conducting a factorial simulation required using unique identification numbers for each study scenario. Numbering the scenarios would make it possible to organize inputs and outputs in later steps of the study.

This study was a full factorial study meaning that all possible options of all variables were combined to make different scenarios. To create a unique number for each scenario, a generalized hierarchical numbering process was created and each scenario was given a unique number starting with one and ending in 2232.

V.2.4 ***Determining the number of runs***

Running complex and detailed VISSIM simulation models was time- and resource-intensive, and it was important to determine the number of simulation runs in a way that the simulation results were reliable while computational and time resources were used efficiently. Traditionally, researchers calculate the required number of runs based on the determined confidence level and acceptable error (49). Rearranging the confidence interval formula gives the following equation for the number of required runs:

$$n = \left\lceil \frac{s^2\, t_c^2}{\epsilon^2} \right\rceil \tag{16}$$

In equation 16 "⌈ ⌉" represents the ceiling function, and ε denotes allowable percentage error of the estimate. s is the standard deviation of the sample of the simulation runs, and t_c is the critical t value. In this study ε was chosen to be 5 percent and the t_c value for 95 percent is 1.96 (*51.*) To determine the appropriate number of simulation runs, the author ran the simulation ten times and calculated a new minimum number of simulation runs based on the mean and standard deviation of the model outputs. This resulted in a small number of calculated required runs (less than four) since the models produced stable outputs, and the standard deviation values were small. The author elected to include additional iterations and ultimately ran ten simulations per scenario.

V.2.5 ***Determining the length of simulation runs***

Each simulation run should run long enough for the system to attain a normal operation state before output data are collected. This "warm up" time should be adjusted based on the network size. In this study each model had four main intersections, and four study segments. Based on this information and observing simulation runs, a ten-minute warm up time was chosen.

Moreover, to determine the actual simulation run time, the author chose one hour to replicate peak and off-peak hours. All other scenarios were also run for a duration of one hour to maintain consistency.

V. 3 **Final Database**

The final database was comprised of four general elements: geometric design characteristics, volume data, travel time, and number of lane changes.

Measuring travel times on the simulated network started by adding vehicle travel time measurement segments at each study segment. After adding these segments, Vissim was configured to store travel time data for each simulation run. The stored values were later exported to tables and added to the final database.

Producing the number of lane changes involved several steps and two sets of Vissim output files were used to extract the number of lane changes:

- A lane change data file that logged each lane change movement. Vissim was configured to produce one lane change output file for each simulation run. This file was in a text format and included all lane change maneuvers pre-defined time periods. The lane change output file in Vissim could not be adjusted to include user-specified variables and only logged the vehicle number, link number, lane number, speed, acceleration, leading and lagging vehicles information.
- A vehicle records output file that logged information about each vehicle in user-specified intervals. Vissim was configured to produce one vehicle record output file for each simulation run. This output file had text formatting and logged locations and other information of each vehicle in the system in defined intervals. In this study, the location of each vehicle along with other information were logged every second. The vehicle records data can be adjusted to research needs. For this study, vehicle numbers, simulation run number, origin zone, destination zone, lane change and delay every second were collected.

The vehicle records file and the lane change output file were produced for each of the simulation runs. For example, for a site with 60 scenarios and ten runs for each scenario, 600 files were created.

A program in R language was written to extract the needed data for each lane change movement in the lane change data file from the vehicle records output file.

To subset the needed lane changes from the vehicle records file a mutual unique identification number was generated. This unique identification number was generated by combining the vehicle number and simulation second. The lane change data were then combined with the main data set.

V.3.1 ***Data cleaning***

After compiling all data, a thorough data cleaning was conducted. Data cleaning (or data scrubbing or data cleansing) is conducted to improve data quality and to ensure statistical model assumptions hold in the data. Data cleaning is an important step before conducting data analysis and can ultimately reduce type I (false positive) and type II (false negative) errors in the statistical model. Removing some data from the database and updating the data are two of most commonly used data cleaning strategies (68). The following data cleaning steps were taken in this study.

V.3.1.1 Removing redundancies in scenarios

The independence assumption states that each data point in the study should come from a different subject. Redundant measurements for a part of data points increase the chance of getting biased outputs (69) and result in meaningless p-values. In our study, since each study site consisted of four study segments, some segments could have two or three sets of runs for the same scenario, while other segments were running under different scenarios. As mentioned previously, the three options for changing driveway options were: a- studying each segment while all current driveways stayed the same, b- removing all driveways, except one driveway and diverting all driveway volumes to one driveway, and c- removing all driveways and diverting all driveway volumes to an appropriate cross-road. For example, for East Chase, TX site, while options a and b yielded two different settings for segments 2 and 3, they yielded the same settings for segments 1 and 4. This happened because segments 1 and 2 only had one driveway. For segments with no driveways in base condition, options a, b, and c would yield the same settings. The author investigated this issue for all study sites and removed all redundant runs from the database. This ensured that the independence condition for all study sites held. Figure 15 shows base conditions for East Chase, TX site.

After removing all redundancies, 8820 data points, representing 882 unique and independent scenarios modeled 10 times each, remained.

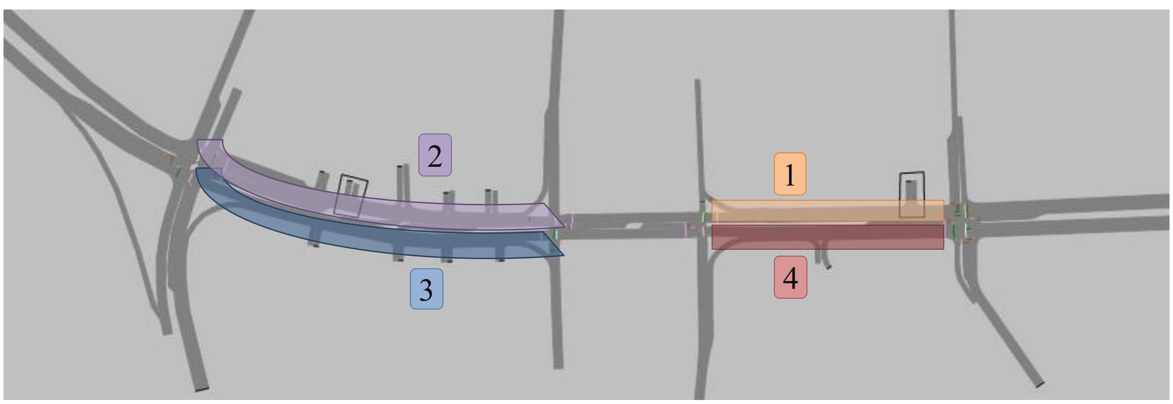

Figure 15 East Chase, TX Study Site

V.3.1.2 Removing outlier results

After running the scenarios, some runs resulted in excessively high travel times. This could have happened because of an irregular condition in that specific run. In real world, this condition can happen in case of an especial event or when a car makes an illegal turn or stop. This can also happen when there is an overflow in one of the segments that blocks cars from moving in another segment, resulting in unrealistic results in upstream or downstream segments. In this study, many of such scenarios were created by excessive volumes, or by converting medians to a raised median. The outlier results were removed from the final database. Instead of travel time that is relative to study segment length, the author calculated speed and removed simulation runs that had speed values less than 10 miles per hour. This left 7907 data points representing 807 scenarios. Compared to 882 scenarios at the beginning of this step, 75 scenarios were completely removed from the database. Appendix C represents scenario combinations for these scenarios.

V.3.1.3 Updating volumes

One of the advantages of the Vissim software is that it produces accurate outputs that can be adjusted to the research needs. Data collection points can be added to the network to collect extra information on the network. Despite having the same input volume for each of ten simulation runs per scenario, actual simulated volumes could vary slightly. Volume data were produced by placing data collection points on appropriate locations on the network and extracting their data after simulations. The input volumes were replaced with the actual simulated volumes in the final database.

V. 4 **Descriptive Statistics**

An exploratory data analysis was conducted to get a better sense of the available data for all the 7907 runs representing 807 scenarios and to select appropriate independent variables. It is noteworthy that the independent variables that were collected using Google Earth® were all updated for each scenario when the final database was compiled. Table 13 shows the descriptive statistics for all variables included in the analysis. The variables are listed in alphabetical order for easier reference. Below are descriptions of the variables:

AuxLt: Binary variable, 1indicates there exists an auxiliary left-turning lane, 0 if there are no auxiliary left turning lanes [No unit]

AuxRt: Binary variable, 1indicates there exists an auxiliary right-turning lane, 0 if there are no auxiliary right turning lanes [No Unit]

DenDw: Density of the driveways [Driveway/mile]

DistDw1: Distance of the first driveway from the upstream intersection [feet]

DistDw1DwN: Distance of the first driveway to the last driveway [feet]

DistSig: Distance from upstream intersection to the first signalized intersection [feet]

DistUPSDwmax: Distance of the upstream intersection to the largest volume driveway [feet]

DistUPSLTLane: Distance of upstream intersection to the beginning of the left-turning lane (if no left-turning auxiliary exist, this is equal to DistSig) [feet]

DistUPSMed: Distance of upstream intersection to the beginning of the first median opening [feet]

DistUPSRT: Distance of upstream intersection to the beginning of right turning lane [feet]

DistUPST1LT: Distance between upstream intersection to the first point where cars can turn left (can be a median opening or at the intersection, whichever is shorter) [feet]

LenMedStor: Length of median storage [feet]

MdensDw: Modified density of the driveways (number of driveways*5280/distance from the start of the first driveway to the end of the last driveway) [Driveway/mile]

MedContin: Binary variable, 1 if the median is continuous through the entire segment, 0 otherwise [No unit]

NL: Number of lanes [Count]

NLC: Number of lane changes [Count]

NoDw: Number of driveways [Count]

NoLTLanes: Number of exclusive left-turning lanes [Count]

NoMed: Number of median openings [Count]

NoRTLanes: Number of exclusive right turning lanes [Count]

SimVolLT: Simulated left-turning volume passed through the downstream intersection [Vehicles/hour]

SimVolRT: Simulated right turning volume passed through the downstream intersection [Vehicles/hour]

SimVolTru: Simulated through volume passed through the downstream intersection [Vehicles/hour]

SimVolUT: Simulated u-turn volume passed through the upstream intersection [Vehicles/hour]

TMed: Median type [Categorical]

TT: Travel Time [Seconds]

VolDwmax: Volume of largest volume driveway [Vehicles/hour]

WidthDw1: Width of the first driveway [feet]

WidthDwMax: Width of the largest volume driveway [feet]

WiMedOp: Length of median opening [feet]

Table 13 Base Statistics for Study Variables

Variable	Min	Max	Range	Sum	Median	Mean	Std.Dev
AuxLt	0	1	1	525	1	0.6	0.49
AuxRt	0	1	1	606	1	0.69	0.46
DenDw	0	43.7	43.7	5001.2	3.2	5.7	8.6
DistDw1	0	1640	1640	501964.5	465	569.1	388
DistDw1DwN	0	1354	1354	332738	50	377	435
DistSig	375	1792	1417	790621	785	896.4	380.5
DistUPSDwmax	112.7	1640	1527.3	570258.1	517	646.6	347.5
DistUPSLTLane	107	1792	1685	655096	632	742.7	408.5
DistUPSMed	0	1792	1792	482992	465	547.6	313.3
DistUPSRT	0	1792	1792	637435	607	722.7	405.6
DistUPST1LT	106	1306	1200	482332	517	546.9	350.4
LenMedStor	0	570	570	81632	0	93	168
MdensDw	0	203	203	53050	17	60	73
MedContin	0	1	1	481	1	0.55	0.5
NL	2	3	1	1876	2	2.13	0.33
NLC	12.3	3901.1	3888.8	565453	509.4	641.1	518.7
NoDw	0	11	11	938	1	1.06	1.93
NoLTLanes	0	1	1	525	1	0.6	0.49
NoMed	0	6	6	831	0	0.94	1.65
NoRTLanes	0	1	1	606	1	0.69	0.46
SimVolLT	0	759	759	69138	37	78	117
SimVolRT	5.3	640.5	635.2	130536.8	114.5	148	127.5
SimVolTru	37.3	2787.6	2750.3	715949	662.2	811.7	524.6
SimVolUT	0	13.3	13.3	290.4	0	0.33	1.8
TMed	1	4	3	1648	2	1.87	0.98
TT	10	337	327	29873	25	34	40
VolDwmax	0	268	268	21171	1	24	45
WidthDw1	0	74	74	16557	21	18.8	19.1
WidthDwMax	0	74	74	16711.1	26	18.9	20.2
WiMedOp	0	270	270	38506	0	44	74

In addition to information presented in Table 11, it is essential to include site specific statistics. The following plots and tables visualize site characteristics for different variables. Figures 16, 17, and 18 illustrate range of simulated through, left-turning, and right turning volumes for each of study sites. Figures 19, 20, 21, and 22 show signal distance, the distance of the first driveway, distance of upstream intersection to the right turning lane, and distance of the upstream intersection to left tuning lane respectively for each of the study sites.

In Figure 23, the number of actual driveways on the base network for each of the study sites. Table 14 shows if there was a left turning lane present at each segment for each study site. In Figures 24, 25, 26, and 27 the Distance of Upstream Intersection to the First Left Turning Movement, Distance of Upstream Intersection to the Largest Volume Driveway, Distance of Upstream Intersection to the First Median Opening and Length of Median Storage for each Segment are shown respectively. Figures 28 shows the number of lanes for each study segment while Table 15 shows if there was a right turning lane present at a segment. Figure 29 illustrates the number of median openings of each study segment.

All figures were produced from the final cleaned database. In cases where there are fewer options available (such as Signal Distance in Figure 19) plots only show the available options.

