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Evaluation and modeling of the effect of midblock pedestrian crossings on arterial traffic

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EVALUATION AND MODELING OF THE EFFECT OF MIDBLOCK PEDESTRIAN CROSSINGS ON ARTERIAL TRAFFIC

by

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ABSTRACT

Evaluation and Modeling of the Effect of Midblock Pedestrian Crossings on Arterial Traffic

by

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Unsignalized midblock pedestrian crossings are one of the critical components of arterial streets designed to provide for safer crossing locations for pedestrians. These crossings are often placed between signalized intersections that are far apart. However when arterial segments with such crossings are being analyzed or designed, the effect of the crossings was not normally taken into consideration. This is primarily due to the inability of current traffic software in modeling these locations.

The objective of this study was to evaluate the effect of midblock pedestrian crossings on the measures of effectiveness of arterial traffic flow using a computer simulation model. The research proposed modeling the midblock pedestrian crossing as an actuated uncoordinated signalized intersection. The study also focused on developing a methodology that can be used for designing optimal signal coordination on arterial segments with unsignalized midblock pedestrian crossings.

The case study location was a segment of Maryland Parkway corridor between Flamingo Road and Tropicana Avenue, Las Vegas, near the UNLV Campus. This segment has existing midblock crossings at Del Mar Avenue and East University Avenue.

As expected, there was significant impact on arterial traffic travel times, delays, queue length, number of stops due to midblock pedestrian crossings. In order to improve the arterial performance optimization of offsets that included the midblock pedestrian crossing was done. However, the resulting measures of effectiveness of arterial traffic were not improved relative to the existing offsets.

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CHAPTER 1

INTRODUCTION

Unsignalized pedestrian midblock crossings are one of the critical components of arterial streets designed to provide for safer crossing locations for pedestrians. These crossings are often placed between signalized intersections that are far apart. However when arterial segments with such crossings are being analyzed or designed, the effect of the crossing on arterial performance is not taken into consideration, due to the inability of current traffic software to model these locations. A midblock pedestrian crossing is a source of additional delay on a roadway segment. Therefore, there is need to model and evaluate the impact of crossings on arterial measures of effectiveness.

1.1 Study Background

Midblock pedestrian crossings are common on arterials where pedestrians from one side cross to other side for business activities, or to enter schools etc. When signalized intersections are far apart, mid-block crossings allow pedestrians to cross arterials more safely than by jaywalking. Also, mid-block crossings are common near bus stops and areas with high density residential and commercial areas. At least 20 midblock pedestrian crossing facilities exist in the Las Vegas metropolitan area (Merrill 2005). Mid-block pedestrian crossings with heavy pedestrian traffic can produce significant control delays and thus increased travel time for vehicles.

Few research publications addressed processes for calibrating the traffic micro simulation models. Published research regarding midblock pedestrian crossing operations has also been limited. Research regarding pedestrian safety statistics at midblock crossings had been addressed in detail, but studies pertaining to impact of midblock crossing on vehicular elements are few.

Therefore, it was determined that there was a need to understand the impact of traffic operations due to midblock pedestrian crossings. Specifically, this study was to construct a procedure to model a pseudo signal at mid-block pedestrian crossing and optimizing the network to get better signal coordination. Since midblock pedestrian crossings are almost on arterials with nearby signalized intersections, a microsimulation model methodology was considered to be particularly useful for planning and designing similar facilities.

1.2 Study Objective

The objective of this study was to evaluate the effect of pedestrian midblock crossings on an Arterial street using a computer simulation model. The research proposes modeling the midblock pedestrian crossing as an actuated uncoordinated signalized intersection herein refers to as a "pseudo signal". The parameters of the pseudo signal will be set in order to reproduce the observed characteristics and inputs of the pedestrian vehicle interactions at the crossing. Software used for modeling was VISSIM v4.2 for micro simulations and SYNCHRO v5.0 was used to optimize the offsets of the model.

This study developed a procedure to model a pseudo signal which reproduces the pedestrian activities at midblock pedestrian crossing. The study also focused on developing a methodology that can be used for designing optimal signal coordination on arterial segments with unsignalized midblock pedestrian crossings. The methodology derived can be taken as prototype design at any unsignalized midblock crossing on arterial segment. Travel times were used to calibrate and validate the model.

1.3 Study Expectation

It was expected that this model would provide a practical, yet effective methodology for developing, calibrating, and validating other microsimulation models involving mid-block pedestrian crossings. Therefore, the methodology for designing a pseudo signal became necessary. The methodology used was expected to be applicable to any unsignalized mid-block pedestrian crossing on arterial segment. Furthermore, the final optimized model with better signal coordination was expected to be capable of generating usable outcomes for arterial traffic operational analyses, as well as planning purposes, such as determining the effect of possible design alternatives which produces better measures of effectiveness on arterial traffic.

This Thesis contains six chapters. The first chapter provides an introduction to the subject to be addressed in this study and the objective of the study. The second chapter summarizes a literature review that was conducted on previous studies about the calibration, pedestrian interaction at midblock pedestrian crossing locations. Third chapter provides the data collection, methodology that was developed to model a pseudo signal, methodology for calibration and to develop the different simulation models for analysis. The fourth chapter presents the case study where in the model was developed, calibrated, and converted into different simulation models for analyses. In the fifth chapter, results were analyzed and discussed. The last chapter provides the conclusions and recommendations from this study.

CHAPTER 2

LITERATURE REVIEW

The relevant literature regarding impact of midblock pedestrian crossing on arterial traffic was found to be less. Minimal research had been published regarding microsimulation of pedestrian mid-block crossings. Most published research was related to the design of Midblock pedestrian crossing, the pedestrian behavior on such crossings or addressing the safety of pedestrians in crossing situations. Furthermore, some data were more easily obtained through previous research than by conducting studies specifically for this project (such as speed distributions for Maryland Parkway Arterial). Finally, it was necessary to establish an experimental study to model a pseudo signal which reproduces the Midblock pedestrian crossing and to study the impact of midblock pedestrian on arterial traffic.

2.1 Previous Mid-block Pedestrian Crossing Models

A few models have been created and published regarding mid-block pedestrian crossings. However, the work done by Kaseko and Karkee (2005) has been the only presented research that uses VISSIM software to model these types of crossings. In their simulation model, Kaseko and Karkee (2005) modeled an isolated mid-block pedestrian crossing. The case study site was a six-lane arterial with raised median, with an AADT of approximately 35,200 vehicles, and a pedestrian mid-block crossing with approximately 100 pedestrians per hour in each direction, modeled as an isolated location.

The objective of this model was to demonstrate the feasibility and concept of simulating and analyzing a microsimulation model with a mid-block pedestrian crossing. The model defined specific behavioral rules for both pedestrians and drivers, and attempted to show how certain input variables (such as the proportion of drivers yielding to pedestrians) affected an outcome (such as vehicular delays at the crosswalk). This research paved the way to develop a model in greater detail and scope by John Merrill as his thesis study (2005).

Kaseko and Karkee (2005) started by identifying certain behavioral characteristics at the case study location. Two types of drivers were defined: drivers who would stop for a pedestrian queued at the crosswalk (non-aggressive), and drivers who would not (aggressive). They furthermore defined crosswalk user behavior as:

- Every vehicle approaching the crosswalk yields to pedestrians already crossing
- Non-aggressive drivers always yield to pedestrians waiting to cross
- Aggressive drivers only yield to pedestrians already crossing
- The proportion of these two types of drivers depends on factors such as the visibility of the crosswalk, knowledge of pedestrian laws, and willful non-compliance
- All pedestrians yield to vehicles that cannot stop safely
- Pedestrians are able to tell if a vehicle plans on yielding

The authors outlined several key variables which were necessary to set in the model. Among these were minimum gap times and headways at the crosswalk. Most of these were defined using the respective conventional theories and dimensions applicable to the case study site.

One major finding in Kaseko and Karkee (2005) was how the proportion of yielding drivers affected various delay measurements. The proportion had a more significant impact on pedestrian and vehicular delays when approximately 25% yielding or below. Above 25%, the relationship was not as pronounced, and was approximately linear. The proportions also affected vehicular queue lengths similarly. Below 25% yielding, average vehicular queue lengths varied greatly. Average queue lengths at 1,350 vehicles per hour increased steadily from 0 to 25% yielding, reaching a maximum average queue length of 24 feet at or above 25% yielding.

Merrill's thesis (2005) determined the proportion of yielding drivers which could provide transportation and law enforcement officials with information necessary to analyze sections of roadway, especially since it is required by law in the State of Nevada to yield to pedestrians in a designated crosswalk. Law enforcement officials could use this information to justify enforcement programs. The proportion of yielding drivers could even be important in public policy decisions, by studying the effect on compliance with overall delays per person in an urban transportation network.

Merrill (2005) proposed a viable policy for the collection and use of this information is critical. However, this proportion can give a reasonable estimate of the willful non-compliance of drivers. The level of accuracy of the procedure provided by Merrill will vary based on each practical application. For example, the proportion of unlawful drivers can be estimated from the proportion of drivers not yielding when the condition to yield is present. This proportion can also be used to justify targeting law enforcement operations at particular pedestrian crossings. For this situation, a small sample size may be sufficient to determine whether targeted enforcement would be justified at the subject mid-block crossing. To determine the amount of delay, a much higher level of accuracy (and number of readings) would be needed. For the study discussed in this thesis, a sample size of 124 observations was chosen, which yielded an error in sample proportion of ± 0.087. Assessing safety concerns or evaluating the performance of traffic control devices may require fewer or more observations than for determining the amount of delay incurred. These considerations are beyond the scope of this thesis, but it would be essential to define policies for use of the proportion of yielding driver data collection procedure for other applications beyond operational analyses.

The research conducted by Merrill (2005) proposed numerous additional research topics. The methodology can be developed for constructing, calibrating, validating, and evaluating the other mid-block pedestrian crossing locations. The effect of pedestrian groups crossing a mid-block crosswalk on yielding behaviors may also be important for further evaluation. A helpful parameter to investigate further would be to measure the average distance between the crosswalk and where vehicles yield; in other words, the difference between the yield bar and actual yield locations. Regarding the proportion of yielding drivers methodology, this procedure of sampling and estimating yielding proportion could be evaluated at several pedestrian mid-block crossings. Another subject for exploration would be how signal offset and green splits affect the delays incurred by vehicles and pedestrians through the corridor.

2.2 Level of Service of Pedestrians

Baltes and Chu (2002) developed a methodology for computing level of service methodology for pedestrians. The objective of the paper was to develop a methodology to determine level of service (LOS) for a mid-block pedestrian crossing. The methodology used was that of participant survey. The crossing conditions were observed at a site for 3 minutes and the site was ranked on a scale of A to F with respect to the amount of difficulty observed. The authors developed two types of level of service predicting models – directional and combined. The directional model made use of the variables for a single side of the mid-block crossing when the variables for each direction of travel were significantly different. The combined model was for those crossings that had relatively uniform values for the variables. The authors found that the errors observed during the computation of levels of service from field measurements were acceptable.

Milazzo, et al. (1999) conducted a research to develop the basis for revised operational analysis procedures for transportation facilities with pedestrian users where flow is interrupted by traffic control devices. This study has the background information on pedestrian walking speeds at signalized crossings and on pedestrian non-compliance at these locations. Then authors provided the new and revised level of service tables for analyzing various types of interrupted-flow pedestrian facilities. The authors believed that pedestrian delay should be the main measure of effectiveness at signalized intersections rather than the space. The research team recommended that the HCM should include some background information that will be helpful for analysts timing signals and performing other operations. The authors of this paper recommended assumed crosswalk walking speeds of 1.2 m/s for most areas and 1.0 m/s for crosswalks serving large numbers of older pedestrians. The authors recommended the use of pedestrian delay as the main measure of effectiveness for unsignalized pedestrian crossing. The recommended threshold for LOS F at signalized crossings is 60 s or more of delay per pedestrian, whereas at unsignalized crossings it is 45 s or more of delay per pedestrian. Delay is defined as time waiting on one side of intersection plus time waiting in median.

Romer and Sathisan (1997) conducted a research focusing on the interrelationships of the three elements of the pedestrian traffic system at a signalized intersection-the walkway, the street corner, and the crosswalk. Key factors that affect each element were identified. The level of service and capacity analyses were performed for the street corner area and the crosswalk. A planning methodology called integrated systems methodology was developed to link the sidewalk, corner area, and crosswalk elements by level of service. This systems methodology could be used to develop a balanced LOS system based on a desired LOS, corresponding pedestrian flow rate, and effective walkway width. The developed integrated systems method provided a tool from which various tables or charts can be generated. Then the methodology was applied to analyze existing pedestrian elements at a signalized intersection, that is, the sidewalk, intersection corner and the crosswalk, with a systems approach that identifies key interrelationships of the individual elements. The practice for sizing crosswalks by Manual on Uniform Traffic Control Devices and sidewalk widths by AASHTO can be replaced by this support tool.

2.3 Simulation Model Calibration Methodology

Chu, et al. (2004) published a paper on a Calibration Procedure for Microscopic Traffic Simulation. The objective of the paper was to present a systematic, multi-stage procedure for calibration and validation of the PARAMICS simulation models. In the procedure, traditionally, model parameters are adjusted until reasonable correspondence between the model and field data is obtained. The study network is a congested corridor consisting of I-15, I-405 and SR-133 in the city of Irvine, California. Basic input data included network geometry, driver behavior, vehicle characteristics, traffic analysis zones, and traffic control and traffic detection systems. The data collected for model calibration included freeway traffic volume and travel time data, and arterial traffic volume data. The travel-time data probe-vehicle based travel time data for Northbound (NB) and Southbound (SB) freeway I-405 collected on October 17 and 18, 2001 by Caltrans was used. The availability of travel time data was another restricting factor for selecting a typical dataset for calibration process. The data of October 17 was chosen because it matched with weekday data closely. The calibration procedure involved calibrating the driver behavior models and route choice model. The number of simulation runs is obtained by running some runs first and determining thereby from the mean and variance of the results obtained. The two parameters used for calibrating driver behavior models are mean target headway and driver reaction time. The two parameters used in the route choice model are perturbation and familiarity. Fine-tuning was done using the trial and error method to reconstruct traffic variations and match congesting pattern of the study network. It was found that 31 runs could achieve meaningful performance measures. The simulated traffic counts and point-to-point travel time data were found to correspond well with the observed measurements.

Hourdakis, Michlopoulos and Kottommannil (2003) published a paper titled, a Practical Procedure for Calibrating Microscopic Traffic Simulation Models using the AIMSUN software developed by Transport Simulation Systems as a case study. The objective of this paper is to present a complete, systematic and general calibration methodology for obtaining accuracy needed in high performance situations. An issue to be addressed is to asses the effectiveness of calibration by means of goodness-of-fit tests. The paper presents such a procedure and was implemented and tested at several tests for assessing freeway ramp metering performance. A Case study was done on Traffic operation of Southern California freeways. The main categories of simulation parameters for this study fall under: global and local. Examples of global parameters are vehicle characteristics like length, width, speed, headway etc. and local parameters such as speed limits. The global parameters were calibrated first followed by local parameters. While calibrating global parameters, the focus should be to attain satisfactory values for three statistics namely RMSP, r and U.

$$RMSP = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{x_i - y_i}{y_i}\right)^2}$$

where :

n = number of observations

x_i = simulated measurement at time i

y_i = actual measurement at time i

$$r = \frac{1}{n-1}\sum_{i=1}^{n}\left(\frac{(x_i - \bar{x})(y_i - \bar{y})}{s_x s_y}\right)$$

where :

$\bar{x}$ = mean of simulated measurements

$\bar{y}$ = mean of actual measurements

s_x = standard deviation of simulated measurements

s_y = standard deviation of actual measurements

$$U = \frac{\sqrt{\frac{1}{n}\sum_{i=1}^{n}(y_i - x_i)^2}}{\sqrt{\frac{1}{n}\sum_{i=1}^{n}y_i^2} + \sqrt{\frac{1}{n}\sum_{i=1}^{n}x_i^2}}$$

$$U_m = \frac{n(\bar{y} - \bar{x})^2}{\sum_{i=1}^{n}(y_i - x_i)^2} \qquad U_s = \frac{n(s_y - s_x)^2}{\sum_{i=1}^{n}(y_i - x_i)^2}$$

$$U_c = \frac{2(1-r)ns_y s_x}{\sum_{i=1}^{n}(y_i - x_i)^2}$$

where :

r = correlation coefficient

For calibration, the objective was to increase the slope and r^2 of the simulated and the actual station volume. After finding a combination of non-incident parameters for calibration, they were calibrated against incident datasets. The effectiveness of calibration was evaluated by comparing the traffic flow, travel time and queue length measurements.

As a measure of effectiveness a chi-square test was performed to compare simulated and actual speed graphs. The paper deals with primarily freeway sections where volume and speed are the primary validation parameters. A t-test was used to compare the two sets of data for a close match. The null hypothesis could be that the mean is equal to that of actual measurements. The Root Mean Square Percent Error (RMSP) was calculated to measure error. The correlation factor (r) was used for goodness-of-fit measure. An estimated number of 300 iterations were performed. The automated calibration procedure required about 9 iterations resulting in reduction of calibration time.

Park and Schneeberger (2003) developed a similar methodology for calibrating and validating traffic microsimulation models. The case study for this methodology was an urban arterial with coordinated actuated signals using VISSIM software. The microscopic simulation models use independent parameters to describe operations and characteristics. These models have default values but some of them also allow the user to input a certain range of values. Some of these parameters, in spite of being difficult to measure on field, also prove to have a substantial impact on the model's performance. To enable performance of the model, the parameters have to be calibrated i.e. tuned to represent field measurements. This paper proposes a multi-step procedure for calibration and validation of a model using a case study. The procedure involved identification of calibration parameters for measure(s) of effectiveness (MOEs) like travel time, delay and speed. The calibration parameters used for calibration in VISSIM were emergency stopping distance, lane-change distance, desired speed, number of observed preceding vehicles, average standstill distance, additive part of desired safety distance, waiting time before diffusion, and minimum headway. The procedure followed in Park and Schneeberger (2003) was presented in nine distinct steps:

1) Determine of measures of effectiveness:

   Identify performance measure(s) like average travel times between two data collection points in the network, uncontrollable input parameters like corridor geometry, traffic counts and signal timings, and controllable parameters like minimum headway, minimum lane change distance and so forth.

2) data collection:

   Gather all uncontrollable parameters and measures of effectiveness data from previously collected data, direct observation, videotape, or other reliable sources

3) Identify calibration parameters:

   Select parameters to adjust that will have the greatest impact on the selected measure(s) of effectiveness, and determine an appropriate range of values for these parameters

4) Design experiment:

   Create table of simulation parameter sets as to minimize correlation between individual parameter values and maximize the coverage of all parameters

5) Run iterative simulations:

   Run several simulations for each parameter set using different random seeds to produce average and standard deviation values for the measure(s) of effectiveness of every set

6) Develop surface function:

   Determine an equation for the measure(s) of effectiveness using regression analysis of the iterative simulation parameter values

7) Determine desirable parameter sets from surface function:

   Use surface function to calculate parameter sets which best approximate the measure(s) of effectiveness collected in the field

8) Evaluate parameter sets:

   Run several iterations of each candidate parameter set to determine if model measure(s) of effectiveness are statistically significant to field measurement distributions, and evaluate the visualization to select the best candidate

9) Collect and validate model with new data set:

   Validate candidate parameter set by comparing outcomes to an untried measure of effectiveness

Park and Schneeberger (2003) noted several times that visualization is extremely important in the selection of candidate parameter sets. The authors reported that in the validation of their case study model, the field measure of effectiveness (maximum queue length on the eastbound segment) ranked around 90% of the simulated values. Although limited statistical analysis was conducted on the validation step, the field values were compared to the simulated values graphically.

2.4 State of Nevada Motor Vehicle and Pedestrian Laws

The State of Nevada requires by law that a vehicle yield to a pedestrian in a crosswalk, or to yield when another vehicle has already stopped at a crosswalk and verify that a pedestrian is not present on the pedestrian crossing before continuing to drive. According to the Nevada Revised Statutes (2004), Chapter 484, Section 325, Articles 1 and 3 pertain to marked mid-block pedestrian crossings:

1. When official traffic-control devices are not in place or not in operation the driver of a vehicle shall yield the right-of-way, slowing down or stopping if need be so to yield, to a pedestrian crossing the highway within a crosswalk when the pedestrian is upon the half of the highway upon which the vehicle is traveling, or when the pedestrian is approaching so closely from the opposite half of the highway as to be in danger.
2. Whenever a vehicle is stopped at a marked crosswalk or at an unmarked crosswalk at an intersection, the driver of any other vehicle approaching from the rear shall not overtake and pass the stopped vehicle until the driver has determined that the vehicle being overtaken was not stopped for the purpose of permitting a pedestrian to cross the highway.

Both of these laws are enforced on a regular basis. Neither law makes an exception for a driver not perceiving the presence of a crosswalk or pedestrian. Having an understanding of these laws and how they are enforced is crucial to understanding and modeling vehicular/pedestrian yielding behavior. As there are likely several other reasons why a driver may fail to yield to a pedestrian, one reason could be that the driver was unaware of the respective laws or driver can be in a hurry and not ready to yield to a pedestrian. It is vital to understand that the drivers do not yield to a pedestrian may not necessarily be the same drivers that are disobeying these two laws.

2.5 Driver and Pedestrian Behavior

While many publications exist that address pedestrian safety (Cui and Nambisan, 2003; Pulugurtha and Nambisan, 2003), few studies have been conducted to address drivers' opinions toward pedestrians. Redmon (2003) discussed a series of four focus groups held by the Federal Highway Administration (FHWA); two were held in Los Angeles and two in Washington, DC. Each focus group consisted of either ten drivers or ten pedestrians, with each group having at least four females and four males. The goal of these group sessions was to assess general attitudes and behaviors of pedestrians and drivers interacting on roadways, and to develop guidelines for improving these relationships.

The pedestrian focus group participants were asked if they knew whether the pedestrian has the right-of-way in their respective locations. Participants from Los Angeles stated that they knew they had the right-of-way, but many participants from other locations were not aware of the respective laws.

The driver focus group participants were asked a similar question as described above, and like the pedestrians, the Los Angeles drivers were aware of the law and other drivers were not. Drivers were also asked about how much the presence of law enforcement influences their behavior at pedestrian crossings. Many drivers indicated that the presence of law enforcement was a major factor in compliance.

One major discovery by the research team was that certain drivers felt respectful to pedestrians, while other drivers felt inconvenienced by the presence of pedestrians. In fact, one participant believed that "most [drivers] would rather drive over [pedestrians] to get them out of the way" (Redmon, 2003). This statement implied that a significant number of drivers felt aggressive toward lawful and unlawful pedestrian use of the roadway.

The distinction between these types of drivers and those who are more accepting of pedestrian use of the roadway should be considered when developing any operational model. The proportion of aggressive drivers is expected to influence much of the delay to users of the corridor.