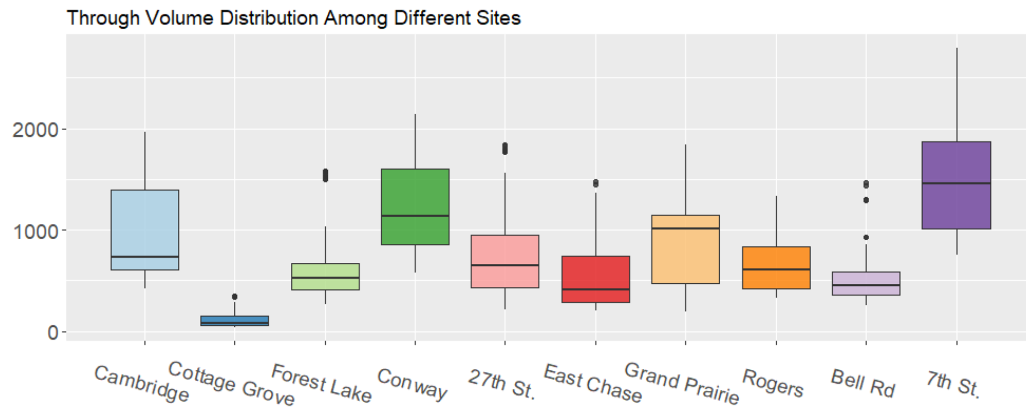

Figure 16 Simulated Through Volumes for Each Study Site

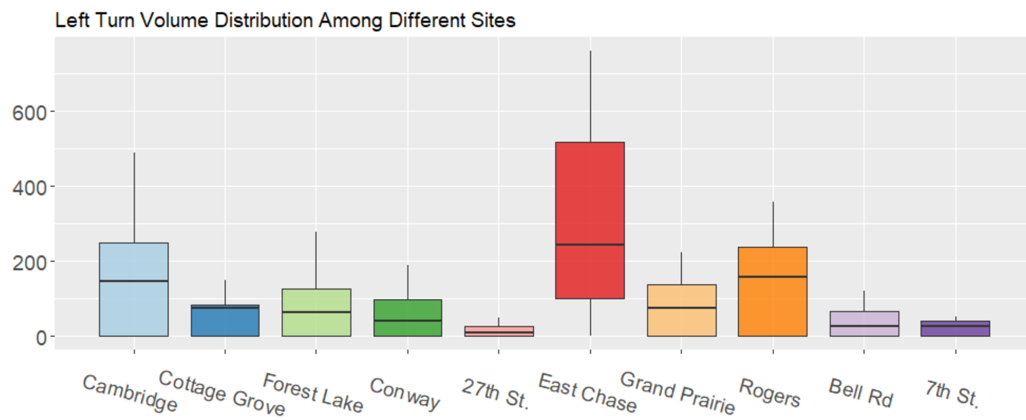

Figure 17 Simulated Left-turning Volumes for Each Study Site

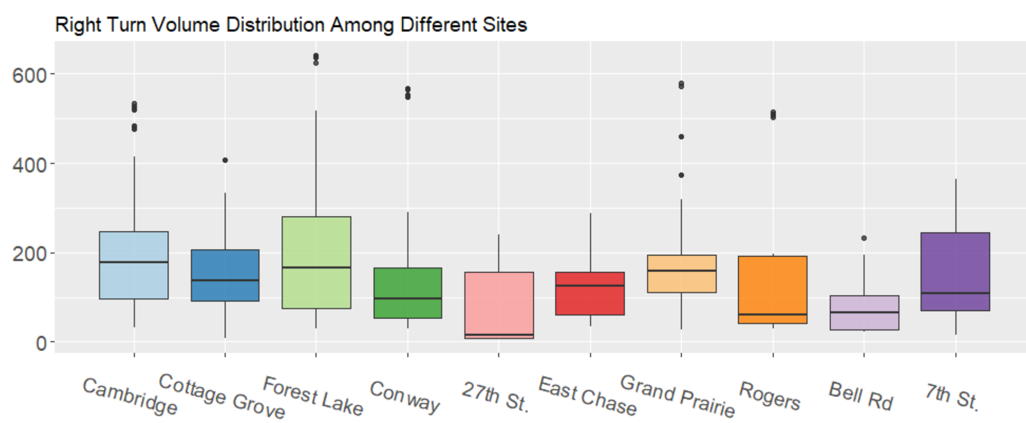

Figure 18 Simulated Right Turning Volumes for Each Study Site

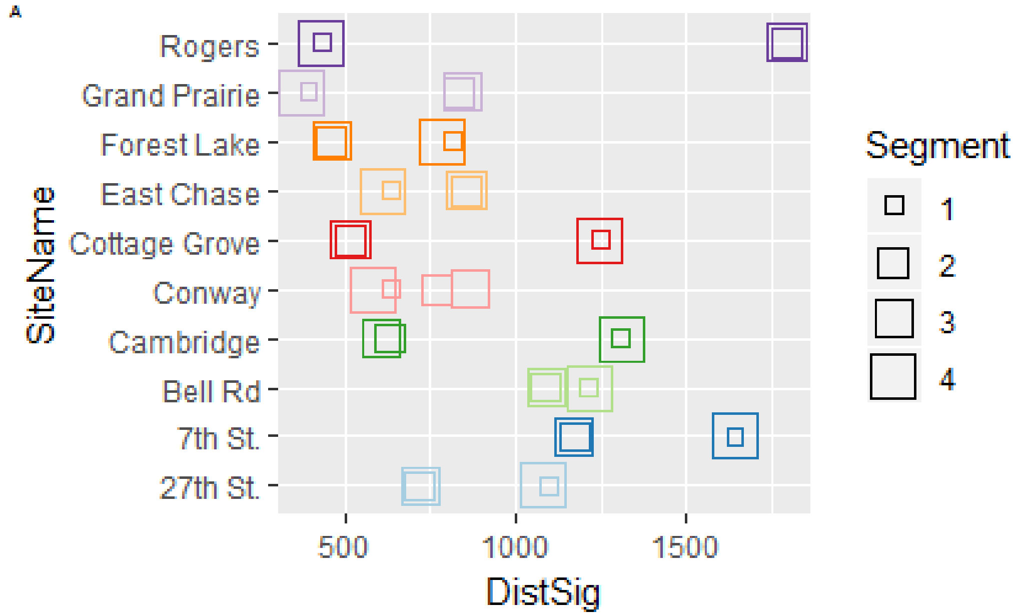

Figure 19 Signal Distance for Each Segment at Each Study Site

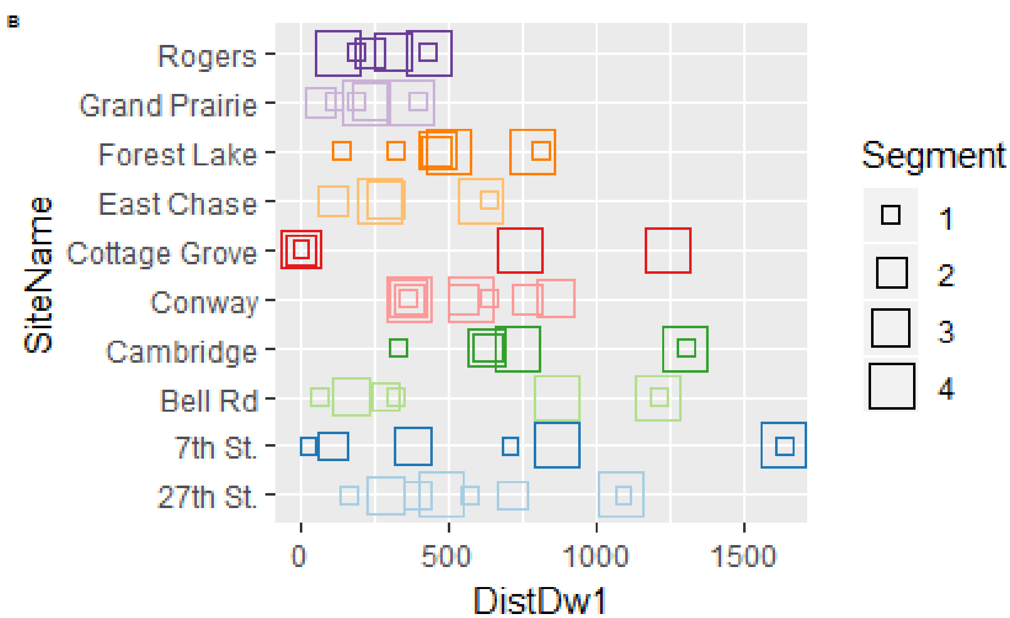

Figure 20 Distance of the First Driveway for Each Segment at Each Study Site

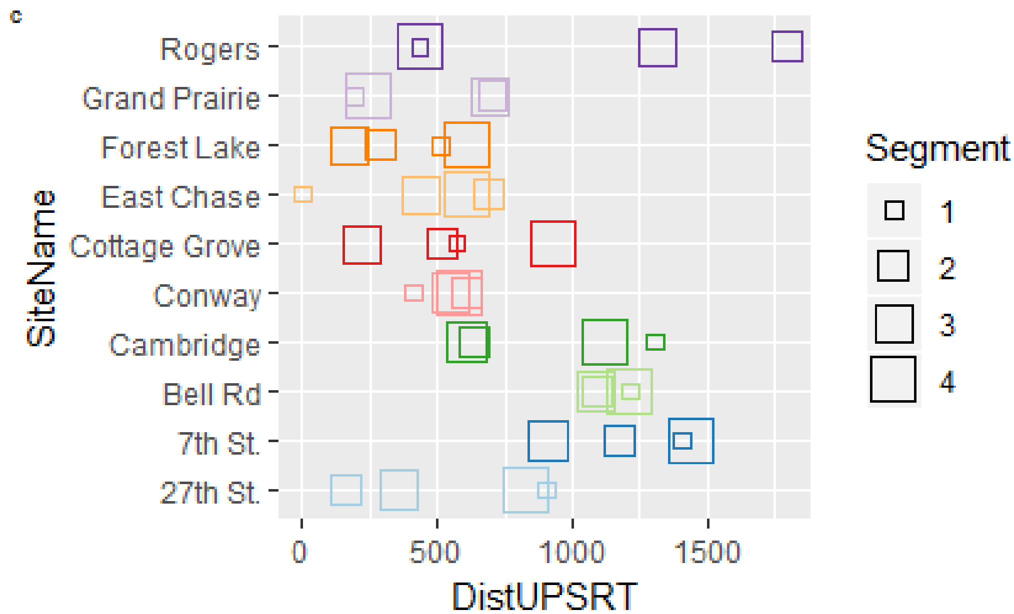

Figure 21 Distance of Upstream Intersection to the Right Turn Lane for Each Segment at Each Study Site

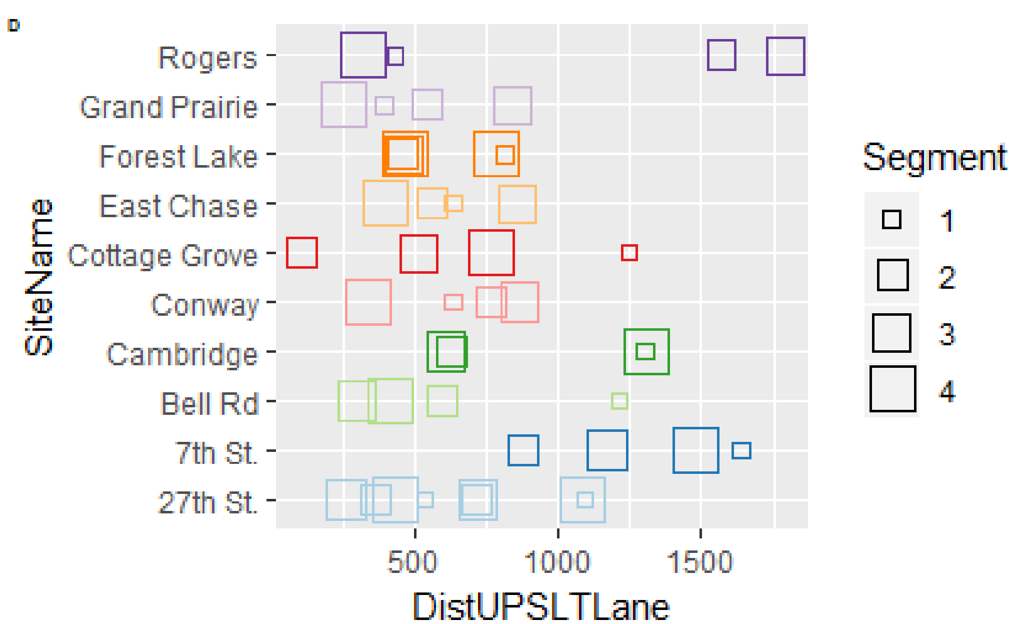

Figure 22 Distance of the Upstream Intersection to the Left-turn Lane for Each Segment at Each Study Site

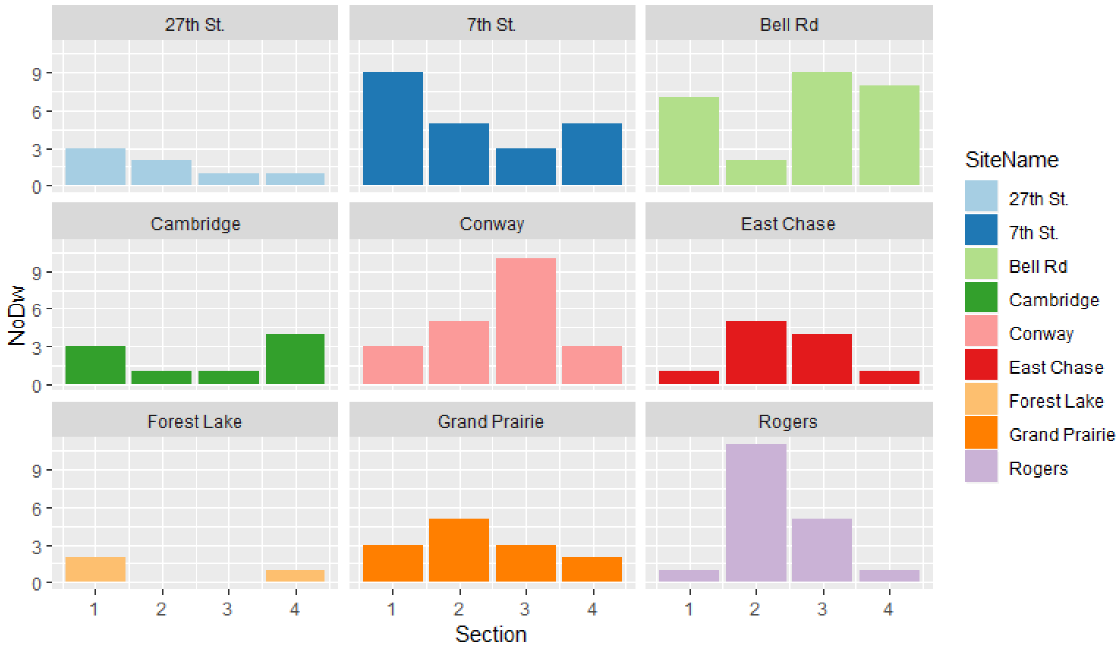

Figure 23 Number of Actual Driveways for Each Study Site

Table 14 Presence of Left Turning Lanes in Each Segment at Each Study Site

	Segment 1	**Segment 2**	**Segment3**	**Segment 4**
7th St.		✓		✓
27th St.		✓		✓
Bell Rd.	✓	✓	✓	
East Chase		✓		✓
Grand Prairie		✓		✓
Conway				✓
Rogers		✓		✓
Cambridge		✓		✓
Forest Lake		✓		✓
Cottage Grove		✓		✓