2.6 Pedestrian Speeds and Start-up Times

Knoblauch, Pietrucha, and Nitzburg (1996) conducted a study to determine the effects of pedestrian ages on walking speeds. It was found that the age of the pedestrian was a significant factor in the walking speeds at crosswalks. The study area included a total of 16 crosswalks located in Richmond, Washington D.C., Baltimore and Buffalo.

Sixteen different crosswalks were selected for this study, which were located in Richmond, Virginia; Washington, D.C.; Baltimore, Maryland; and Buffalo, New York. Each intersection was controlled by a traffic signal, although some of the intersections did not have pedestrian crosswalk signals. Geometric conditions such as medians, number of lanes, street-widths etc. and environmental conditions like wind speed were noted for each locality. The pedestrian age was estimated by the observers, whose information previously was verified with actual ages. The study did not include children under 13, pedestrians with heavy objects and those that entered the crosswalk diagonally or ran while entering the crosswalk. The observers were also careful to exclude pedestrians crossing diagonally through intersections and pedestrians running while entering the crosswalk.

Knoblauch, Pietrucha, and Nitzburg (1996) concluded that the difference between the mean and 15th percentile values for crossing speeds of younger and older pedestrians was statistically significant. They furthermore stated that the impacts of many environmental and geometric conditions were also statistically significant.

The authors determined the data suggested that pedestrians who cross illegally tend to walk faster than those who cross legally. Data on mean and 15th percentile crossing speeds were analyzed for both legal and illegal crossing. Although the authors compiled information on these values for the entire set of pedestrians (7,123), the results from the law-abiding pedestrians (4,460) is of much greater interest.

Certain environmental and geometric conditions also affected the crossing speeds. Some of these conditions were only significant when combined with age. The results showed that the pedestrians who crossed illegally had higher crossing speeds than those who walked legally. Certain geometric and environmental conditions were also found to affect crossing speed but not crossing start-up times. The factors like wind, temperature, number of lanes, lane width gender were found have significant impact on crossing speeds

The authors noted that of the pedestrians who crossed legally, those crossing alone walked faster than pedestrians walking in a group. In addition, the authors stated that there was no difference in crossing speeds regardless of the presence of a pedestrian crossing signal.

Pedestrian start-up times were only addressed at intersections where a pedestrian signal was located. It was defined as the time it takes between the signal changing to "Walk" and the pedestrian completing a step off of the curb. Knoblauch, Pietrucha, and Nitzburg (1996) noted that environmental and geometric considerations were not statistically significant to the start-up speeds for either age group. However, the day of the week was not a significant factor between the different age groups. Furthermore, the authors concluded that single pedestrians and the group of pedestrians had identical average start-up times, and that this was not a significant factor in differentiating start-up times.

It should be noted that for an operational analysis, the mean values and ranges would be of most use. The information provided in Knoblauch, Pietrucha, and Nitzburg (1996) could be used to reproduce a distribution of pedestrian speeds and start-up times that reasonably approximate a normal distribution. Table 2-2 is a compilation of individual and group pedestrian speeds by Knoblauch, Pietrucha, and Nitzburg (1996) at pedestrian crossings. Table 2-3 shows start-up times recommended by Knoblauch, Pietrucha, and Nitzburg (1996).

Table 2-2 Speed Measurements at Pedestrian Crossings for Individual Pedestrians Crossing Legally, from Knoblauch, Pietrucha, and Nitzburg (1996)

Measurement (feet per second)	Individual pedestrians		Pedestrian groups	
	Mean	15^{th} percentile	Mean	15^{th} percentile
Younger pedestrians	5.04	4.19	4.66	3.86
Older pedestrians	4.15	3.23	4.00	3.12

Table 2-3 Start-up Time Measurements for Pedestrians at Crossings, as Compiled from Knoblauch, Pietrucha, and Nitzburg (1996)

Measurement (seconds)	Mean	85^{th} percentile
Younger pedestrians	1.93	3.06
Older pedestrians	2.48	3.76

The model by Kaseko and Karkee (2005) presented useful concepts toward modeling vehicle and pedestrian interactions. Merrill's thesis (2005) based on Kaseko and Karkee (2005) presented the calibrating and validating methodologies for practical model between two intersections and also level of service analyses for pedestrians at midblock pedestrian crossing was performed. Both Hourdakis, Michlopoulos and Kottommannil (2003) and Park and Schneeberger (2003) discussed the steps to calibrate and validate the simulation models. This research model was calibrated based on the methodology discussed by Park and Schneeberger (2003). Since this network has nine intersections and two directions, generating candidate parameter sets would take more time. So, combinations of calibration parameters were done and error was minimized when field travel times compared with calibrated travel time.

CHAPTER 3

METHODOLOGY

3.1 Methodology Overview

The methodology that has been developed for this study is a synthesis of techniques. The Following are the steps involved in Methodology:

- Location Identification
- Data Collection
- Model Building
- Model Calibration
- Simulation Analysis of different cases

Travel time data collection methodology was similar to the methodology recommended by the Manual of Traffic Engineering Studies (Institute of Transportation Engineers [ITE] 1994). Other data collection techniques, such as signal data, transit data, speed distribution, traffic volumes utilized the accepted practice methods from professional experience and knowledge of the levels of accuracy necessary. The calibration methodology developed was similar to those outlined in Park and Schneeberger (2003). Analyses and discussions of different simulation cases considered are shown in Figure 3-1, and potential implications will be addressed in the Case Study Results (Chapter 4), and Analysis and Discussion (Chapter 5) and Conclusions (Chapter 6).

The different cases considered for simulation runs are as follows:

- Case 1 - Model with midblock pedestrian crossings and using existing offsets and splits.
- Case 2 - Model without midblock pedestrian crossings and using existing offsets and splits.
- Case 3 – Model with midblock pedestrian Crossings, but with offsets and splits were optimized only for signalized intersections, ignoring pedestrian crossings.
- Case 4 - Model without midblock pedestrian crossings and offsets and splits were in Case 3.
- Case 5 - Model with midblock pedestrian crossings. Offsets and splits were optimized were optimized for the signalized intersections as well as the midblock pedestrian crossings. The midblock crossings were modeled as pseudo actuated uncoordinated signals.
- Case 6 – Model with midblock pedestrian Crossings. Offsets and splits with considering pedestrians and modeling midblock pedestrian crossings as actual actuated coordinated signals.

The results obtained with these cases were then compared and discussed as follows:

- Case 1 was compared with Case 2 to evaluate the effect of midblock pedestrian crossings on arterial traffic with existing offsets.

- Case 3 was compared with Case 4 to evaluate the effect of midblock pedestrian crossings on arterial traffic with optimized offsets based only on the signalized intersections.
- Case 3 was compared with Case 5 and Case 6 to see whether the optimal design of offsets has improved or not.

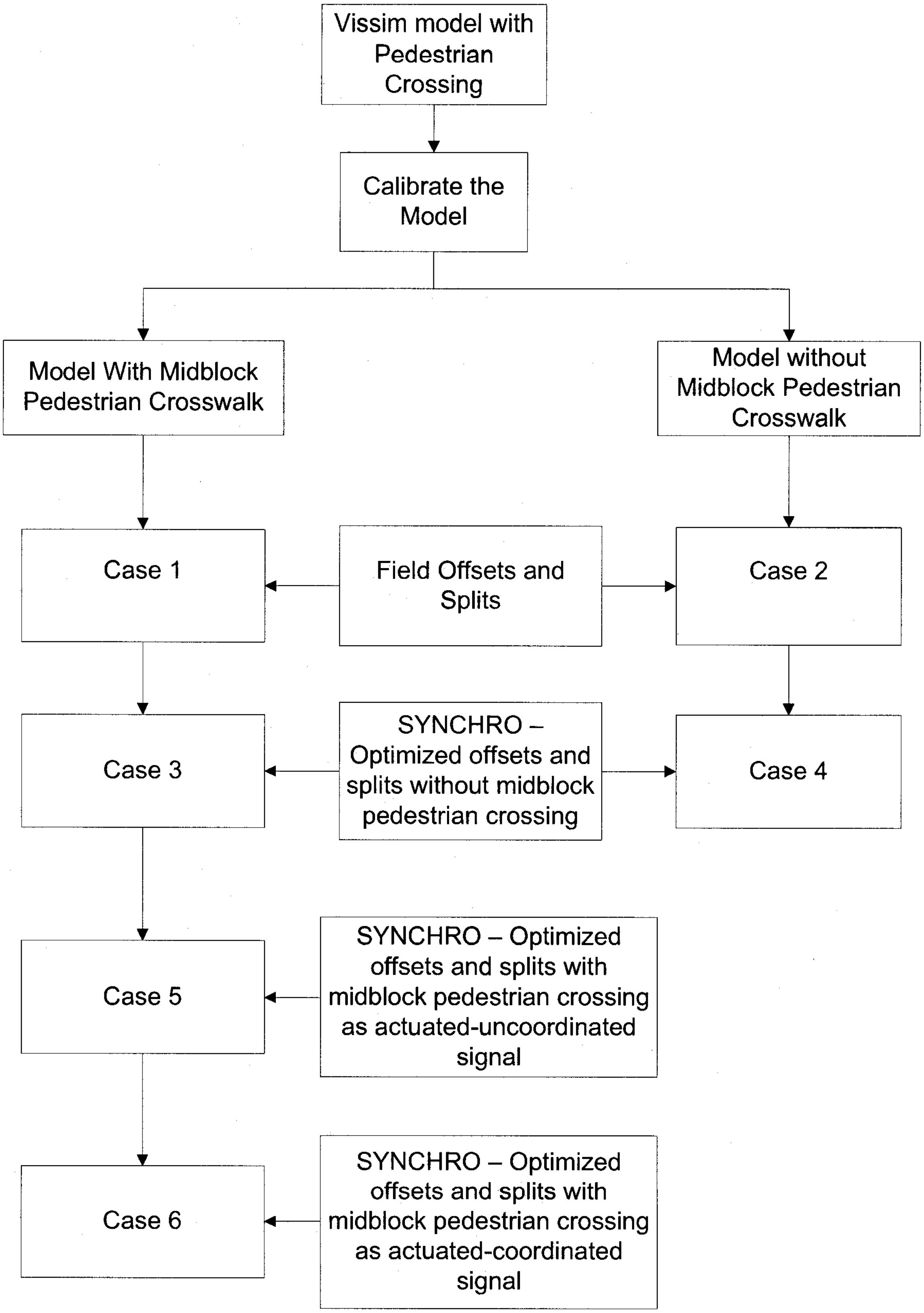
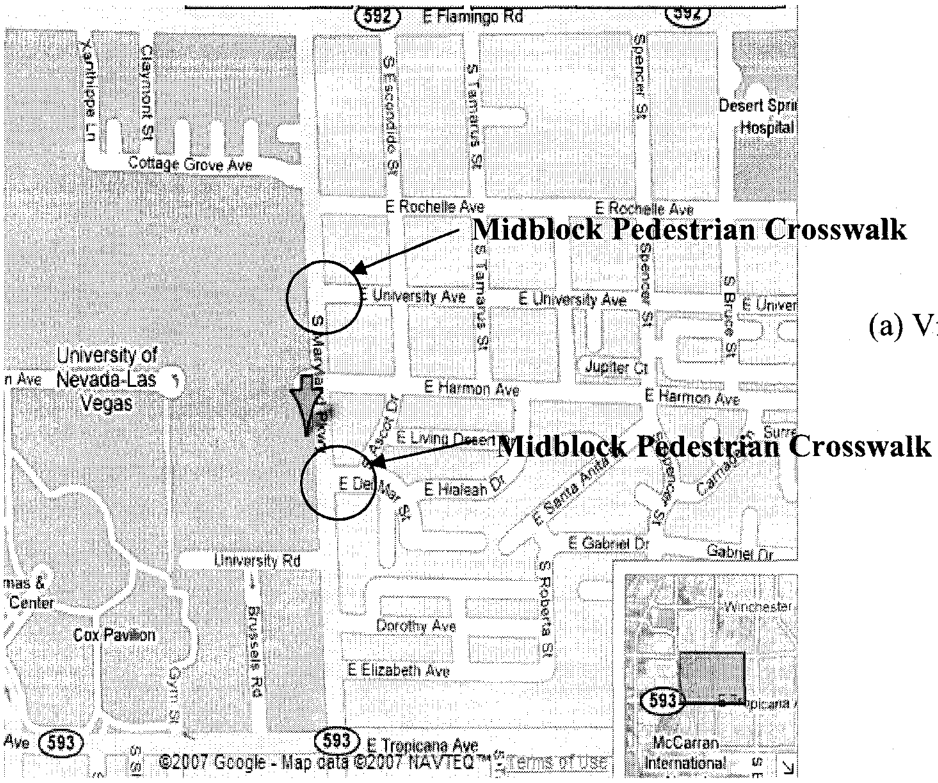