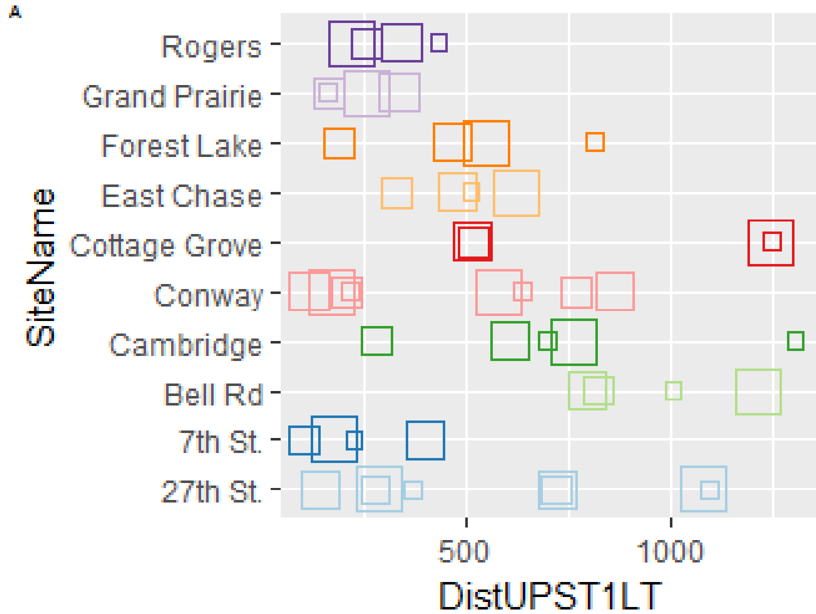

Figure 24 Distance of Upstream Intersection to the First Left Turning Point

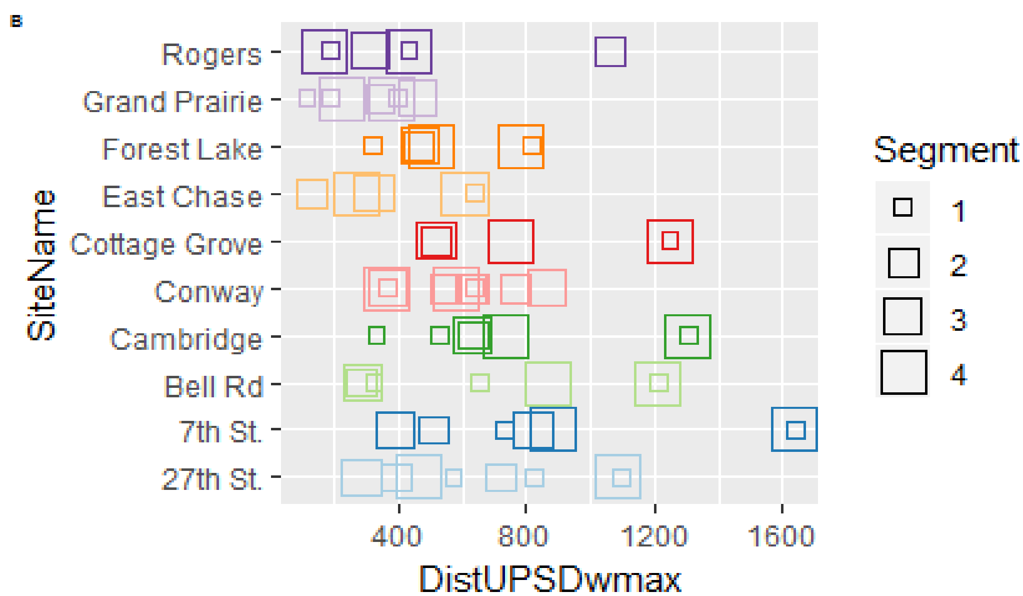

Figure 25 Distance of Upstream Intersection to the Largest Volume Driveway

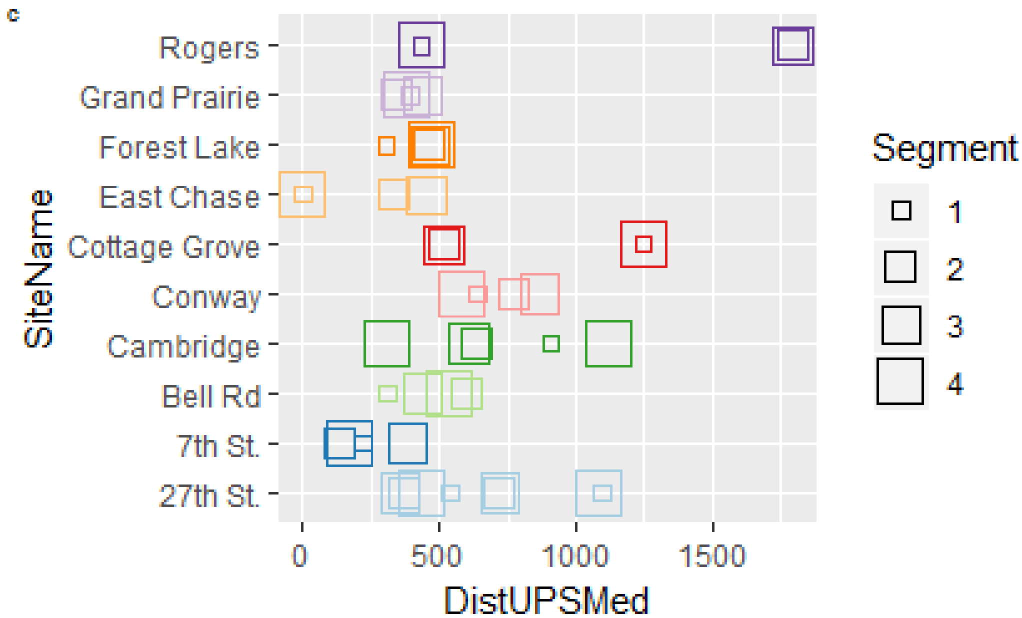

Figure 26 Distance of Upstream Intersection to the First Median Opening

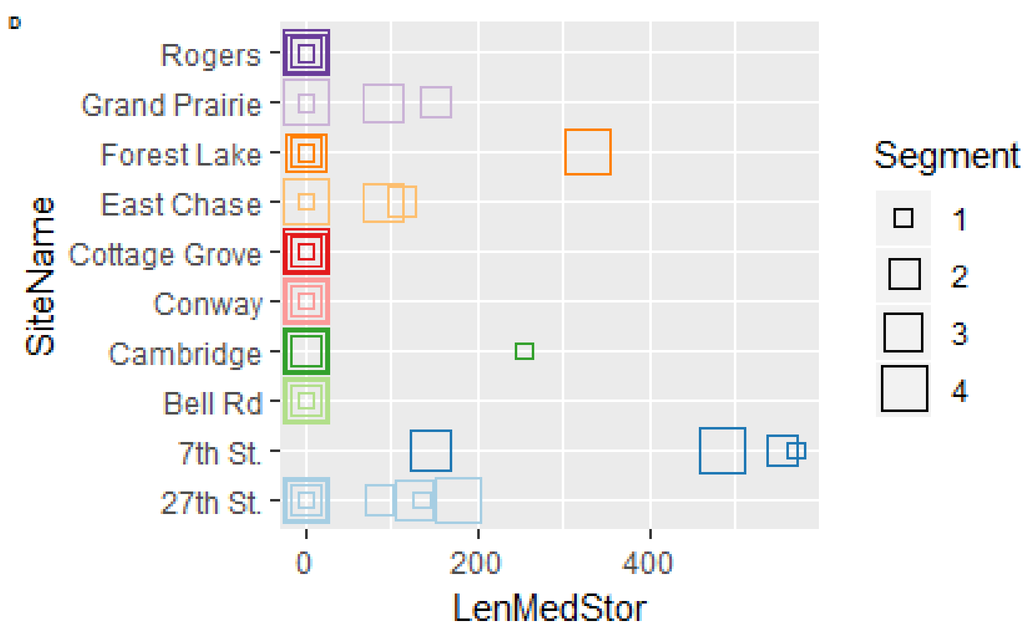

Figure 27 Length of Median Storage for Each Segment

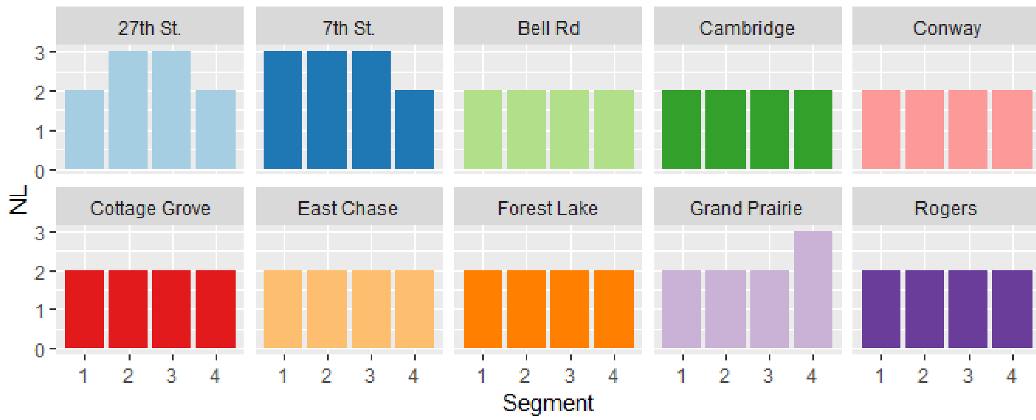

Figure 28 Number of Lanes in Each Segment in Each Study Site

Table 15 Presence of Right Turn Lane

	Segment 1	**Segment 2**	**Segment3**	**Segment 4**
7th St.	✓		✓	✓
27th St.	✓	✓	✓	✓
Bell Rd.				
East Chase	✓	✓	✓	
Grand Prairie	✓	✓	✓	✓
Conway	✓	✓	✓	
Rogers			✓	
Cambridge				✓
Forest Lake	✓	✓	✓	✓
Cottage Grove	✓		✓	✓

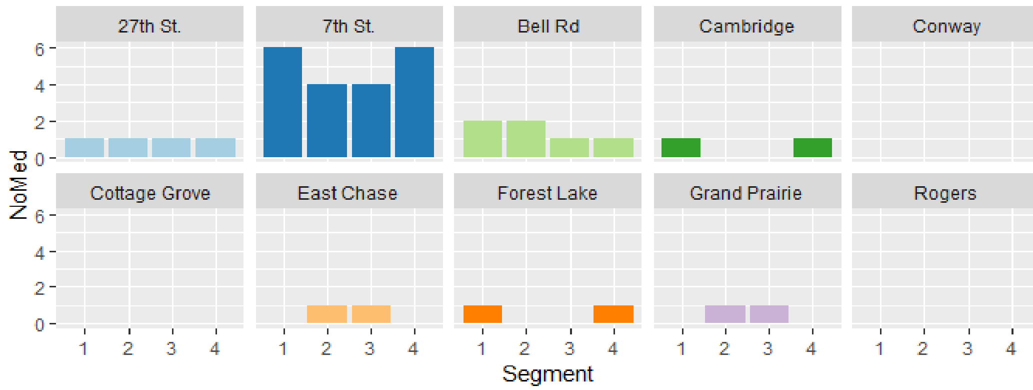

Figure 29 Number of Median Openings in Each Segment at Each Study Site

For each of the independent variables illustrated in above tables and figures, it can be seen that the study sites cover a wide range of characteristics. For example, the actual through volumes range from about 200 vph in Cottage Grove to around 1800 in 7th Street. Similarly, left turn and right turn volumes range from near zero to over 600 vph in different sites.

Creating different scenarios added more diversity to the study samples and made it possible to investigate unbuilt options. For example, Figure 20, "Distance of the First Driveway" shows the wide range of options created for the distance of the upstream intersection to the first driveway. Similar graphs for other distance measurements show the wide range of distance options covered in this study.

Some variables such as distance of the first signalized intersection were not changed by the scenario management plan and stayed constant for each study site. Despite staying constant, the study sample covered a wide range of signal distance options, from 375 feet to approximately 1800 feet.

Having a database with different variables, each covering a wide spectrum of possible and practical options, made it a good representative sample of urban arterials near diamond interchanges. Data from these samples was used to conduct analysis in the next chapter.

CHAPTER VI

ANALYSIS OF SIMULATION OUTPUT

The objective of this study was to take a comprehensive look at the impact of different design options on the operational performance of urban arterials near diamond interchanges. Field data were collected from 10 such sites around the U.S. These sites were then modeled in VISSIM, and those models were validated and calibrated. Then characteristics of the sites were modified and modeled so that 2232 scenarios were modeled and recorded. These results were then cleaned and used to build a multiple regression model to predict the number of lane changes. The multiple regression analysis was done using forward selection method to determine the preliminary independent variables and their coefficients. In the forward selection method, the first model is built without any of the independent variables. This means that at the first step of the forward selection method no predictions are made. At each step, one variable is added to the model until all independent variables are included. The independent variable that gives the best improvement to the fit (having the smallest RSS or highest R squared) is added at each step. The best model of each size is stored in a database. This process produces one model of each size (from one independent variable to the total number of independent variables). After different models with different sizes are made, the final selection among these models is done based on Adjusted R squared, Cp, or BIC. For the number of lane change models, table 16 shows the error criteria for the best model of each size and Figures 34-37 illustrate R squared, RSS, BIC, and Cp. These figures show as the number of variables increased the models performed more efficiently. Models of size 5, 6, or 7 were reasonably capable of predicting the number of lane changes without compromising performance criteria. Selecting a smaller number of independent variables makes it easier to understand and implement for the practitioner and decreases the amount of effort to collect site specific data.