Figure 3-1 Flow Chart of Simulation Analysis Methodology

3.2 Case Study Location

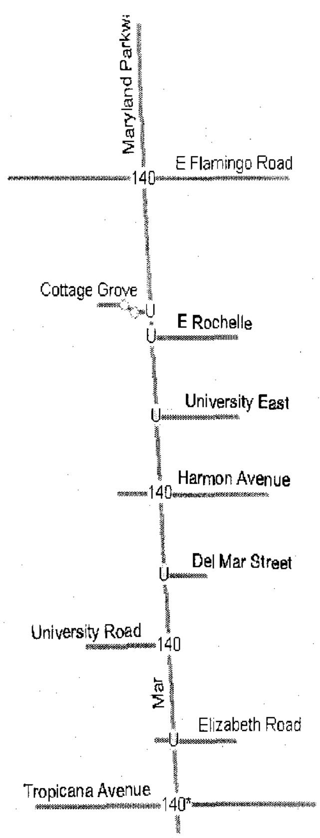
A segment of Maryland Parkway corridor between Flamingo Road and Tropicana Avenue, Las Vegas, (near the UNLV Campus) was used as the study area. This segment has existing midblock crossings at East Del Mar St. and East University Avenue as shown in Figure 3-2. The existing midblock pedestrian crossings are "Danish offset" type crossings. The roadway segment has a posted speed limit of 30 miles per hour. According to the Nevada Department of Transportation (2006), the average annual daily traffic count on Maryland Parkway near the study location was 40,343 vehicles. This portion of Maryland Parkway is also currently being considered for major redevelopment as a part of UNLV Mid Town Project. Figure 3-2 shows a map of the area for orientation purposes, with the project location specified.

All observations for this study were conducted during the midday period, 11:00 AM to 1:00 PM. This was because significant pedestrian activity happens around this time due to staff and students. This period reflects the daily peak for pedestrian traffic at these crosswalks. The midday period typically extends from 11:00 AM through 2:00 PM, and so it was determined that observations during this time should adequately represent midday traffic behaviors.



North

(a) Vicinity Map



(b) SYNCHRO Model Representation

Figure 3-2 Vicinity map of the project location on Maryland Parkway

3.3 Data Collection

Data collection was necessary along the Maryland Parkway study corridor. Standard License Plate study was performed as published in the Manual of Traffic Engineering Studies (ITE 1994). Information pertaining to the general geometry of the corridor was needed to develop a representative model. Studies such as pedestrian crossing time, pedestrian volume, number of pedestrian occurrences studies provided required pedestrian information within the study area. The pedestrian count, pedestrian crossing occurrences and the pedestrian crossing time were studied as one such field study. Sample data collection sheets are included in Appendix I for reference. Detailed results of these studies can be found in the Case Study Results (Chapter 4).

Measures of effectiveness selection

The calibration measures of effectiveness used were the northbound and southbound travel times from downstream to downstream of intersections. Travel times for each segment in the corridor were used as the calibration measures of effectiveness. Park and Schneeberger (2003) used the travel time over the entire network for one direction of traffic for calibration of the case study model. Travel times for calibrating the model were determined through a standard license plate survey, as published in the Manual of Transportation Engineering Studies (ITE 1994) which was discussed in Chapter 4.

Pedestrian Data

Pedestrian counts and the crossing time of pedestrians, and number of pedestrian occurrences were surveyed at two mid-block pedestrian crosswalks. Pedestrian counts, the crossing time of pedestrians, and number of pedestrian occurrences were surveyed at two mid-block pedestrian crosswalks at midday between 11:00 AM to 11:45 AM for eastbound pedestrians and 12:00 PM to 12:45 PM for westbound pedestrians

Geometry of Study Corridor

Information on the geometry of the study corridor was necessary for an accurate model. General information regarding the geometry of the corridor obtainable through aerial photography was sufficient. Determining the vertical grade was also important, since acceleration and deceleration of heavier vehicles can be affected by changes in elevation. Grades of less than 1% would likely not affect heavy vehicles over short distances. Since the study corridor is of shorter length, it was considered that the grades are less than 1% and it would not affect the heavy vehicles acceleration and deceleration factors.

Transit Data

The routes and timetables of public transit vehicles were collected from website of Regional Transportation Commission of Southern Nevada. Buses, especially those which board/alight in the rightmost lane might cause significant travel time delays to the corridor.

Signal Data

It was also necessary to determine signal timings for the four signalized intersections in the study corridor. Signal Log for the intersections were obtained from the Freeway and Arterial System of Transportation (FAST) traffic management center, which is operated by the Regional Transportation Commission (RTC) of Southern Nevada. It includes signal id, timestamp of events, and the description of the events. The site investigation was performed on Flamingo-Maryland signalized intersection to validate the collected data. The average nominal green split times was derived for the midday time period.

Table 3-1 Signal Phase Log Data Sample

SignalID	EventDate	EventTime	Event	Type
2133	18-Apr-06	11:00:21 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	3
2133	18-Apr-06	11:00:21 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	8
2133	18-Apr-06	11:00:45 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	4
2133	18-Apr-06	11:00:58 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	7
2133	18-Apr-06	11:01:21 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	1
2133	18-Apr-06	11:01:21 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	5
2133	18-Apr-06	11:01:40 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	2
2133	18-Apr-06	11:01:42 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	6
2133	18-Apr-06	11:02:41 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	3
2133	18-Apr-06	11:02:41 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	8
2133	18-Apr-06	11:03:05 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	4
2133	18-Apr-06	11:03:42 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	1
2133	18-Apr-06	11:03:42 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	5
2133	18-Apr-06	11:04:00 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	2
2133	18-Apr-06	11:04:02 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	6
2133	18-Apr-06	11:05:01 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	3
2133	18-Apr-06	11:05:01 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	8
2133	18-Apr-06	11:05:25 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	4
2133	18-Apr-06	11:05:38 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	7
2133	18-Apr-06	11:06:02 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	1
2133	18-Apr-06	11:06:02 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	5
2133	18-Apr-06	11:06:20 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	2
2133	18-Apr-06	11:06:23 AM	PHASE_VEHICLE_DISPLAY_CHANGED_TO_GREEN	6

From Table 3-1, it can be found that the signal phase log only records the time of events that phases change to green, not the events that change to yellow or red. Therefore, the duration of green phase cannot be derived directly from the data. It can be derived only by looking at the time that conflicting phases change to green. Since all our intersections are four legged and three legged, it is easy to determine the conflicting phases. Excel was used to automate the calculations and find the green splits. Specifically, the signal phase duration is derived by first searching for the event "Phase_Vehicle_Display_Changed_To_Green" for each signal/intersection and ordering them by the time they occurred. For each signal phase changed to green, the next conflicting phase is looked for in the event list. The time elapse between these two events is the phase duration. It is noted that the phase duration derived in this way also includes the time for yellow and all-red because the signal phase log data don't contain any yellow phase information. However, this problem wouldn't affect the derivation of signal cycle length and offset which are the required data for this study. In this study, signal cycle length is the time that elapses between two successive coordinated phases. In theory, any phase can be used to derive the cycle length. Since the coordinated phase is cycling and its ending time is fixed, it is chosen to calculate the offset in this study.

Desired Speed Distribution

Desired speeds, or free flow speed distribution was collected from Merrill (2005). A standard law-enforcement radar gun was used in his sample speeds collection. Speed distribution chart (shown in Appendix II) is from University road to Harmon segment in Maryland Parkway. However, the study area along Maryland Parkway also has the same posting speed limit and other surrounding factors. Hence, the same speed distribution is accounted for this study. Since the desired speed was necessary to input into VISSIM as a speed distribution, uninhibited vehicle speeds (as opposed to actual speed of vehicles during the study period) were necessary.

Traffic Volume and Turning Movements

Intersection volumes, Lane geometry were collected from Kimley-Horn Associates for the AM and PM peak hours. Since this study is on midday period, with the help of traffic counts from Nevada Department of Transportation the intersection turning movements were adjusted with the factors. The factors were derived from the Figure 3-3 showing the hourly volume from the Nevada Department of Transportation website. The hourly volume curve has peaks at Mid Day and PM rather than AM. The proportion difference between PM Peak and Mid Day time were derived as a factor and Mid Day Volumes were derived. Table 3-2 is the intersection volumes and lane geometry of the study area. The number of turning movements along the study corridor was an important factor to collect because it would affect the integrity of the volume counts throughout the model, as well as being important in calculating the Baltes and Chu (2003) "quality of service".