Table 16 Error Criteria for Models per Model Size for NLC

Model size	R_Squared	RSS	AdjR_Squared	Cp	BIC
1	0.274	1.35E+08	0.273	2010.594	-212.842
2	0.466	99115131	0.464	1297.418	-422.143
3	0.575	78921521	0.573	892.670	-576.203
4	0.645	65814440	0.643	630.662	-697.684
5	0.705	54665718	0.703	408.099	-821.975
6	0.734	49365429	0.732	303.339	-887.317
7	0.755	45527245	0.752	228.029	-937.821
8	0.766	43477653	0.763	188.745	-963.738
9	0.777	41407492	0.774	149.047	-991.574
10	0.785	39858341	0.782	119.843	-1011.9
11	0.788	39304493	0.785	110.687	-1015.2
12	0.790	38917410	0.787	104.891	-1015.62
13	0.792	38496244	0.789	98.407	-1016.74
14	0.795	37933450	0.791	89.071	-1020.56
15	0.797	37692760	0.792	86.223	-1018.49
16	0.799	37278768	0.794	79.884	-1019.72
17	0.801	36866742	0.796	73.585	-1021
18	0.803	36622848	0.797	70.673	-1019.12
19	0.804	36329305	0.799	66.760	-1018.23
20	0.805	36085826	0.800	63.856	-1016.41
21	0.813	34673282	0.807	37.404	-1038.01
22	0.815	34375627	0.809	33.408	-1037.53
23	0.816	34075417	0.810	29.361	-1037.15
24	0.817	33910771	0.811	28.045	-1034.01
25	0.819	33605146	0.812	23.889	-1033.83
26	0.819	33553046	0.812	24.839	-1028.37
27	0.819	33512846	0.812	26.030	-1022.66
28	0.819	33511377	0.812	28.000	-1016.13

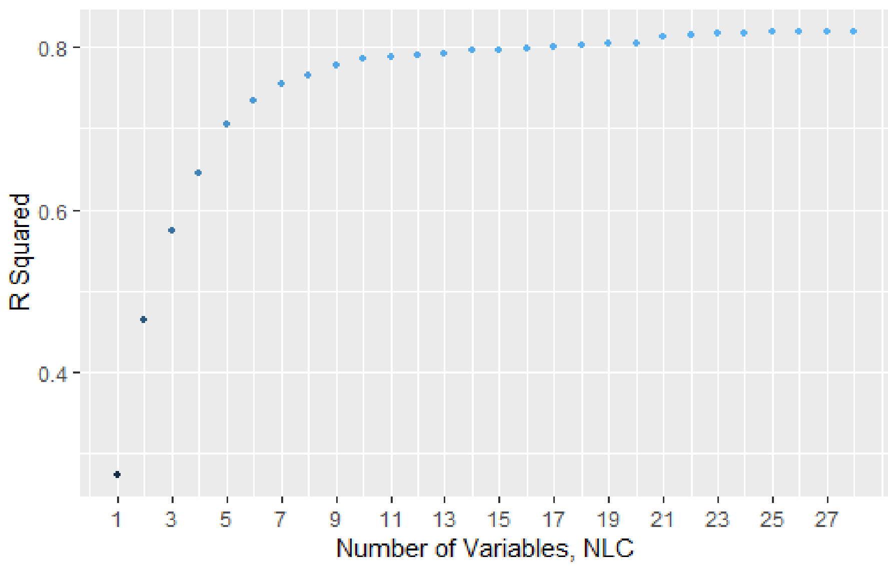

Figure 30 R Squared Values for Different Model Sizes - Number of Lane Changes

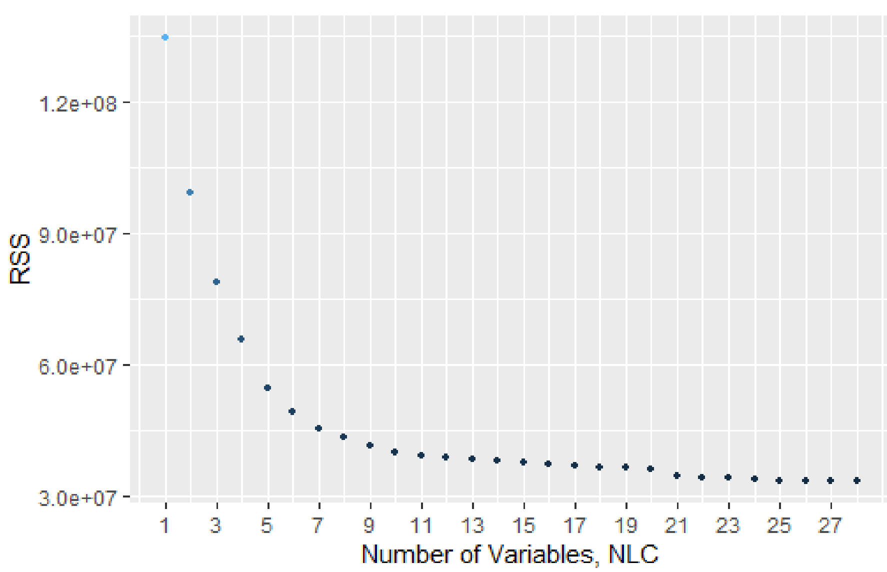

Figure 31 RSS Values for Different Model Sizes - Number of Lane Changes

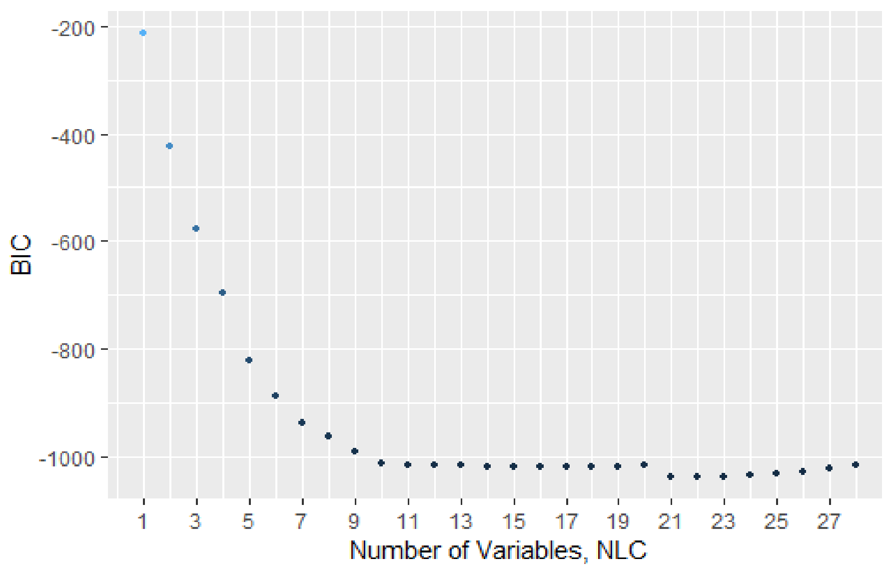

Figure 32 BIC Values for Different Model Sizes - Number of Lane Changes

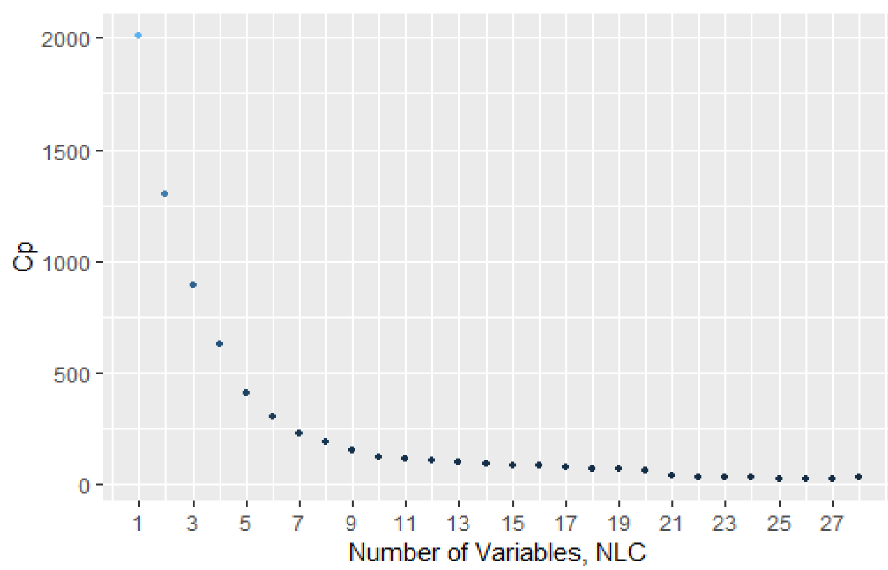

Figure 33 Cp Values for Different Model Sizes - Number of Lane Changes

Tables 17, 18, and 19 illustrate models with 5, 6, and 7 variables respectively.

Table 17 Size 5 Regression Model for Number of Lane Changes

Variable	Estimate	Std. Error	T value	Pr (>|t|)
Intercept	25.2658	32.2155	0.784	0.43309
SimVolTru	0.26131	0.02356	11.091	<2e-16
SimVolLT	2.18954	0.09771	22.408	<2e-16
SimVolRT	1.761	0.09007	19.552	<2e-16
DistUPSRT	-0.1373	0.03605	-3.81	0.00015
NoMed	75.0619	9.51994	7.885	9.35E-15

Table 18 Size 6 Regression Model for Number of Lane Changes

Variable	Estimate	Std. Error	T value	Pr (>|t|)
(Intercept)	48.68171	30.98231	1.571	0.116
SimVolTru	0.249729	0.02261	11.043	<2e-16
SimVolLT	2.245397	0.09384	23.928	<2e-16
SimVolRT	1.740492	0.08634	20.160	<2e-16
NoDw	54.49867	6.13079	8.889	<2e-16
DistUPSRT	-0.24287	0.03653	-6.649	5.18e-11
NoMed	79.10744	9.13371	8.661	<2e-16

Table 19 Size 7 Regression Model for Number of Lane Changes

Variable	Estimate	Std. Error	T value	Pr (>|t|)
(Intercept)	37.78651	35.69098	1.059	0.290
SimVolTru	0.252381	0.02303	10.960	<2e-16
SimVolLT	2.209709	0.11033	20.028	<2e-16
SimVolRT	1.750425	0.08786	19.923	<2e-16
NoDw	54.25839	6.14537	8.829	<2e-16
DistUPSRT	-0.24028	0.03678	-6533	1.09e-10
NoMed	77.85012	9.36247	8.315	3.49e-16
NoLTLanes	16.19293	23.30527	0.616	0.538

Since the p-value in the Size 7 repression model indicates that this variables is not significant, this variables should be removed from the model and should not be used to derive conclusions. Moreover, although the p-values for the Intercept in all three models indicated that they should be removed from the models, it is better to keep them, as removing them would result in a model that underestimates the number of lane changes.

The next step in the analysis was to model travel time based on the same independent variables included in modeling the number of lane changes but adding number of lane changes as an independent variable. This step was done to investigate if number of lane changes had a correlation with travel time in urban arterials near diamond interchanges. Similar to modeling the number of lane changes, a forward stepwise variable selection method was used. As noted in modeling the number of lane changes, forward selection method starts with no prediction and one variable that best improves model performance is added at each step until all variables are included in the last model. After finding the best model for each model size, Adjusted R squared, Cp, or BIC criteria is used to select the best model. This study was conducted to provide guidelines for practitioners and enable them to improve performance of urban arterials near diamond interchanges, thus it is important to keep models simple and comprehensible. Table 20 shows error criteria for each model size for travel time. Figures 36-39 illustrate R squared, RSS, BIC, and Cp for each model size. As seen in figures 36-39, even with smaller number of independent variables travel time regression models had high performance.

Table 20 Error Criteria for Models per Model Size for TT

Number of Variables	R Squared	RSS	Adj R Squared	Cp	BIC
1	0.446	533900.9	0.445	51044.10	-3713.16
2	0.919	78299.02	0.919	2093.86	-15846.4
3	0.924	72912.65	0.924	1517.12	-16288.4
4	0.928	69077.67	0.928	1107.07	-16621.4
5	0.930	67157.64	0.930	902.77	-16790.9
6	0.932	65839.16	0.932	763.10	-16907.6
7	0.932	65239.34	0.932	700.66	-16956.7
8	0.933	64652.03	0.933	639.55	-17005.2
9	0.934	63477.04	0.934	515.30	-17112.4
10	0.935	62869.43	0.935	452.02	-17164.5
11	0.935	62433.27	0.935	407.16	-17199.8
12	0.936	62069.64	0.935	370.09	-17228
13	0.936	61571.38	0.936	318.55	-17270.2
14	0.936	61238.59	0.936	284.79	-17295.7
15	0.937	60899.92	0.937	250.41	-17322.1
16	0.937	60483.69	0.937	207.68	-17356.7
17	0.937	60300.44	0.937	189.99	-17367.1
18	0.938	59925.72	0.938	151.73	-17397.8
19	0.938	59354.54	0.938	92.36	-17449.6
20	0.939	58974.36	0.939	53.51	-17481.5
21	0.939	58770.65	0.939	33.63	-17494.7
22	0.939	58662.2	0.939	23.97	-17497.6
23	0.939	58618.37	0.939	21.26	-17493.6
24	0.939	58589.65	0.939	20.18	-17487.9
25	0.939	58580.66	0.939	21.21	-17480.1
26	0.939	58577.56	0.939	22.88	-17471.7
27	0.939	58574.32	0.939	24.53	-17463.3
28	0.939	58571.21	0.939	26.20	-17454.9
29	0.939	58569.38	0.939	28.00	-17446.3

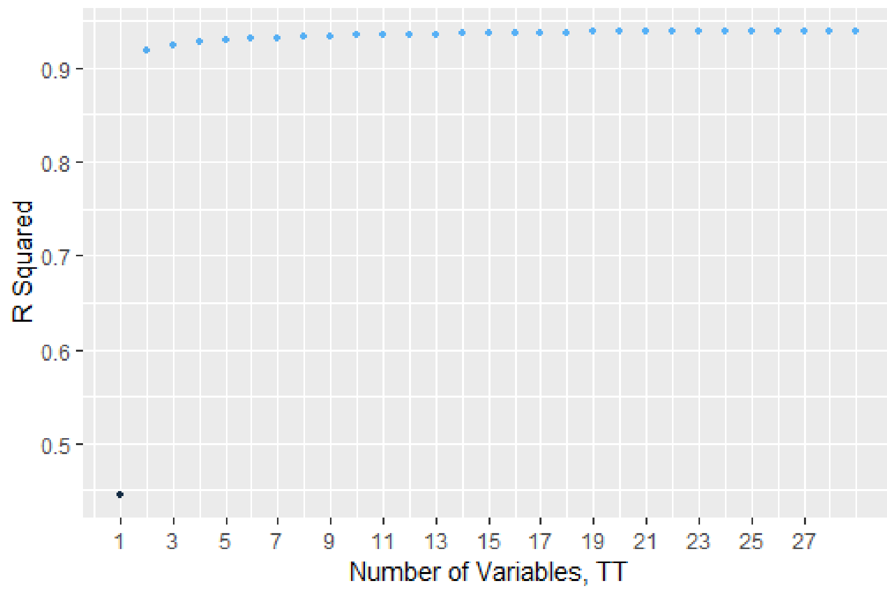

Figure 35 R Squared Values for Different Model Sizes – Travel Time

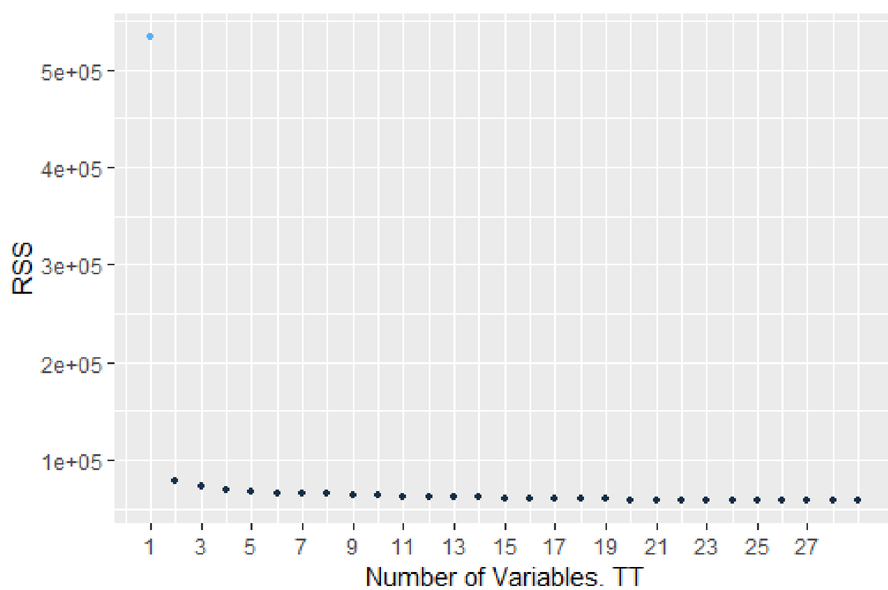

Figure 34 RSS Values for Different Model Sizes – Travel Time

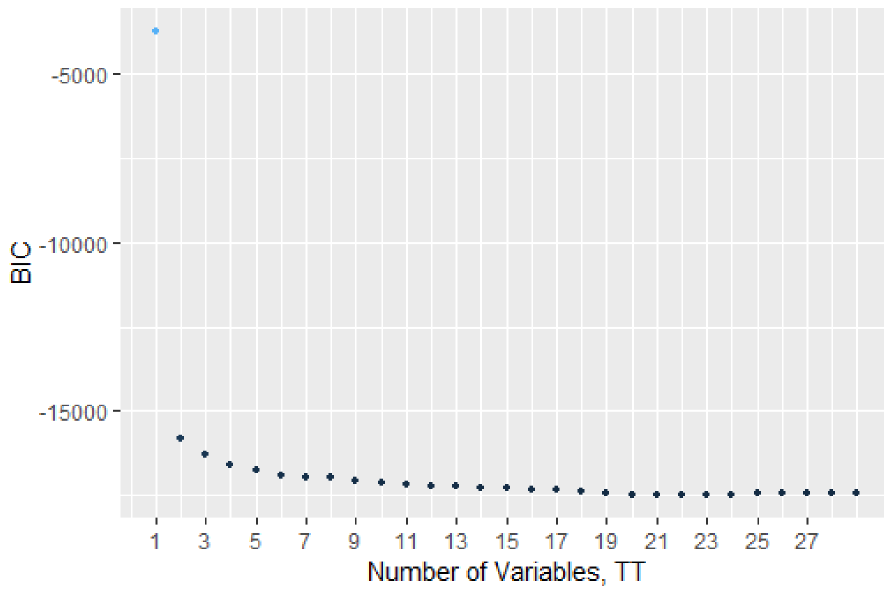

Figure 36 BIC Values for Different Model Sizes – Travel Time

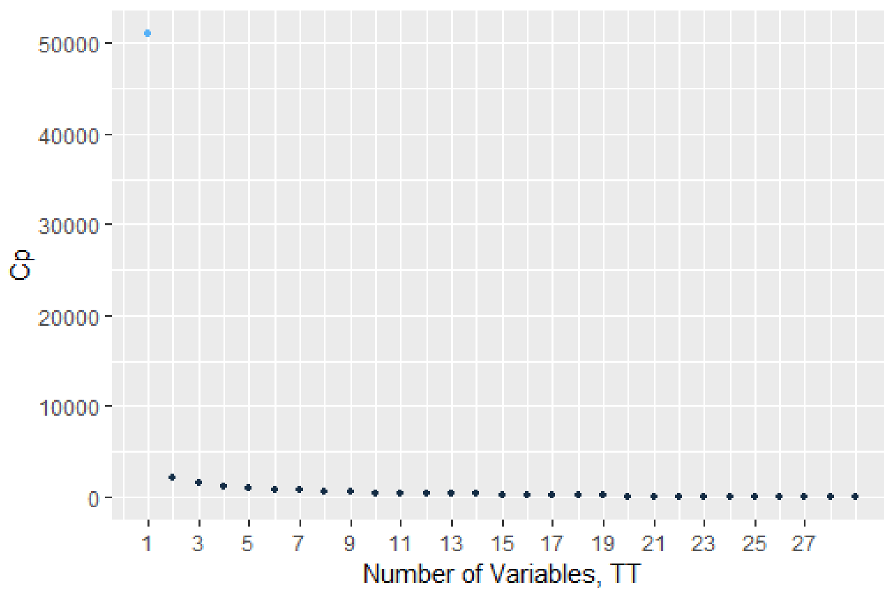

Figure 37 Cp Values for Different Model Sizes – Travel Time

Tables 21, 22, and 23 show model specifications for the travel time model with 3, 4, and 5 independent variables respectively. The models with size 4 and 5, both included number of lane changes as one of the statistically significant independent variables and this proved that number of lane changes impacts travel time on urban arterials near diamond interchanges.

Table 21 Size 3 Regression Model for Travel Time

Variable	Estimate	Std. Error	T value	Pr (>|t|)
(Intercept)	2.849	0.259	11.02	<2e-16
DistSig	0.020	0.000	79.46	<2e-16
DenDw	0.262	0.011	23.44	<2e-16
Upstream1	5.930	0.202	29.41	<2e-16

Table 22 Size 4 Regression Model for Travel Time

Variable	Estimate	Std. Error	T value	Pr (>|t|)
NLC	0.006	0.0002	33.246	<2e-16
DistSig	0.020	0.0002	82.357	<2e-16
DenDw	0.189	0.010	17.619	<2e-16
Upstream1	6.239	0.189	32.989	<2e-16

Table 23 Size 5 Regression Model for Travel Time

Variable	Estimate	Std. Error	T value	Pr (>|t|)
(Intercept)	-2.593	0.327	-7.931	2.48e-15
NLC	0.005	0.000	30.425	<2e-16
DistSig	0.020	0.000	83.655	<2e-16
DenDw	0.170	0.107	15.892	<2e-16
Upstream1	8.754	0.274	31.980	<2e-16
NoLTLanes	3.345	0.266	12.596	<2e-16

As seen in Tables 21-23, volumes (through, left turning and right turning) were present in all regression models for the number of lane changes. This showed that, as expected, the number of vehicles have a positive correlation with the number of lane changes. Aside from volume, distance of upstream intersection to the beginning of right turning lane had a negative correlation with the number of lane changes in all three models. This implied that providing longer right turning lanes decreased the number of lane changes at these locations. The number of median openings had a large positive correlation with the number of lane changes. Based on the models, each median opening increased the number of lane changes by 75-79. This finding was in accordance with other studies on median openings.

Size 6 and size 7 models included the number of driveways and it can be seen that each driveway increased the number of lane changes by 54. **This showed that providing access to land uses on urban arterials near diamond interchanges should be done with care.** The number of left turning lanes was present in the model with 7 independent variables and had a positive correlation with the number of lane changes. This fact meant that in segments where a left turning lane was present, more vehicles conducted lane change maneuvers. In other words, the left turning lanes were installed because there were enough left turning vehicles to warrant it. Therefore the higher number of lane changes was not caused by the number of left turning lanes. Instead, high number of left turning maneuvers was caused by the high number of left turning vehicles.

Since proving an auxiliary left turning lane at intersections usually has other design criteria including MUTCD guidelines, it was not reasonable to include the number of left turning lanes in the final model. Moreover, the p-value for the variable representing the number of left turning lanes, implied that the impact of this variable is not significantly higher than zero. Based on these points, the model with 6 independent variables was the best model for the number of lane changes.

For travel time, the distance between the upstream and downstream signalized intersections and the density of driveways were two independent variables included in all three models. While the number of lane changes was added to the model in step 4 of the forward stepwise selection it was also included in the size 5 model. One other variable that was present in all three models was a binary variable that was 1 for segments that were located upstream to the terminal intersection and 0 for the segments located downstream to the terminal intersection. The results showed that segments located upstream to the terminal intersection had 6-9 seconds higher travel times compared to similar segments located downstream to the terminal intersection.

Moreover, the number of left turning lanes was present in the size 5 model. Similar to the number of lane changes models, since providing left turning lanes is usually inevitable, it should not be present in the final model and the model with 4 variables was the best model for travel time.

CHAPTER VII

CONCLUSIONS

The objective of this research was to investigate the impact of design characteristics on the operational performance of urban arterials near freeway interchanges. This study started with collecting and reducing video data and design variables from 10 study sites and then building simulation models in a simulation interface. A simulation model was built for each study site and calibrated to ensure accuracy and precision. Different options for volume, the number of driveways, median, and control type of the right turning movement at the terminal intersection replaced the original setup at each site to build different scenarios. After running 10 simulations for each scenario, the R programming language was used to process simulation outputs and build a database. After cleaning the final database in the data cleaning process, a regression analysis was conducted. A multiple regression model consists of an intercept and a coefficient for each variable in the model. The coefficient value implies how much the response value would change if the variable increased one unit while all other variables stayed unchanged. Since traffic flow patterns may change in different states of traffic, volume was added to the independent variables. Instead of adding one volume variable, through, left-turning and right turning volumes were adjusted to account for different paths of traffic flow.

The first analysis step in this study was to evaluate the impact of design variables, namely the number of driveways, median characteristics and impact of control type of the right turning movements at the terminal intersection, on the number of lane changes. To select the best model among all possible regression models, a forward stepwise model selection method was used. The forward stepwise selection method provides the best model for each model size (size represents the number of independent variables in the regression model), and the researcher makes the final choice of model size based on model performance criteria, experience and study conditions.

As discussed in the results section, this study aimed to provide guidelines for the practitioners to improve performance of urban arterials near diamond interchanges. A well performing regression model with a smaller number of independent variables makes the model more practical and efficient.

To model the number of lane changes, three models with five, six and seven variables that had good performance were selected and studied for final model selection. Fit statistics for these models are listed in Table 24 and show that all models performed well in predicting the response values.

Table 24 Fit Statistics for NLC Models with 5, 6, and 7 Independent Variables

Fit Measure	5 variables	6 variables	7 variables
RSME	127452	120887	121277
R_Squared	0.705	0.734	0.755
RSS	5.5E+07	4.9E+07	4.6E+07
AdjR_Squared	0.703	0.732	0.752
Cp	408.099	303.339	228.029
BIC	-821.98	-887.32	-937.82

Table 25 shows which variables were selected for each model size. It can be seen that volume, number of median openings and distance of the upstream intersection to the beginning of the right turning lane were present in all models. Number of driveways was present in models with 6 and 7 variables and number of left turning lanes was only included in the model with 7 variables. Providing left turning lanes in intersection design has other design criteria such as the percentage of left turning movements at an approach, and it is not practical to include it in the final model. On the other hand, number of driveways, number of median openings and distance of the upstream intersection to the beginning of the right turn lane are design variables that could be adjusted during planning stages. For this reason, the model with six variables was selected.

$$NLC = 48.68 + 0.25 \times \text{SimVolTru} + 2.24 \times \text{SimVolLT} + 1.70 \times \text{SimVolRT} + 54.5 \times NoDW - 0.24 \times \text{DistUPSRT} + 79.1 \times \text{NoMed} \quad (18)$$

where NLC is the number of lane changes, SimVolTru is the through traffic volume a, SimVolLT is the left-turning volume, and SimVolRT is the right turning volume all taken from the simulation results. NoDw is the number of driveways along the study segment, DistUPSRT is the distance of the right turning lane from the upstream intersection, and NoMed is the number of median openings.

Table 25 Variables Present at Each NLC Model Size

Variable	**5 variables**	**6 variables**	**7 variables**
SimVolTru	✓	✓	✓
SimVolLT	✓	✓	✓
SimVolRT	✓	✓	✓
NoDw		✓	✓
DistUPSRT	✓	✓	✓
NoMed	✓	✓	✓
NoLTLanes			✓

Raised medians have been proven to improve the safety of urban roads, and our study results showed they greatly reduced lane changes. The number of median openings had the largest coefficient (79.10), among all variables. This meant that, keeping all other factors unchanged, each median opening contributed to 79 lane change maneuvers. On the other hand, while the location of the first driveway or the location of the largest volume driveway were not in our final model, the number of driveways largely impacted the NLC with a coefficient of 54.5. This shows that giving multiple access locations among the design segments should be conducted with care, and it is recommended to keep the number of driveways to a minimum.

Another interesting observation was the reverse impact of the distance of the upstream intersection to the right turning lane. A shorter distance leads to a greater number of lane changes and this shows that locating exclusive right turning lanes too close to the terminal intersection increased the number of lane changes on the entire segment.

Finally, the number of left-turning lanes increased the number of lane changes, but this observation probably should not be used as a design recommendation. The fact that the number of left-turning lanes was among the highest contributing variables likely indicated there were many left-turning vehicles necessitating exclusive left-turning lanes, and therefore many vehicles needing to change lanes. For the travel time model, three models with 3, 4, and 5 variables were chosen to select the final model from. Table 26 shows the fit statistics for these models.

Table 26 Fit Criteria for TT Models with 3, 4, and 5 Independent Variables

Fit Measure	3 variables	4 variables	5 variables
RSME	64.8868	62.8111	60.0203
R_Squared	0.924	0.928	0.93
RSS	72912.7	69077.7	67157.6
AdjR_Squared	0.924	0.928	0.93
Cp	1517.12	1107.07	902.77
BIC	-16288	-16621	-16791

Selected variables for each model size are summarized in Table 27.

Table 27 Selected Variables for Each TT Model Size

Variable	3 variables	4 variables	5 variables
NLC		✓	✓
DistSig	✓	✓	✓
DenDw	✓	✓	✓
Upstream1	✓	✓	✓
NoLTLanes			✓

As discussed in selecting the best model for the number of lane changes, the presence of the number of left turn lanes in the model with five variables indicated that there were higher number of left tuning vehicles necessitating left turn lanes. For this reason, the model with four variables was chosen as the final model for travel time and the number of left turning lanes was excluded from the model.

$$TT = -0.007 + 0.006 \times NLC + 0.02 \times DistSig + 0.19 \times DenDw + 6.24 \times Upstream1 \quad (19)$$

where TT is travel time, NLC is the number of lane changes, DistSig is the distance of the upstream intersection to the first signalized intersection, DenDw is the density of driveways (number of driveways/segment length), and Upstream is a binary variable indicating a segment is located upstream to the freeway on-ramp (Upstream =1) or downstream to the freeway off-ramp (Upstream= 0), and NoLTLanes is the number of left-turning lanes.

Travel time was measured in seconds in this study, and as expected the number of lane changes was one of the variables in two of the models. This indicated that the factors that contribute to the number of lane changes indirectly impact travel time. The density of the driveways also had a positive relationship with the travel time, and this again indicated that the higher the number of driveways, the higher the travel time. During the variable selection method, there were some observations that are noteworthy.