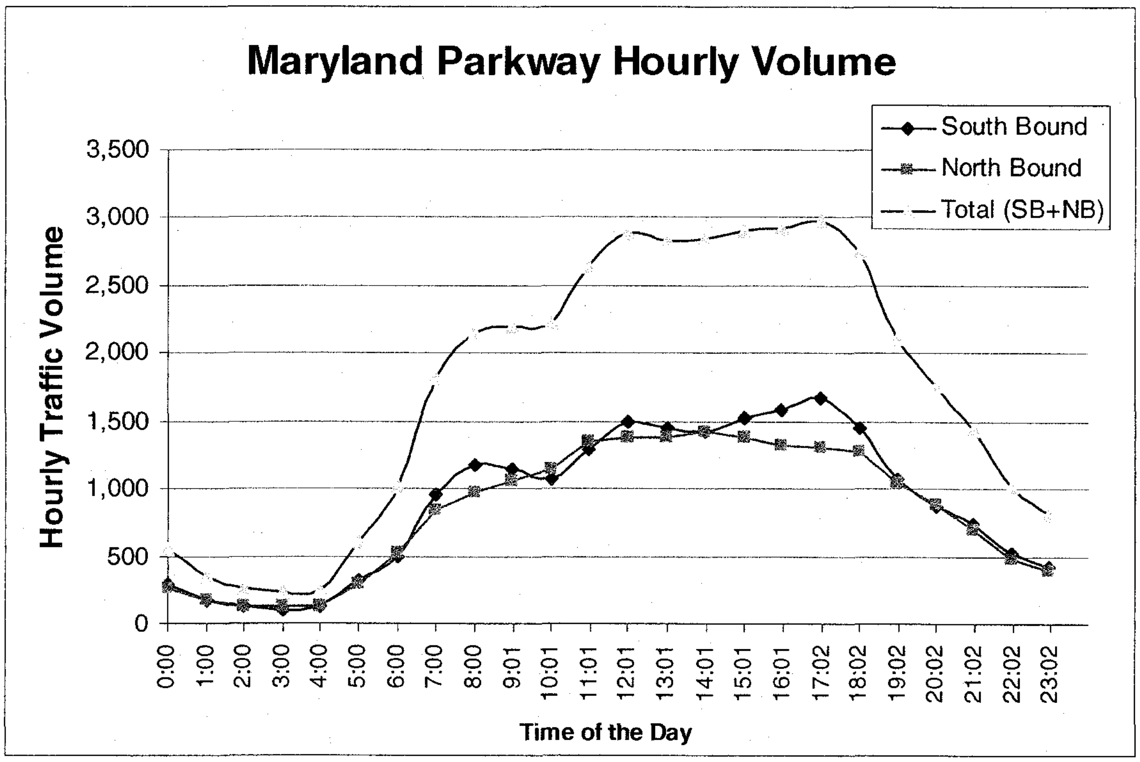


Figure 3-3 Hourly Volume Graph for Maryland Parkway from NDOT

Table 3-2 Maryland Parkway Intersections Volume

Intersection	Time	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Flamingo Road	AM	168	1156	303	197	1685	299	270	564	222	290	712	234
	MD	271	1471	291	199	1493	384	422	1066	170	333	760	168
	PM	337	1923	403	224	1597	391	338	971	103	408	1030	206
	Lanes	2	3	1	2	3	1	2	3	"s"	2	3	"s"
Cottage Grove	AM			143					1056			951	261
	MD			188					1658			1240	10
	PM			229					1412			1626	31
	Lanes			1				1	3			3	"s"
Rochelle Avenue	AM						15	302	1041	20		1094	
	MD						167	126	1491	38		1428	
	PM						154	111	1258	20		1855	
	Lanes						1		3	"s"		3	
University East	AM				9		68		1295	55	94	1000	
	MD				159		272		1383	32	53	1375	
	PM				63		64		1325	51	40	1815	
	Lanes				1		1		3	"s"	1	3	
Harmon Avenue	AM	54	23	56	204	66	223	117	1073	95	67	830	112
	MD	58	32	45	226	25	168	97	1189	138	179	1268	87
	PM	66	41	62	254	27	171	93	1139	132	219	1552	107
	Lanes	1	1	1	1	1	1	1	3	"s"	1	3	"s"
Delmar Street	AM				39		38		1247	146	50	1040	
	MD				29		69		1355	12	64	1475	
	PM				36		66		1298	27	63	1805	
	Lanes				1		1		3	"s"	1	3	
University Road	AM	41		25				203	1352			898	181
	MD	92		91				166	1275			1349	155
	PM	104		103				159	1221			1651	190
	Lanes	1		1				1	3			3	"s"
Elizabeth	AM	21		18	11		42	16	1492	14	15	881	27
	MD	2		87	48		120	28	1319	11	9	1364	67
	PM	12		60	39		24	27	1344	80	11	1736	7
	Lanes	1		1	1		1	1	3	"s"	1	3	"s"
Tropicana Avenue	AM	455	1105	72	94	1446	238	304	829	56	201	363	346
	MD	438	1440	156	180	1131	218	253	702	138	422	810	267
	PM	544	1823	197	194	1219	235	242	672	132	517	991	327
	Lanes	2	3	"s"	2	3	"s"	2	3	"s"	2	3	"s"

Key: AM – AM Peak 7:30 to 8:30 AM Traffic Volume
PM – PM Peak 4:30 to 5:30 PM Traffic Volume
MD – Mid Day 12:00 to 1:00 PM Traffic Volume
s – Shared lane

3.4 Model Building

The methodology developed for this project was divided into three stages: model development with and without pseudo signals at crosswalk locations using VISSIM, calibration, and optimization of offsets using SYNCHRO. Model development was perhaps the most crucial stage, since any omissions in the model design could severely impact the calibration and later stages. Care was taken to ensure that the level of accuracy required for each data collection effort would be large enough to allow for proper model function, but that the data collection process was resourceful and not overly cumbersome. Model calibration was done before modeling pseudo signals and the calibrated model (with pseudo signal) offsets were optimized and analyses was performed. The following subsections demonstrate the previously explained process by utilizing the case study. The process explained in the case study is much more in-depth than the brief description in the methodology.

3.4.1 Model in VISSIM v4.20

VISSIM v4.20, developed by Planung Transport Verkehr AG in Karlsruhe, Germany, had the necessary features to model intricate traffic operations on a microscopic level. Other micro simulation models such as CORSIM and SIMTRAFFIC do not have the option of signal changing parameters at an intersection by which pseudo signal was modeled.

The aerial photography obtained from Clark Country Department of Public Works was loaded and scaled as a background in the VISSIM model. All general roadway geometries (links), as well as turning movements (connectors), were then defined from the aerial photography. Though it is not 100% accurate, the aerial photographs were best suited to draft the connectors and links for most of the corridor. The bus transit routes, via South Maryland Parkway was input into VISSIM by defining two bus transit routes with average headways of 12 minutes. Stations were then input using approximate locations from the Clark County Ariel Map. The results were one transit line each direction with buses operating at headway published on the respective CAT route timetables (RTC 2004). Dwell time was entered as an approximate distribution required by VISSIM.

Signals were input into the VISSIM model by processed signal log data provided by FAST. NEMA Controller was used to give signal details in VISSIM. The average travel times through the corridor are the measurable outcomes for calibration. Turning movements at all the intersections were then input into the model as static route choices. Route guidance was specified for vehicles to follow the route to account for given turning movements.

VISSIM is capable of generating vehicles on any link. However, vehicles must be generated at the entry point into the model. To ensure that the correct number of vehicles pass through the intersection, the vehicles generated per hour would need adjustment at each source in order to balance the network. The network was balanced so that the entering vehicles in the link were equal to the exiting vehicles from the link. Exiting and entering driveways was ignored. It would be impractical and too time-consuming to input every driveway for each segment. The midblock pedestrian crossing was modeled as pseudo signal in this case study. The calibration process was done and validated with the field travel times on all segments other than two crosswalk locations. Then the crosswalk locations were modeled as pseudo signals in calibrated model.

3.4.2 Pseudo Signal Design

Pseudo signal is a modeled signal at midblock pedestrian crossing location which reproduces the activities happen at midblock pedestrian crossing. The study area consist two midblock pedestrian crossings at Maryland Parkway-Delmar street intersection and at Maryland Parkway-East University Avenue intersection. Instead of midblock pedestrian crossing, four legged intersection was created with an actuated signal controller.

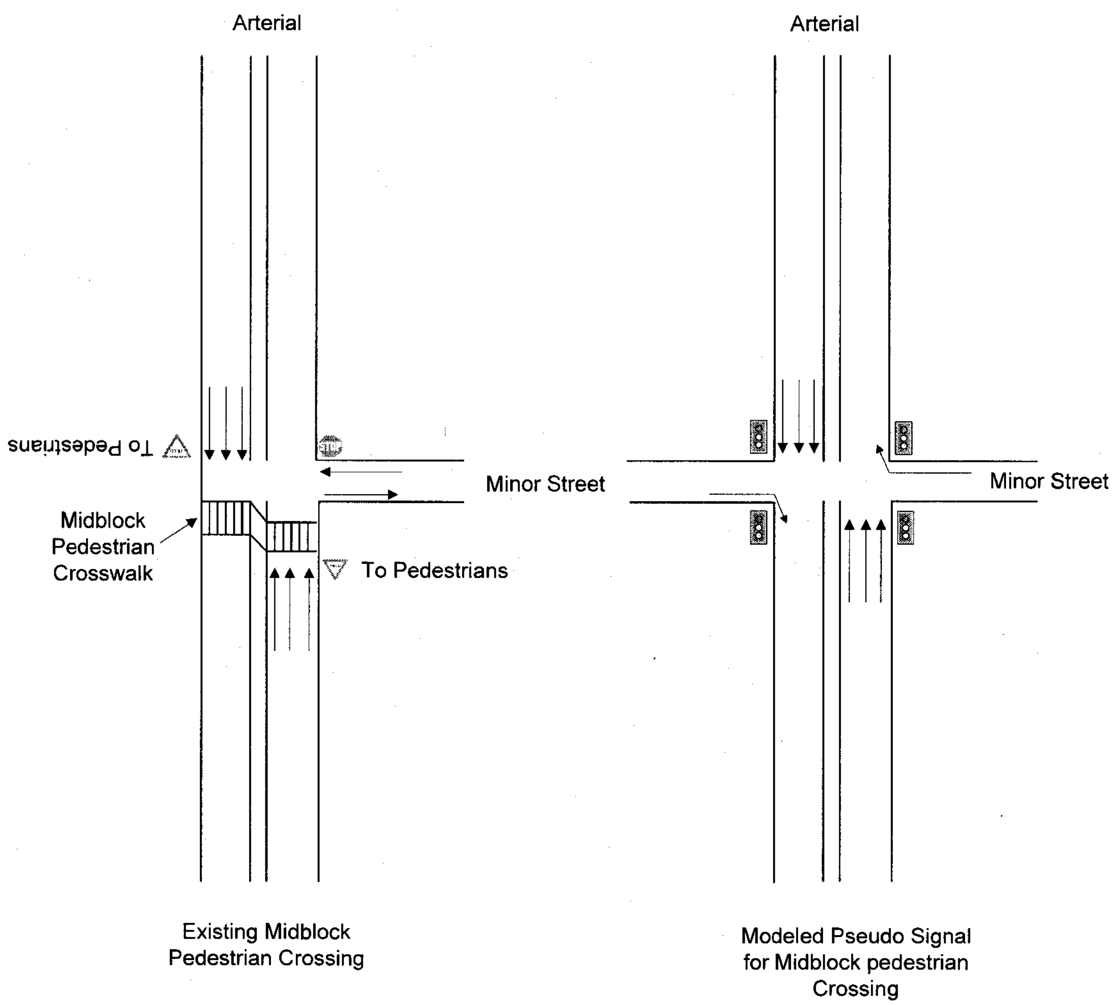


Figure 3-4 Modeled Pseudo Signal for Existing Midblock Pedestrian Crossing

The number of pedestrians crossing the road at the crosswalk location was converted into equivalent number of vehicles in respective direction. Eastbound pedestrians crossing the road at the crosswalk location were converted into equivalent eastbound vehicles and westbound pedestrians were converted into equivalent westbound vehicles. Pedestrian counts, the crossing time of pedestrians, and number of pedestrian occurrences were surveyed at two mid-block pedestrian crosswalks at midday between 11:00 AM to 11:45 AM for eastbound pedestrians and 12:00 PM to 12:45 PM for westbound pedestrians. One pedestrian occurrence was counted as when the vehicle stops for the pedestrian(s) to cross the road at midblock pedestrian crossing location. A pedestrian count obtained for 45 minutes was converted into pedestrians per hour. Number of pedestrian occurrences also converted into pedestrian occurrences per hour. Minimum, maximum and average pedestrian crossing times were computed from the surveyed data. To design a crosswalk as an actuated signal controller, maximum and minimum green splits, cycle length was required.

The number of occurrences of pedestrians is converted into number of equivalent actuations by vehicles, i.e. the pedestrian occurrence at the beginning of crosswalk was considered as the approaching vehicles on the detectors in respective direction. The cross direction vehicles (NB and SB) yielding to pedestrians on field were considered as conflicting direction vehicles (NB and SB) stopping for Eastbound Right (WBR) and Westbound Right (WBR) vehicles. Actuated signal was designed as two phase signal. First phase NB and SB direction and second phase as WBR and EBR direction. Total number of occurrences of pedestrians per hour is equal to the total number of actuations by vehicles per hour. The cycle length of a pseudo signal is equal to seconds per hour divided by number of occurrences in the field per hour. The signal timings were calculated as follows:

- The maximum green split on Minor Street is equal to the maximum crossing time of pedestrian.
- The minimum green split on Minor Street is equal to the average crossing time of pedestrian.
- Major Street (Arterial) green split is calculated as difference of cycle length and green split of Minor Street.