1- Although the two final models for the number of lane changes and travel time did not have any variables in common, the characteristics of driveways showed up in both models. Number of driveways was present in the final model for the number of lane changes, while density of driveways showed up in the final travel time model. Moreover, the number of lane changes was present in the travel time model and this indicated that the contributing factors to the number of lane changes, indirectly contribute to travel time.
2- Driveway volume, driveway width, nor location of the driveways had a strong correlation with either the number of lane changes or travel time. While this may seem counterintuitive, the fact that other driveway characteristics were included

in the final models proved that number of access points had greater impact on the performance of these facilities than other characteristics of those access points.

3- The type of median was not a significant factor in either of the models. There were six independent variables that captured median characteristics. Median type, a binary variable to represent if a median was continuous or not, the length of median storage, distance of the upstream intersection to the median opening, a binary variable to represent if there was a median storage and the length of the median storage. Despite other studies signify improved safety with raised medians, our results showed that as long as the median was uniform along the segment, number of lane changes or travel time were not impacted.

4- This study only included study sites with two lanes in each direction which is the most common number of lanes for urban arterials near diamond interchanges. These results may not hold if the number of lanes is not two per direction.

5- Urban arterials near diamond interchanges are the transition points in an urban environment. To prepare urban infrastructure for automated and connected vehicles similar studies to this study should be done by introducing connected and automated vehicles to the system and studying how these facilities can best accommodate AVs and CVs.

This study tried to take a comprehensive look into the impact of design characteristics on the operational performance of urban arterials near diamond interchanges, it used VISSIM as the simulation tool. In VISSIM, car following behavior can be adjusted to reflect the driving behavior at each site. In this study, driving behavior were not adjusted and all simulations were run under default driver behavior settings. More research can be done to collect driver behavior characteristics at each site and use the results to identify risky traits that might have a negative impact on the operational performance of urban arterials.

Another important issue in the vicinity of urban arterials near diamond interchanges is the presence of heavy vehicles. While the proportion of heavy vehicles in the traffic flow was not a design characteristic and was not included in this study it can be studied independently. The results of such study can be used for transit and freight planning in a way that heavy vehicles do not impose a negative impact on urban arterials.

As noted before, almost all of the study sites in this research had two lances per direction. An extension to this study can identify urban arterials with fewer or more lanes and study the impact of design characteristics on them. There were other limitations to the model parameters that should be taken into considerations. For example, the distance between the terminal intersection and the first signalized intersection was between 375 and 1792 feet. Using the outcomes of this research for longer or shorter corridors should be done with care and it is recommended that the models be validated for other segment lengths before using them for decision making. As another example, the maximum driveway volume in this study was 268 vph. If there are driveways with significantly higher volumes, the models presented in this research should be validated before being used. Other limitations and boundaries for model parameters can be found in Table 11.

VII. 1 **Applications**

This section provides examples of how findings of this research can be implemented:

1- For segments were there is an operational performance issue such as unreasonable travel times, it is recommended that the number of median openings and the number of access points are studied. Although the number of median openings had a larger coefficient in the number of lane changes model, closing (combining) driveways might be the first step. This recommendation is based on the fact that taking another access point at the same side of the road might not increase the travel time for the vehicles that preferred the closed driveway, but closing the median opening, would significantly increase the travel time for the vehicles that use that median opening.

2- If closing (combining) driveways was not enough to improve the operational performance of traffic, the next step is to close median openings. This might increase the travel time for some users, but the total decrease in travel time for the users, would compensate for that loss.

3- The next step is to investigate the right turning lane configurations and try to provide a right turning lane starting from the beginning of the segment, or only provide a right turning bay that is long enough to accommodate the first signalized intersection.

4- For segments that are being designed as new facilities, keeping the number of driveways and the number of median openings to a minimum makes the traffic on the new facility operate optimally. Moreover, it is recommended that if providing several driveways is required, an overall right turning lane be provided. This can provide smooth access to and from the driveways without introducing disturbance the main traffic flow.

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APPENDIX A

TRAVEL TIME EXTRACTION FROM GPS FILES

The following steps were taken to derive travel time information from the files the GPS device saved.

```
################Deriving Travel Time from GPS log files ################
rm(list = ls()) #Clear memory
library(lubridate)   # Install package before running library command
setwd("C://......")#Set the directory where log files are located.
startingdir <- ("C://...")  #Set the starting directory
files <- list.files(startingdir) # Set the input files.

#Build the data frame with specific filed types
sec1time<- data.frame(Name=character(),
                      StartTime=integer(),
                      EndTime=integer(),
                      TimeDifference=character(),
                      stringsAsFactors=FALSE)

sec2time<-data.frame(Name=character(),
                     StartTime=integer(),
                     EndTime=integer(),
                     TimeDifference=character(),
                     stringsAsFactors=FALSE)
```

```
sec3time<-data.frame(Name=character(),
                     StartTime=integer(),
                     EndTime=integer(),
                     TimeDifference=integer(),
                     stringsAsFactors=FALSE)

sec4time<-data.frame(Name=character(),
                     StartTime=integer(),
                     EndTime=integer(),
                     TimeDifference=character(),
                     stringsAsFactors=FALSE)

#Read files one by one and remove unnecessary points in each file based on longitude (latitude is almost
the same for start and end points)
for ( i in 1:length(files)){
  df<- read.table(files[i],sep=",",fill = TRUE, skip=7, header=TRUE)
  df$SystemTime.hh.mm.ss.ms<-  sub("(.*):", "\\1.", df$SystemTime.hh.mm.ss.ms)  #To convert travel
time type (millisecond section)
  sec1=subset(df, df$SiteName== "WB Site Name" & df$Lon.DD.DDDD.>=-91.000000 &
df$Lon.DD.DDDD. <=-91.999999)   #For Site 1
  sec2=subset(df, df$SiteName== "WB Site Name" & df$Lon.DD.DDDD.>=-92.000000 &
df$Lon.DD.DDDD. <=-92.999999)   #For Site 1
  sec3=subset(df, df$SiteName== "EB Site Name" & df$Lon.DD.DDDD.>=-93.000000 &
df$Lon.DD.DDDD. <=-93.999999)   #For Site 1
  sec4=subset(df, df$SiteName== "EB Site Name" & df$Lon.DD.DDDD.>=-94.000000 &
df$Lon.DD.DDDD. <=-94.999999)   #For Site 1
```

```
  if(nrow(sec1)>0){
    sec1time[i, 1]<-paste0(sec1[1,2])
    sec1time[i, 2]<-sec1$SystemTime.hh.mm.ss.ms[1]
    sec1time[i, 3]<-sec1$SystemTime.hh.mm.ss.ms[(nrow(sec1))]

    sec2time[i, 1]<-paste0(sec2[1,2])
    sec2time[i, 2]<-sec2$SystemTime.hh.mm.ss.ms[1]
    sec2time[i, 3]<-sec2$SystemTime.hh.mm.ss.ms[(nrow(sec2))]

  } else if (nrow(sec3)>0){

    sec3time[i, 1]<-paste(sec3[1,2])
    sec3time[i, 2]<-sec3$SystemTime.hh.mm.ss.ms[1]
    sec3time[i, 3]<-sec3$SystemTime.hh.mm.ss.ms[(nrow(sec3))]

    sec4time[i, 1]<-paste(sec4[1,2])
    sec4time[i, 2]<-sec4$SystemTime.hh.mm.ss.ms[1]
    sec4time[i, 3]<-sec4$SystemTime.hh.mm.ss.ms[(nrow(sec4))]

  }
}

#Calculate travel time
sec1time$StartTime<-strptime( sec1time$StartTime, "%H:%M:%OS")  #Conver the Start Time field type to HHMMSS
sec1time$EndTime<-strptime( sec1time$EndTime, "%H:%M:%OS")      #Conver the End Time field type to HHMMSS
```

```
sec1time$TimeDifference<-as.numeric(sec1time$EndTime-sec1time$StartTime)  #Calculate time
difference

sec2time$StartTime<-strptime( sec2time$StartTime, "%H:%M:%OS")
sec2time$EndTime<-strptime( sec2time$EndTime, "%H:%M:%OS")
sec2time$TimeDifference<-as.numeric(sec2time$EndTime-sec2time$StartTime)

sec3time$StartTime<-strptime( sec3time$StartTime, "%H:%M:%OS")
sec3time$EndTime<-strptime( sec3time$EndTime, "%H:%M:%OS")
sec3time$TimeDifference<-as.numeric(sec3time$EndTime-sec3time$StartTime)

sec4time$StartTime<-strptime( sec4time$StartTime, "%H:%M:%OS")
sec4time$EndTime<-strptime( sec4time$EndTime, "%H:%M:%OS")
sec4time$TimeDifference<-as.numeric(sec4time$EndTime-sec4time$StartTime)

#Combine four outputs
bigdf<- rbind(sec1time, sec2time, sec3time, sec4time)
output=paste0("Site1TravelTimes",".csv")

#Write the file
write.table(bigdf, file=output,sep=",", row.names=FALSE)
```

APPENDIX B

MODEL VALIDATION

This appendix includes model validation results for all study sites except Cambridge. For the number of lane changes, only segments where the camera coverage permitted collecting lane change data, the segment was validated. For travel time if a minimum of three segments were valid, the model was assumed to be valid.

7th St.

Table 28 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for 7th St.

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.94796	0.6444	No
Segment 2	0.89595	0.3506	No
Segment 3	0.86764	0.09382	No
Segment 4	0.94493	0.6091	No

Table 29 Wilcoxon Rank Sum Test Result Summary for 7th St.

Measurement vs. Simulation	W	p-value	Valid?
Segment 1	20	0.6	Yes
Segment 2	7	0.4	Yes
Segment 3	20	0.6	Yes
Segment 4	9	0.1	Yes

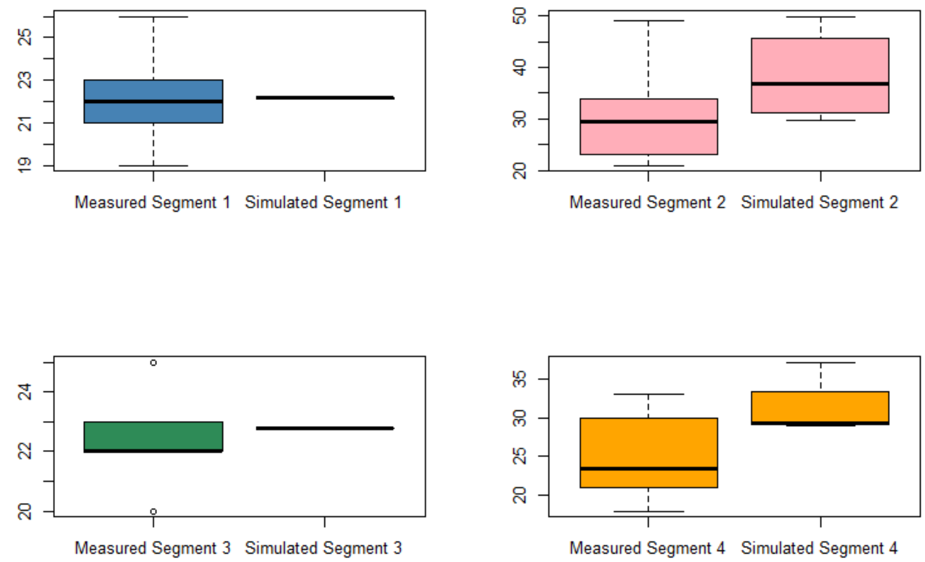

Figure 38 Box Plots of Measured and Simulated Travel Times for 7th St.

Table 30 Validation of Number of Lane Changes for 7th St.

	Segment 1	**Segment 2**	**Segment 3**	**Segment 4**
Mean	508.2	316.4	656.2	141.9
Standard Error	5.751908	38.24924	5.879909	3.124989
Median	513	320	653.5	142
Mode	489	321	653	145
Standard Deviation	18.18913	120.9547	18.59391	9.882083
Sample Variance	330.8444	14630.04	345.7333	97.65556
Count	10	10	10	10
Upper Level	521.2117	402.9258	669.5013	148.9692
Lower Level	495.1883	229.8742	642.8987	134.8308
Measured	NA	NA	NA	136
Valid?				Yes

27^{th} Street

Table 31 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for 27^{th} St.

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.80118	0.02947	Yes
Segment 2	0.81862	0.04513	Yes
Segment 3	0.93849	0.6252	No
Segment 4	0.88575	0.2532	No

Table 32 T-Tests for 27^{th} St. Measured and Simulated Travel Times Segments 1, 2

	Segment 1 M	Segment 1 S	Segment 2 M	Segment 2 S
Mean	20.125	21.1075	62.375	70.535
Variance	4.696429	0.001758	506.5536	3.0233
Observations	8	4	8	4
Pooled Variance	3.288028		355.4945	
Hypothesized Mean Difference	0		0	
df	10		10	
t Stat	-0.88481		-0.70674	
P(T<=t) two-tail	0.397036		0.495867	
t Critical two-tail	2.228139		2.228139	

Table 33 Wilcoxon Rank Sum Test Result Summary for 27^{th} St. Segments 3, 4

Measurement vs. Simulation	W	p-value	Valid?
Segment 1	20	0.3	Yes
Segment 2	20	0.8	Yes

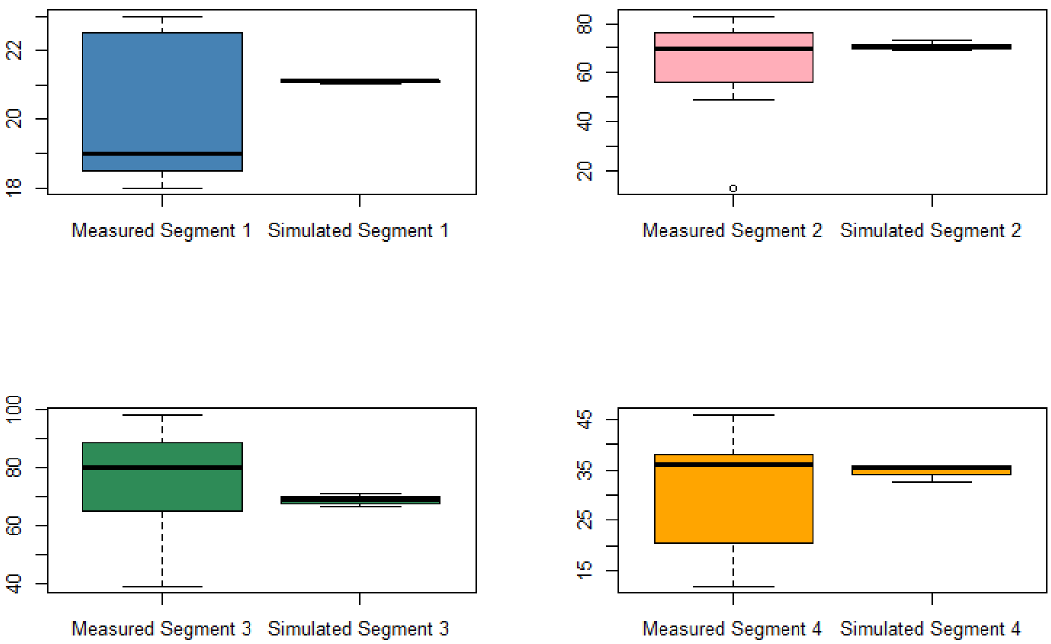

Figure 39 Box Plots of Measured and Simulated Travel Times for 27th St.