These values are entered into NEMA Controller in VISSIM software. However the number of pedestrians on the field was not equal to the number of vehicles in simulation. So, the equivalent number of vehicles in simulation was found by trial and error procedure. To start with, the number of pedestrians surveyed on the field was entered into the network and ran simulation. The above design procedure was shown as Flowchart in Figure 3-5.

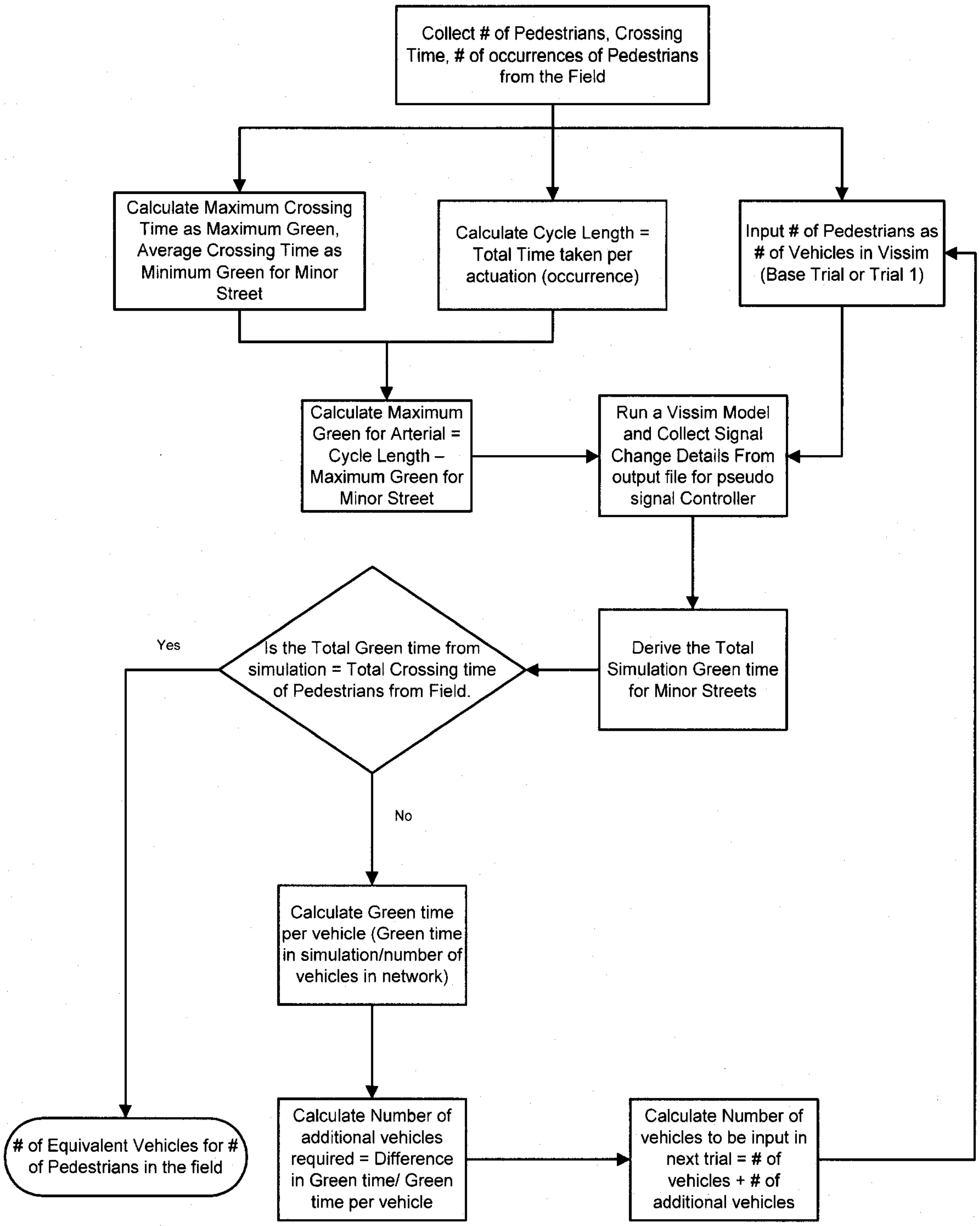


Figure 3-5 Flowchart of Pseudo Signal Design

VISSIM is flexible simulation software and have more options for evaluation of results. The simulation was performed and the Signal change output file was generated using the option in evaluation type of VISSIM output file. The typical output file was shown in the Table 3-3. The signal change output file provided a chronological list of all signal group (phase) changes of all selected signal controllers. A signal controller was given in each signal and phase numbers were given for all phases in that particular signal controller.

The output file (*.LSA) contains:

- File title
- Path and name of the input file (File)
- Simulation comment (Comment)
- Date and time of the evaluation (Date)
- List of all signal groups
- Data section containing one line for each signal change event of each signal group. The columns contain the following data (from left to right):

- Simulation time [s]
- Cycle time [s]
- SCJ (Signal Controller) no.
- Signal group no.
- New signal state
- Time since last signal change (= length of previous signal display)
- SCJ type

Table 3-3 Sample Output File for Signal Change

Simulation Cycle Time	SCJ Time	Signal Group	New Signal State	Time since last State	Controller Type	Signal Change
13	71	1	2	green	13	NEMA
13	138	3	6	green	13	NEMA
13	138	3	2	green	13	NEMA
13	118	4	6	green	13	NEMA
13	13	2	2	green	13	NEMA
19	19	5	5	amber	18	NEMA
19	19	5	1	amber	18	NEMA
21	21	2	6	amber	20	NEMA
22	22	5	5	red	3	NEMA
22	22	5	1	red	3	NEMA
23	23	6	5	amber	22	NEMA
23	23	6	1	amber	22	NEMA
23	23	5	6	green	23	NEMA
24	24	2	6	red	3	NEMA
25	25	2	5	green	25	NEMA
26	26	6	5	red	3	NEMA
26	26	6	1	red	3	NEMA
27	27	6	6	green	27	NEMA
30	88	1	6	amber	29	NEMA
33	91	1	6	red	3	NEMA

From the signal change output file, the total green time in simulation of the required signal group was derived. Then, this total green time was compared with the average total pedestrians crossing time in the field of that direction (i.e) the eastbound vehicles total green time was compared with average total Eastbound pedestrians crossing time. The average total eastbound pedestrians crossing time can be calculated as product of average crossing time of all pedestrians and the number of occurrences of pedestrians per hour.

After a few trials, both were nearly equal and the numbers of equivalent vehicles were found. Both the pedestrian crosswalks were designed as actuated signal controller and the travel time was recorded for evaluating purposes.

3.5 Model Calibration

3.5.1 Identification of calibration parameters

Once the model design was completed, variables within VISSIM had to be selected for calibration. The proposed calibration procedure was similar to part of the procedure used by Park and Schneeberger (2003) in their research. Since VISSIM has many global variables which can be changed from the default values, it was necessary to determine which variables had the greatest impact on the travel times being used for calibration.

To determine which of these variables had the greatest impact on travel times, all reasonably possible variables were identified and the default values recorded. Each parameter has its own acceptable range which was used for calibration process. Next, each parameter was assigned a feasible minimum and maximum value. Theses parameters include the desired speed distribution, lane-change distance, number of observed preceding vehicles, average standstill distance, additive part of desired safety distance, waiting time before diffusion, and minimum headway.

Desired Speed Distribution

The desired speed distribution is an important parameter with a significant influence on roadway capacity and achievable travel speeds. The desired speed is the speed a vehicle "desires" to travel at if it is not hindered by other vehicles. This is not necessarily the speed at which the vehicle travels in the simulation. If not hindered by another vehicle, a driver will travel at his or her desired speed (with a small stochastic variation called oscillation). The more vehicles differ in their desired speed, the more platoons are created. If overtaking is possible, any vehicle with a higher desired speed than its current travel speed is checking for the opportunity to pass - without endangering other vehicles. The default speed distribution is (20, 30) mph. The speed distribution used is (24.5, 47.9) mph. The desired speed distribution was the s-curve with minimum 24.5 mph, 15th percentile 27.9 mph, 85th percentile 37.8 mph and maximum 47.9 mph.

Lane Change Distance

The lane-change distance parameter is used along with the emergency stopping distance parameter to model drivers' behavior to stay on their routes. The lane-change distance defines the distance at which vehicles will begin to attempt to change lanes. The default value for lane-change distance is 656 feet. Acceptable values for lane-change distance were 492 ft to 984 ft. These values were selected to ensure a vehicle had a reasonable distance to make a lane change before it reached the intersection. Too small values would force vehicles into the emergency stopped condition.

Number of Observed Preceding Vehicles

The number of observed preceding vehicles affects how well vehicles in the network can predict other vehicles' movements and reacts accordingly. The VISSIM default value for this parameter is two vehicles. One to three vehicles were used in this study.

Average Standstill Distance

Average standstill distance defines the average desired distance between stopped cars and also between cars and stop lines, signal heads and so forth. The default value for average standstill distance is 6.56 ft. Acceptable ranges of values used for this parameter were 3.28 ft to 9.84 ft. Larger or smaller values appeared to be unreasonable.

Waiting Time before Diffusion

Waiting time before diffusion defines the maximum amount of time a vehicle can wait at the emergency stop position waiting for a gap to change lanes to stay on its route. When this time is reached, the vehicle is taken out of the network (diffusion). Sixty seconds is the default value. Other values used in the study were 20 and 40 seconds.

Minimum Headway

The minimum headway distance defines the minimum distance to the vehicle in front that must be available for a lane change in standstill condition. The default value is 1.64 ft. the acceptable range used in the case study was between 1.64 ft and 13.12 ft. The default value appeared to too small of a distance for vehicles to attempt a lane change. It did not appear realistic that a vehicle would attempt a lane change given headway of 1.64 ft. As a result, larger values were assumed to be more reasonable.

The simulation was run twenty times with all default variables entered into the model. Simulated travel times were recorded for all travel time section along the northbound and southbound of the Maryland Parkway arterial. Next, each parameter was individually assigned the feasible minimum and maximum values, while maintaining the default values for every other variable. The simulation was run, and respective travel times were recorded for each scenario. The same random seed number was used in the default and trial simulations. The travel times generated by the minimum and maximum values were then compared to the travel times generated by using all default values. Then the combinations of values of different calibration parameters were used to match the simulated travel time with the obtained field travel time. The closest simulated travel time and the actual travel time from the field were chosen for each segment. Methods for comparing the two distributions included calculating the test statistics mentioned in Hourdakis (2003), comparing the sample means and standard deviations, and comparing the two proportion histograms. But due to eight travel time segments in each direction, only error difference was calculated and restricted to 30% difference between field travel time and simulated travel time. From the standard deviation of twenty runs with 5% error, the number of runs was calculated.

$$\text{Number of Runs} \geq \frac{Z^2\, S^2}{\epsilon^2}$$

Where Z = Z value of 95% confidence interval

S = Travel time standard deviation of sample runs

ϵ = acceptable error of travel time in sec

With above equation, each segment gives different number of runs and the maximum was 353 runs and the second maximum was 44 runs. Because of time limitations, the second maximum was considered. Each simulation run was executed 50 times, with each run using a different random seed value.

3.6 SYNCHRO Optimization of Offsets and Splits

The validated model was built in SYNCHRO with same intersection turning volumes, lane geometry and green splits. SYNCHRO v5.0 is macroscopic simulation software was used only for optimization of offsets and splits of the arterial corridor. All other model parameters were transferred to SYNCHRO including the actuated signal controls (pseudo signal) which was designed at cross walk locations. Different cases were performed using optimization of offsets of the network.

First, the offsets of the network were optimized without crosswalk. The obtained offsets were transferred to VISSIM no crosswalk model and VISSIM with crosswalk model. The MOEs like travel time, delay, queue length and number of stops were obtained through simulations.

Second, two crosswalk locations (pseudo signal) were selected as actuated uncoordinated in signal window of SYNCHRO and other four intersections were selected as pre-timed control, then the model was optimized for offsets and splits. The splits at actuated uncoordinated signals were locked in order to use the field splits which were obtained from pedestrian walking time. The optimized offsets were transferred to VISSIM model and the measures of effectiveness were generated with the simulation runs.

Third, the two crosswalk locations (pseudo signals) were changed to actuated-coordinated in SYNCHRO i.e. the actual signals were used at two crosswalk locations. The splits at actuated coordinated signals were locked in order to use the field splits which were obtained from pedestrian walking time. Then the model was optimized for offsets. The optimized offsets were transferred to VISSIM model and the measures of effectiveness were generated with the simulation runs.

CHAPTER 4

CASE STUDY

The objective of this research is to evaluate the impact of midblock pedestrian crossing on arterial level of service and to develop the methodology to model the midblock pedestrian crossing as pseudo signal. The following steps were considered in this case study.

Using the methodology as explained in previous chapter, data was collected from the field and the model was developed in VISSIM 4.20, calibrated to reproduce the actual operations at the study location during the midday period. The pseudo signal was modeled at the two crosswalk locations in the study area. Since the network has 9 intersections, the calibration to the exact match with the field travel times was more difficult. This Calibrated model with midblock pedestrian Crosswalk called as Case 1. Also, total delays, queue length and number of stops were obtained through simulations in VISSIM for entire network to see the effect of midblock pedestrian crossing on arterial traffic.

Based on Calibrated model, evaluate the MOEs without pedestrian crossing hereafter called as “Case 2”. Case 2 represents the typical practice in the field that underestimates the delays at midblock pedestrian crossing locations. To evaluate the effect of midblock pedestrian crossing on arterial traffic, Case 1 and Case 2 was compared.

Case 3 is evaluating the MOEs of the arterial using the model without midblock pedestrian crossing using the optimized offsets and splits from SYNCHRO without midblock pedestrian crossing. This is just repeating the Case 1 condition but with optimized offsets. With the same offsets and splits from SYNCHRO, evaluate the MOEs of the arterial using the model with midblock pedestrian crossing. This scenario is called as Case 4. Case 3 and Case 4 was compared to see the effect of midblock pedestrian crossing on arterial traffic after optimization of offsets.

In Case 3, the pedestrians were not accounted at midblock pedestrian crossing and only the intersections were considered while optimizing the offsets and splits in SYNCHRO. In order to account for pedestrian traffic at midblock pedestrian crossing, pseudo signal at midblock pedestrian crossing was considered as actuated-uncoordinated signal and the offsets and splits were optimized. With this optimized offsets and splits, VISSIM model with midblock pedestrian crossing was run to obtain the MOEs of arterial. This was called as Case 5. Case 4 and Case 5 was compared to see the impact of incorporation of pedestrians while optimizing the offsets and splits on arterial traffic.

In order to test the impact of actual signals at midblock pedestrian crossing, the pseudo signals at midblock pedestrian crossing was optimized for offsets and splits as actuated-coordinated signal. With this optimized offsets and splits, the VISSIM model with midblock pedestrian crossing as pseudo signal was run to obtain the LOS of arterial. This situation was called as Case 6. Case 6 was compared with Case 4 and Case 5 to see the effectiveness of actual signals in place of midblock pedestrian crossing.

To evaluate the impact at different traffic conditions, the same procedure was followed for AM and PM peak traffic and the results for entire network was generated for cases 3, 4, 5 and 6. Since, the pedestrian flow data was not available for AM and PM peak traffic, the same pedestrian data and same midday pedestrian flow was used for AM and PM traffic. Green splits, cycle length and offsets for Maryland Parkway corridor along study area was derived from Signal Event Log for AM traffic and for PM traffic, the same midday signal data was used.

4.1 Case 0 – Field Travel Times

To determine actual travel times for calibrating the simulation model, a standard license plate survey was conducted on November 7, 2006 (Tuesday) on northbound and southbound Maryland Parkway between the hours of 11:00 AM and 1:00 PM. Observers were stationed at downstream of nine intersections including mid-block crossing locations as shown in Table 4-1.

The observers recorded the first three characters on each license plate as well as the time at which the vehicle crossing the downstream location. The recorded times between matching license plates produced average link travel time at each of the eighteen segments. The North bound segments are Tropicana to Elizabeth, Elizabeth to University Road, University Road to Del Mar, Del Mar to Harmon, Harmon to University East, University East to Rochelle, Rochelle to Cottage Grove and Cottage Grove to Flamingo.

Table 4-1 Maryland Parkway Intersections between Tropicana and Flamingo

No.	Cross streets on Maryland Parkway	Type of Intersection
1	Tropicana Avenue	Signalized Intersection
2	Elizabeth	Unsignalized Intersection
3	University Road	Signalized Intersection
4	Del Mar Street	Midblock Crosswalk Location / Unsignalized Intersection
5	Harmon Avenue	Signalized Intersection
6	University East	Midblock Crosswalk Location / Unsignalized Intersection
7	Rochelle Avenue	Unsignalized Intersection
8	Cottage Grove	Unsignalized Intersection
9	Flamingo Road	Signalized Intersection

The Southbound segments are Flamingo to Cottage Grove, Cottage Grove to Rochelle, Rochelle to University East, University East to Harmon, Harmon to Del Mar, Del Mar to University Road, University Road to Elizabeth and Elizabeth to Tropicana. Table 4-2 and Table 4-3 shows the calculated travel times on each segment.

Table 4-2 Link travel time on Maryland Parkway NB from license plate study

Segment	Travel Time (sec)	Standard Deviation	Sample size
Tropicana - Elizabeth	13.6	2.20	88
Elizabeth – University Road	21.1	6.97	58
University Road – Del Mar	22.1	11.90	82
Del Mar - Harmon	27.0	12.78	36
Harmon – University East	18.9	5.09	36
University East - Rochelle	15.3	2.84	173
Rochelle – Cottage Grove	4.8	2.13	115
Cottage Grove – Flamingo	52.8	19.24	65

Table 4-3 Link travel time on Maryland Parkway SB from license plate study

Segment	Travel Time (sec)	Standard Deviation	Sample size
Flamingo – Cottage Grove	22.9	1.81	31
Cottage Grove - Rochelle	6.0	1.43	19
Rochelle – University East	17.5	5.15	20
University East - Harmon	19.9	10.22	53
Harmon – Del Mar	24.4	7.89	78
Del Mar – University Road	19.6	15.42	51
University Road - Elizabeth	27.2	7.07	61
Elizabeth - Tropicana	79.2	23.02	55

4.2 Case 1 – Calibrated Model with Midblock Crossings as Pseudo Signal

The geometry of Maryland Parkway was necessary to obtain for an accurate model. The digital aerial photography was obtained from the Clark County Department of Public Works website which was used for exact dimensions of the study network. Bus timetables along Maryland Parkway, Flamingo and Tropicana were accessed on the website for Citizen's Area Transit (CAT) on November 29, 2004. Bus stop locations were determined through the aerial photography published by Clark County Department of Public Works website.

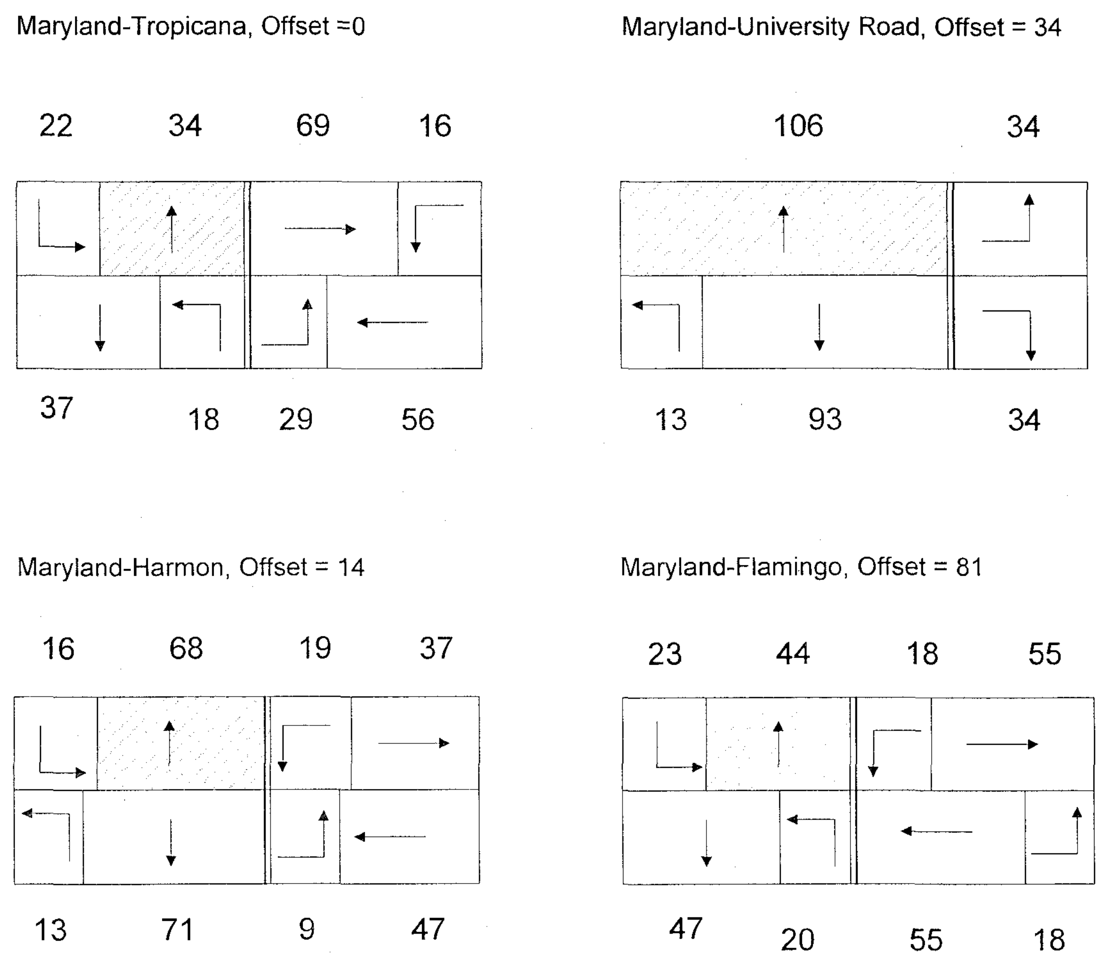
The green times, cycle lengths, offsets, cycle phasing, and signal actuation design can be derived from signal log as discussed in methodology (Chapter 3). Tropicana was taken as master intersection and northbound start of red was taken as reference phase. Total splits and offsets along study network were shown in Table 4-4 and phase diagrams with phase durations were shown in Figure 4-1. The Shaded phase of the intersections shown in Figure 4-1 represents the reference phase.