Table 34 Validation of Number of Lane Changes for 27th St.

	Segment 1	**Segment 2**	**Segment 3**	**Segment 4**
Mean	400.4	261.6	457.4	102.9
Standard Error	4.287449	2.809508	3.670301	1.545244
Median	403	261.5	453.5	102
Mode	413	#N/A	453	98
Standard Deviation	13.55811	8.884443	11.60651	4.886489
Sample Variance	183.8222	78.93333	134.7111	23.87778
Count	10	10	10	10
Upper Level	410.0989	267.9555	465.7028	106.3956
Lower Level	390.7011	255.2445	449.0972	99.40442
Measured	396	258	451	100
Valid?	Yes	Yes	Yes	Yes

Bell Road

Table 35 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for Bell Rd.

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.92709	0.4199	No
Segment 2	0.91645	0.3283	No
Segment 3	0.7714	0.006504	Yes
Segment 4	0.97729	0.949	No

Table 36 Wilcoxon Rank Sum Test Result Summary for Bell Rd. Segment 1, 2, 4

Measurement vs. Simulation	W	p-value	Valid?
Segment 1	20	0.7	Yes
Segment 2	20	0.8	Yes
Segment 4	4	0.02	No

Table 37 T-Tests for 27th Street Measured and Simulated Travel Times Segment 3

	Segment 3 M	Segment 3 S
Mean	27.1	30.2225
Variance	138.5444	0.110425
Observations	10	4
Pooled Variance	103.9359	
Hypothesized Mean Difference	0	
df	12	
t Stat	-0.51771	
P(T<=t) two-tail	0.614072	
t Critical two-tail	2.178813	

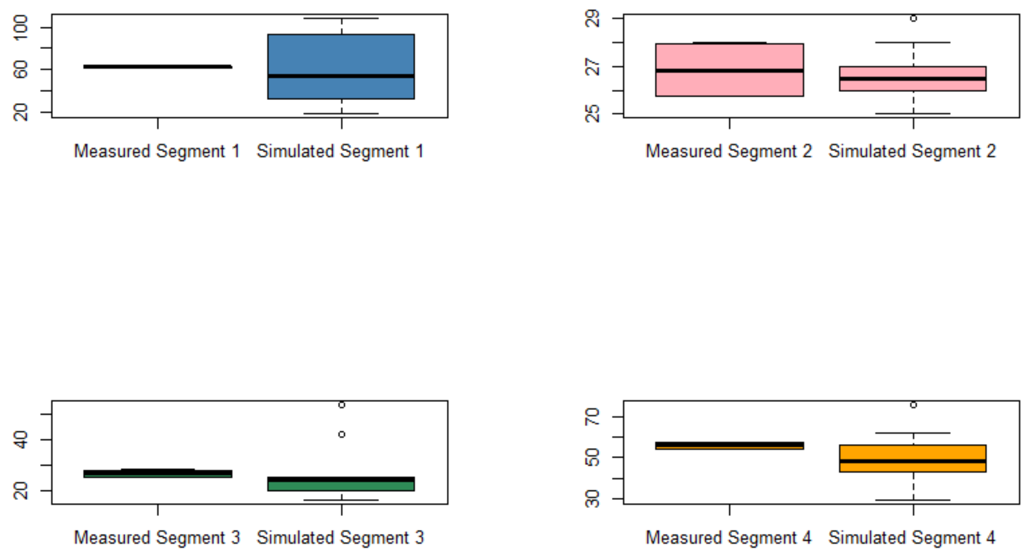

Figure 40 Box Plots of Measured and Simulated Travel Times for Bell Rd.

Table 38 Validation of Number of Lane Changes for Bell Rd.

	Segment 1	**Segment 2**	**Segment 3**	**Segment 4**
Mean	763.6	470.6	1071.2	506.8
Standard Error	5.031457	8.902185	17.3729	9.531002
Median	764.5	476	1067.5	502.5
Mode	#N/A	#N/A	#N/A	481
Standard Deviation	15.91086	28.15118	54.93794	30.13967
Sample Variance	253.1556	792.4889	3018.178	908.4
Count	10	10	10	10
Upped Level	774.9819	490.7381	1110.5	528.3606
Lower Level	752.2181	450.4619	1031.9	485.2394
Measured	NA	452	NA	NA
Valid?		Yes		

East Chase

Table 39 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for East Chase

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.94273	0.6853	No
Segment 2	0.86337	0.2725	No
Segment 3	0.94466	0.683	No
Segment 4	0.97137	0.85	No

Table 40 Wilcoxon Rank Sum Test Result Summary for East Chase

Measurement vs. Simulation	W	p-value	Valid?
Segment 1	10	0.2	Yes
Segment 2	20	1	Yes
Segment 3	4	0.06	Yes
Segment 4	20	0.9	Yes

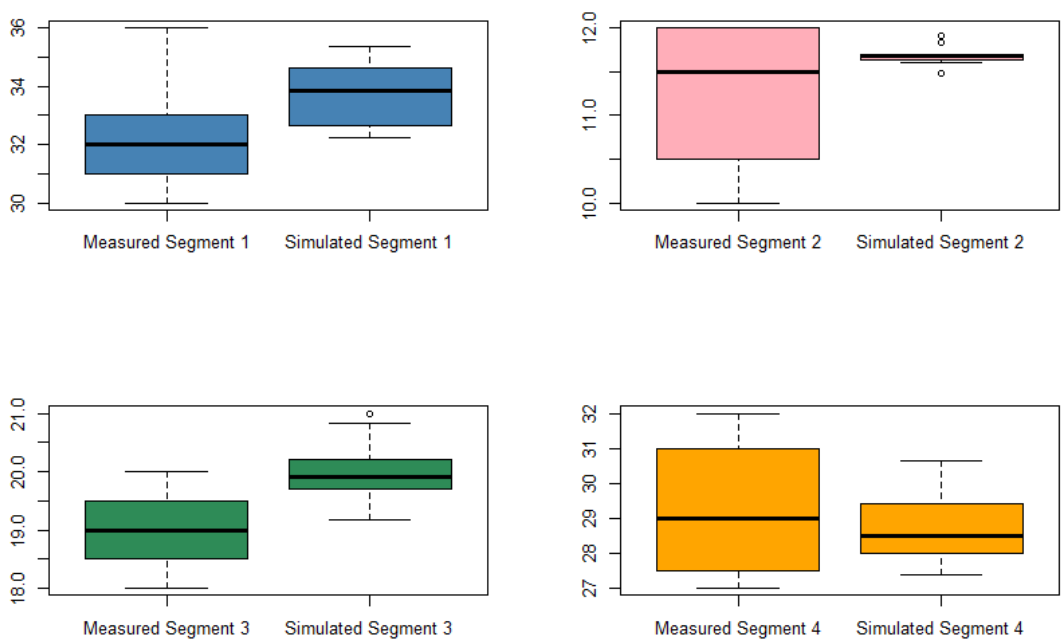

Figure 41 Box Plots of Measured and Simulated Travel Times for East Chase

Table 41 Validation of Number of Lane Changes for East Chase

	Segment 1	**Segment 2**	**Segment 3**	**Segment 4**
Mean	373.6	822.5	595.7	234.4
Standard Error	3.350622	5.447222	5.848172	2.868217
Median	375	822	597	234
Mode	362	833	572	234
Standard Deviation	10.5956	17.22563	18.49354	9.070097
Sample Variance	112.2667	296.7222	342.0111	82.26667
Count	10	10	10	10
Upper Level	381.1796	834.8225	608.9295	240.8884
Lower Level	366.0204	810.1775	582.4705	227.9116
Measured	NA	816	602	NA
Valid?		Yes	Yes	

Grand Prairie

Table 42 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for Grand Prairie

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.96429	0.6369	No
Segment 2	0.75	0.7725	No
Segment 3	0.945	0.683	No
Segment 4	0.82137	0.85	No

Table 43 Wilcoxon Rank Sum Test Result Summary for Grand Prairie

Measurement vs. Simulation	W	p-value	Valid?
Segment 1	10	0.6	Yes
Segment 2	20	0.9	Yes
Segment 3	20	0.1	Yes
Segment 4	20	0.3	Yes

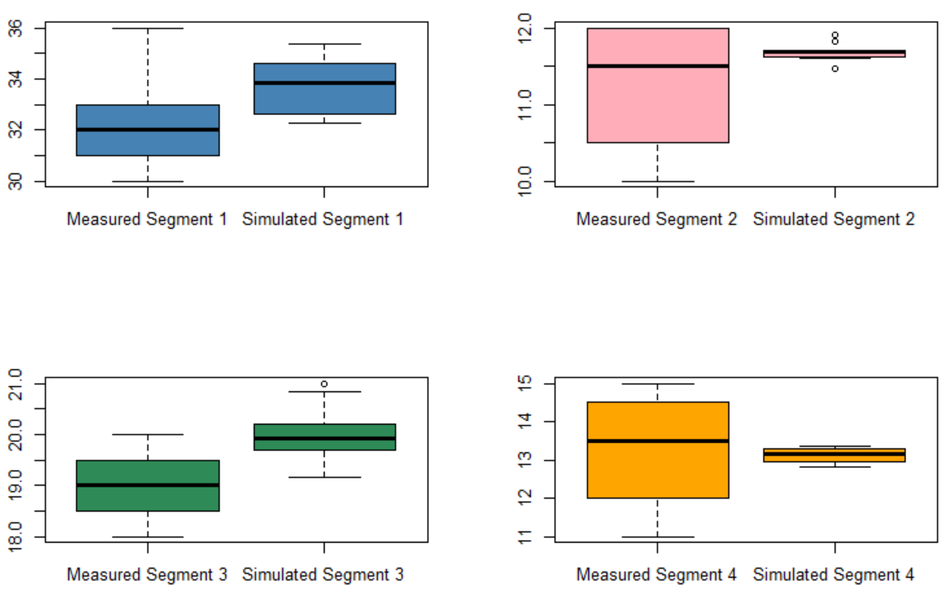

Figure 42 Box Plots of Measured and Simulated Travel Times for Grand Prairie

Table 44 Validation of Number of Lane Changes for Grand Prairie

	Segment 1	**Segment 2**	**Segment 3**	**Segment 4**
Mean	202.4	363.5	663.9	152.4
Standard Error	5.432822	3.448832	8.756521	1.446836
Median	202	363	670	151.5
Mode	#N/A	#N/A	#N/A	#N/A
Standard Deviation	17.18009	10.90617	27.69055	4.575296
Sample Variance	295.1556	118.9444	766.7667	20.93333
Count	10	10	10	10
Upper Level	214.6899	371.3018	683.7086	155.673
Lower Level	190.1101	355.6982	644.0914	149.127
Measured	193	NA	NA	156
Valid?	Yes			Yes

Conway

Table 45 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for Conway

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.99291	0.9719	No
Segment 2	0.72863	0.02386	Yes
Segment 3	0.77526	0.06475	No
Segment 4	0.86337	0.2725	No

Table 46 Wilcoxon Rank Sum Test Result Summary for Conway Segments 1, 2, 4

Measurement vs. Simulation	W	p-value	Valid?
Segment 1	30	0.1	Yes
Segment 2	20	1	Yes
Segment 4	20	0.8	Yes

Table 47 T-Tests for Conway Measured and Simulated Travel Times Segment 3

	Segment 3 M	Segment 3 S
Mean	26.5	20.209
Variance	25.66667	0.203761
Observations	4	10
Pooled Variance	6.569488	
Hypothesized Mean Difference	0	
df	12	
t Stat	4.148775	
P(T<=t) two-tail	0.000675	
t Critical two-tail	1.782288	

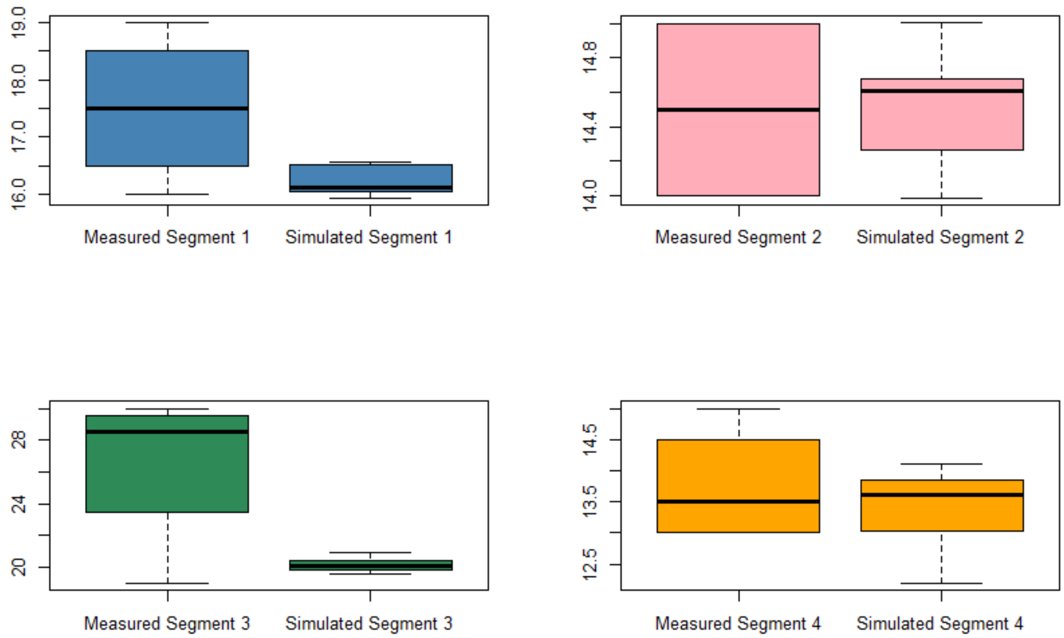

Figure 43 Box Plots of Measured and Simulated Travel Times for Conway

Table 48 Validation of Number of Lane Changes for Conway

	Segment1	**Segment2**	**Segment3**	**Segment4**
Mean	494.2	126.4	313.2	145.8
Standard Error	6.452906	4.752076	7.077036	1.271919
Median	494.5	128.5	308.5	146.5
Mode	#N/A	143	#N/A	141
Standard Deviation	20.40588	15.02738	22.37955	4.022161
Sample Variance	416.4	225.8222	500.8444	16.17778
Count	10	10	10	10
Upper Level	508.7975	137.1499	329.2094	148.6773
Lower Level	479.6025	115.6501	297.1906	142.9227
Measured	NA	NA	NA	146
Valid?				Yes

Rogers

Table 49 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for Rogers

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.91462	0.5072	No
Segment 2	0.85789	0.2528	No
Segment 3	0.97697	0.884	No
Segment 4	0.68923	0.008638	Yes

Table 50 Wilcoxon Rank Sum Test Result Summary for Rogers Segment 1, 2, 3

Measurement vs. Simulation	W	p-value	Valid?
Segment 1	22	0.8392	Yes
Segment 2	17	0.7333	Yes
Segment 3	19	0.9451	Yes

Table 51 T-Test for Rogers Measured and Simulated Travel Times Segment 4

	Segment 4 M	Segment 4 S
Mean	30.8575	30.654
Variance	0.582558	0.169603
Observations	4	10
Pooled Variance	0.272842	
Hypothesized Mean Difference	0	
df	12	
t Stat	0.658529	
P(T<=t) two-tail	0.522633	
t Critical two-tail	2.178813	

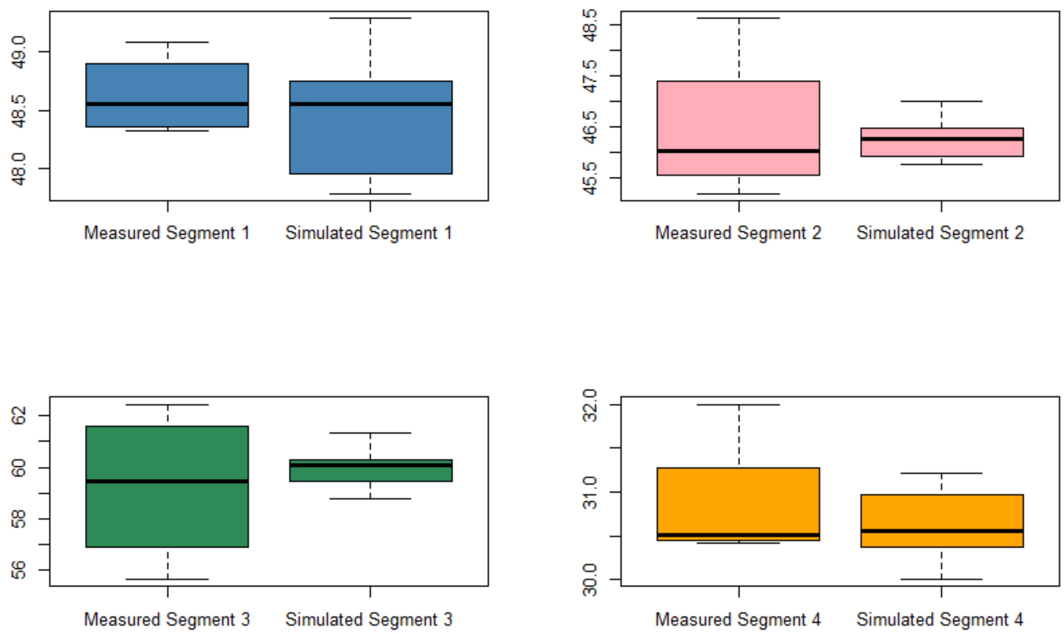

Figure 44 Box Plots of Measured and Simulated Travel Times for Rogers

Table 52 Validation of Number of Lane Changes for Rogers

	Segment 1 S	**Segment 2 S**	**Segment 3 S**	**Segment 4 S**
Mean	216.3	415.7	1175.3	335.9
Standard Error	2.40855	7.180297	10.26326	4.949635
Median	215.5	414.5	1177.5	331.5
Mode	212	441	1148	349
Standard Deviation	7.616503	22.70609	32.45527	15.65212
Sample Variance	58.01111	515.5667	1053.344	244.9889
Count	10	10	10	10
Upper Level	221.7485	431.943	1198.517	347.0969
Lowe Level	210.8515	399.457	1152.083	324.7031
Measured	214	NA	NA	336
Valid?	Yes			Yes

Forest Lake

Table 53 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for Forest Lake MN

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.48517	1.602e-06	Yes
Segment 2	0.82081	0.005217	Yes
Segment 3	0.79726	0.002505	Yes
Segment 4	0.36279	1.878e-07	Yes

Table 54 T-Tests for Forest Lake Measured and Simulated Travel Times Segments 1 and 2

	Segment 1 Measured	Segment 1 Simulated	Segment 2 Measured	Segment 2 Simulated
Mean	17.57388	22.35573	9.09797	18.00161
Variance	87.832	2.1349	14.031	13.125
Observations	33	40	33	40
Pooled Variance	40.75898		13.53386	
Hypothesized Mean Difference	0		0	
df	71		71	
t Stat	-3.185		-10.2916	
P(T<=t) two-tail	0.002151		1.01E-15	
t Critical two-tail	1.993943		1.993943	

Table 55 T-Tests for Forest Lake Measured and Simulated Travel Times Segments 3 and 4

	Segment 3 Measured	Segment 3 Simulated	Segment 4 Simulated	Segment 4 Simulated
Mean	12.04869	15.19094	19.41599	15.19094
Variance	67.069	3.1344	1.7700	3.1344
Observations	32	40	40	40
Pooled Variance	31.44835		2.452239	
Hypothesized Mean Difference	0		0	
df	70		78	
t Stat	-2.36255		12.06606	
P(T<=t) two-tail	0.020934		1.61E-19	
t Critical two-tail	1.994437		1.990847	

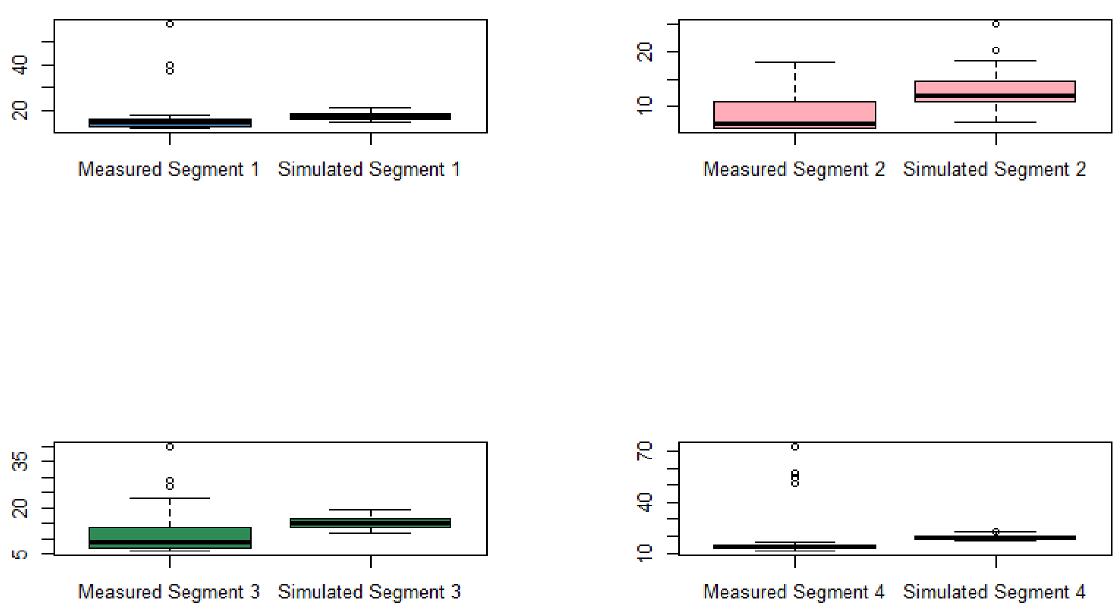

Figure 45 Box Plots of Measured and Simulated Travel Times for Forest Lake

Table 56 Validation of Number of Lane Changes for Forest Lake

	Segment 1	Segment 2	Segment 3	Segment 4
Min	745.4281	341.0175	333.4168668	672.4887
Max	873.5605	425.5601	434.8751775	776.999
Mean	801.2954	386.5381	391.9471159	724.8661
Standard Deviation	28.81036	27.80833	26.83035664	22.04556
Lower bound	772.4851	358.7298	365.1167592	702.8205
Upper bound	830.1058	414.3464	418.7774725	746.9116
Measured NLC	781	390	388	697
Valid?	Yes	Yes	Yes	Yes

Cottage Grove

Table 57 Shapiro-Wilk Normality Test Result Summary for Travel Time Measurements for Cottage Grove

Measurement	w-value	p-value	Normal Distribution?
Segment 1	0.94443	0.4068	No
Segment 2	0.9034	0.09112	No
Segment 3	0.89655	0.07078	No
Segment 4	0.91634	0.1473	No

Table 58 Wilcoxon Rank Sum Test Result Summary for Cottage Grove

Measurement vs. Simulation	W	p-value	Valid?
Segment 1	50	0.003487	
Segment 2	146	0.5095	Yes
Segment 3	108	0.4623	
Segment 4	152	0.3809	

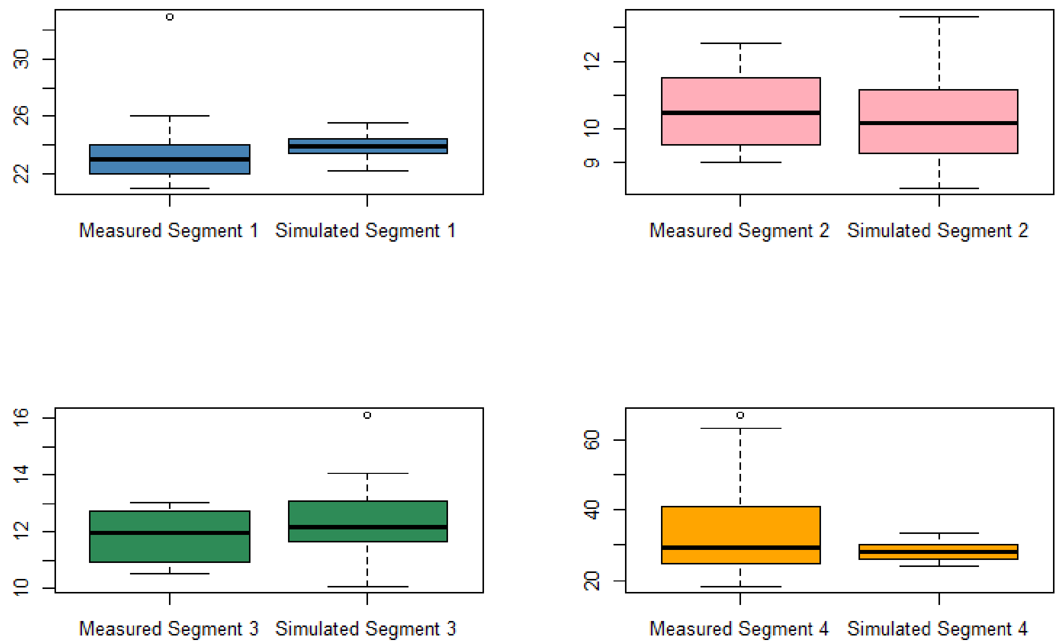

Figure 46 Box Plots of Measured and Simulated Travel Times for Cottage Grove

Table 59 Validation of Number of Lane Changes for Cottage Grove

	Segment 1	**Segment 2**	**Segment 3**	**Segment 4**
Mean	636.4	142.7	198.3	319.4
Standard Error	10.49571	2.748939	3.82114	7.398498
Median	648	144	201.5	320
Mode	#N/A	152	191	329
Standard Deviation	33.19036	8.692909	12.08351	23.39611
Sample Variance	1101.6	75.56667	146.0111	547.3778
Count	10	10	10	10
Upper Level	660.143	148.9185	206.944	336.1366
Lower Level	612.657	136.4815	189.656	302.6634
Measured	NA	140	198	NA
Valid?		Yes	Yes	

APPENDIX C

SCENARIO COMBINATIONS FOR ELIMINATED SCENARIOS

Volume =1: Off peak conditions

Volume =2: Peak conditions

Volume =3: 0.75* capacity

Volume =4: Capacity

Driveway=0: Base Conditions

Driveway=1: One driveway for each plaza

Driveway=2: No driveways

Median=0: Base conditions

Median =1: Raised median

Right Turn=0: Yield control

Right Turn=1: Stop control

Site	Segment	Volume	Driveway	Median	Right Turn
Forest Lake	1	4	1	1	0
Forest Lake	1	4	2	1	0
Forest Lake	2	3	1	1	0
Forest Lake	2	3	1	1	1
Forest Lake	2	3	2	1	0
Forest Lake	2	3	2	1	1
Forest Lake	2	4	1	1	1
Forest Lake	2	4	2	1	1
27th St.	1	3	0	0	0
27th St.	1	3	0	1	0
27th St.	1	3	1	0	0
27th St.	1	3	1	1	0
27th St.	1	3	2	0	0
27th St.	1	3	2	1	0
27th St.	1	4	0	0	0
27th St.	1	4	0	1	0
27th St.	1	4	1	0	0
27th St.	1	4	1	1	0
27th St.	1	4	2	0	0
27th St.	1	4	2	1	0
East Chase	1	3	2	1	0
East Chase	1	4	2	1	0
Grand Prairie	4	1	1	1	0
Grand Prairie	4	1	1	1	1
Grand Prairie	4	2	1	1	0
Grand Prairie	4	2	1	1	1
Grand Prairie	4	2	2	1	0
Grand Prairie	4	2	2	1	1
Grand Prairie	4	3	1	1	0
Grand Prairie	4	3	1	1	1
Grand Prairie	4	3	2	1	0
Grand Prairie	4	3	2	1	1

Site	Segment	Volume	Driveway	Median	Right Turn
Grand Prairie	4	4	1	1	0
Grand Prairie	4	4	1	1	1
Grand Prairie	4	4	2	1	0
Grand Prairie	4	4	2	1	1
Rogers	1	1	1	0	0
Rogers	1	1	1	1	0
Rogers	1	1	2	0	0
Rogers	1	1	2	1	0
Rogers	1	2	1	0	0
Rogers	1	2	1	1	0
Rogers	1	2	2	0	0
Rogers	1	2	2	1	0
Rogers	1	3	1	0	0
Rogers	1	3	1	1	0
Rogers	1	3	2	0	0
Rogers	1	3	2	1	0
Rogers	1	4	1	0	0
Rogers	1	4	1	1	0
Rogers	1	4	2	0	0
Rogers	1	4	2	1	0
7th St.	1	4	0	0	0
7th St.	1	4	0	1	0
7th St.	1	4	1	0	0
7th St.	1	4	1	1	0
7th St.	1	4	2	0	0
7th St.	1	4	2	1	0