Table 4-4 Total splits and offsets on Maryland Parkway along Study Network

	offset	SBT	SBL	NBT	NBL	EBT	EBL	WBT	WBL
Tropicana/ Maryland	0	37	22	34	18	69	29	56	16
University Road/Maryland	34	93		106	13		34		
Harmon/ Maryland	14	71	16	68	13	37	9	47	19
Flamingo/ Maryland	81	47	23	44	20	55	18	55	18

Intersection volumes, Lane geometry were collected from Kimley-Horn Associates for the AM and PM peak hours. Since this study is on midday period, with the help of traffic counts from Nevada Department of Transportation the intersection turning movements were adjusted with the factors as described in the Methodology (Chapter 3).

The model was built in VISSIM 4.20 with all adequate geometry conditions. Each segment was built as links and connectors were given to connect the lanes. The green splits, conflicting phases and offsets were entered into NEMA Controller of VISSIM 4.20. Travel time sections were given from downstream to downstream intersection as surveyed in the field. As discussed in methodology (Chapter 3), the speed distribution was obtained from Merrill (2005). As a global calibration parameter speed distribution existing at the field was input to the model.



Key: Shaded phase represents the reference phase of the intersection

Figure 4-1 Phase Diagrams and Signal Timings of Signalized Intersections in Study Area

4.2.1 Pseudo Signal Design

The midblock pedestrian crossing was modeled as a four legged signalized herein referred to as a "pseudo signal". Timing parameters of this pseudo signal were designed so as to reproduce the observed effect of the crossing on vehicular traffic. Operational parameters to be determined are number of pedestrians crossing the road at crosswalk location which was converted into equivalent number of vehicles in respective direction. Actuated signal was designed as two phase signal. First phase NB and SB direction and second phase as WBR and EBR direction. Green splits and cycle length were calculated as discussed in the methodology (Chapter 3).

Pedestrian counts were taken at the mid-block crosswalks on November 9, 2006 (Thursday) between 11:00 AM to 11:45 AM in Eastbound direction and 12:00 PM to 12:45 PM in westbound direction and then projected from 45 minutes count to hourly counts. Pedestrian crossing times and the number of pedestrian occurrences were surveyed on November 9, 2006 along eastbound direction of crosswalks from 11:00 AM to 11:45 AM and along westbound direction of crosswalks from 12:00 PM to 12:45 PM. The observed pedestrian flow and crossing times were tabulated in Table 4-5. One occurrence was counted as when the vehicles stop for pedestrian(s) to cross the road at the midblock crossing from one end to other end.

Table 4-5 Pedestrian occurrences and crossing time at crosswalk locations

	Del Mar Crosswalk		University East Crosswalk	
	Westbound	Eastbound	Westbound	Eastbound
Number of Pedestrians Per Hour	116	121	88	77
Maximum Crossing time of Pedestrian (sec)	27	27	27	29
Minimum Crossing time of Pedestrian (sec)	16	13	13	14
Average crossing time of Pedestrian (sec)	20	20	20	19
Number of occurrences/Hr. (average of 2 days of surveys)	60	69	40	55

Calculation of Signal Timings of Pseudo Signal:

Maximum total split of Minor Street was considered as Maximum of Eastbound and Westbound Crossing time. Minimum total split of Minor Street was taken as average crossing time of pedestrian. Yellow time was assumed as 3 sec and all-red time was assumed as 1 sec. Green time was calculated as difference of total split and yellow and all-red time. The Signal timings was shown in Table 4-6

Table 4-6 Signal Timings of Minor Street at Pseudo Signal

	Del Mar		University East	
	Westbound Right (sec)	Eastbound Right (sec)	Westbound Right (sec)	Eastbound Right (sec)
Maximum Total Split (Maximum of EB and WB)	27	27	29	29
Minimum Total Split (Average Crossing Time of Pedestrian)	20	20	20	20
Yellow Time	3	3	3	3
All Red Time	1	1	1	1
Maximum Green Time	23	23	25	25
Minimum Green Time	17	17	17	17

Calculation of Cycle Length:

Total number of occurrences of pedestrians per hour is equal to the total number of actuations by vehicles per hour.

Total number of actuations per hour (consider maximum) on Del Mar = 69

Total number of actuations per hour (consider maximum) on University East = 55

Cycle length = Seconds in an hour/Number of occurrences in an hour

Cycle length = 3600/ Number of actuations per hour = 3600/69 = 52 sec

Cycle length = 3600/55 = 66 sec

Calculation of Total Splits on Major Street:

Major Street (Arterial) Total split = cycle length – Maximum total split of Minor Street.

Maryland Parkway NB total split at Del Mar = 52 – 27 = 25 sec

Maryland Parkway SB total split at University East = 66 – 29 = 37 sec

To convert the eastbound pedestrians and westbound pedestrians into equivalent eastbound vehicles and equivalent westbound vehicles, the actual numbers of pedestrians were entered into network as an input for iteration 1. The derived green splits and cycle length values were entered into NEMA Controller of pseudo signals in VISSIM software and the simulation was ran for 10 runs. From the signal change output file, the average total green time in simulation for 10 runs) of the required signal group was derived. Then, this total green time was compared with the average total pedestrians crossing time in the field of that direction (i.e) the eastbound vehicles total green time was compared with average total Eastbound pedestrians crossing time. The average total eastbound pedestrians crossing time can be calculated as product of average crossing time of all pedestrians and the number of occurrences of pedestrians per hour. The Typical calculations of iteration 1 was shown in Table 4-7

Table 4-7 Iteration 1 showing calculations of # of equivalent vehicles for pedestrians

	Del Mar EB	Del Mar WB	University East EB	University East WB
Average crossing time of pedestrian	20	20	20	20
Number of occurrences of pedestrians	66	66	55	55
Total crossing time of pedestrians (should be equal to green time in field)	1380	1380	1100	1100
Total number of vehicles input in network	119	119	83	83
Total Green time from simulation	1081	1064	861	843
Difference in Green time (between field and simulation)	299	316	239	257
Green time per vehicle from simulation (Green time in simulation/number of vehicles in network)	9.1	8.9	10.4	10.2
Number of additional vehicles required = Difference in Green time/ Green time per vehicle	32.9	35.3	23.0	25.3
Number of vehicles to be input in iteration 2	152	154	106	108

The number of vehicles to be input in iteration 2 will become input for next iteration. After a few iterations, total crossing time of pedestrians and total green time from simulation were nearly equal and also the input # of vehicles was equal to the number of vehicles to be input in next iteration. Once this was achieved, the numbers of equivalent vehicles were found. A graph showing the iterations done and the sum of squares of error was shown in figure 4-3. The graph was converged when the iterations was increased. The sum of the squares of error of Iteration 6 was found to be converged. The further iterations was continued to check the sum of the squares of error. From the graph it was observed there was an oscillation in sum of squares of errors after iteration 6. So, the number of vehicles obtained by iteration 6 was chosen as equivalent number of vehicles for the equivalent pedestrians in the field for pseudo signals.

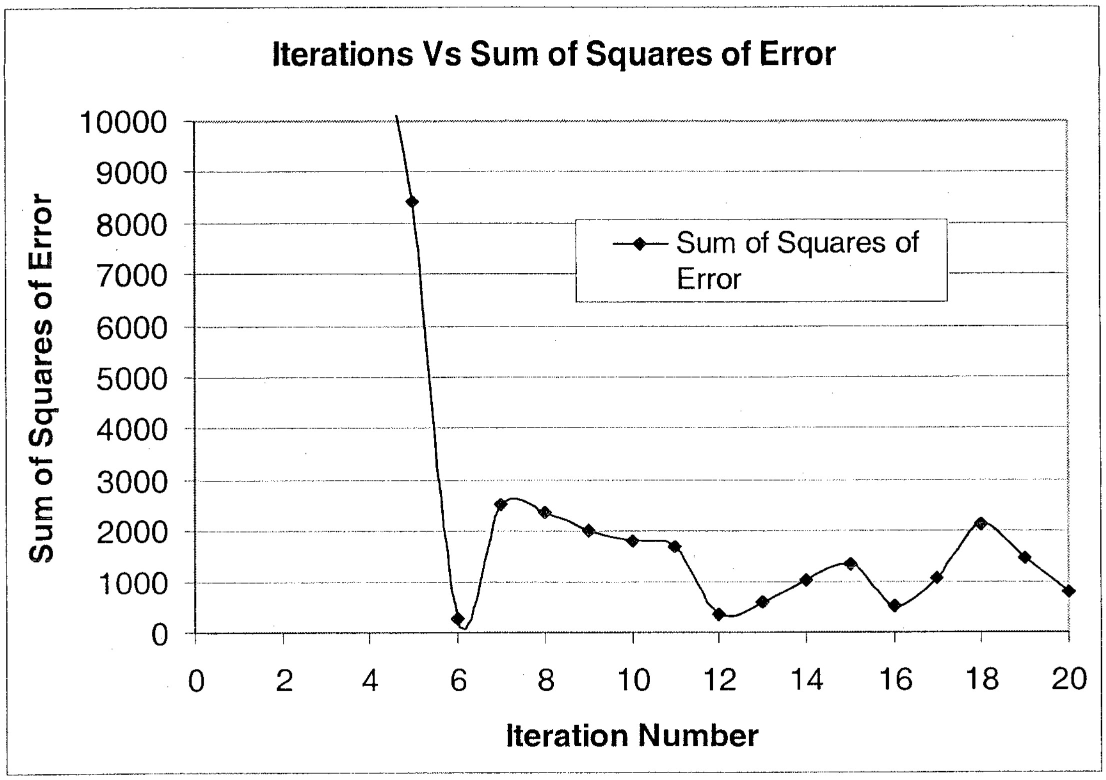


Figure 4-3 Graph showing the Sum of Squares of Error in Green Time of Pseudo Signal

4.2.2 Model Calibration

By following the methodology described in Section 3.5.1, the following candidate global VISSIM parameters were chosen for calibration, followed by a brief definition given by VISSIM 4.20 help manual.

- Minimum Headway: Minimum distance necessary from the subject vehicle to the vehicle in front while stopped for a lane change. Default value is 1.64 ft. The values used for calibration is 3.28, 6.56, 9.84 and 13.12 ft.
- Maximum Deceleration, Own Vehicle: Maximum deceleration rate of the subject vehicle that the subject vehicle will make a lane change. The values used were -16.40 ft/s^2 to -13.22 ft/s^2

- Maximum Deceleration, Trailing Vehicle: Maximum deceleration rate of the front vehicle that the subject vehicle will make a lane change. the values used for calibration were -13.12 ft/s^2 to -9.84 ft/s^2
- Accepted Deceleration, Own Vehicle: Accepted deceleration rate of the subject vehicle that the subject vehicle will make a lane change. The values used for calibration were -6.56 ft/s^2 to -3.28 ft/s^2
- Accepted Deceleration, Trailing Vehicle: Accepted deceleration rate of the ahead vehicle that the subject vehicle will make a lane change, -3.28 ft/s^2 to -1.64 ft/s^2
- Number of Observed Vehicles: Number of vehicles perceived by a subject vehicle. The values used for calibration were 1, 2, 3 and 4.
- Average Standstill Distance: Average distance between stopped vehicles. the values used were 13.12 to 1.64 ft
- Additive Part of the "Weidemann 74" Car-following Model: Factor involved in the computation of desired safety distance. The values used were 1 to 3
- Multiplicative Part of the "Weidemann 74" Car-following Model: Factor involved in the computation of desired safety distance. The values used were 2 to 4.
- Waiting Time before Diffusion: Waiting time before diffusion defines the maximum amount of time a vehicle can wait at the emergency stop position waiting for a gap to change lanes to stay on its route. When this time is reached, the vehicle is taken out of the network (diffusion). The values used were 20, 40 and 60 sec.

The above mentioned ten parameters were chosen for the process. Since, there are 18 travel time segments; the calibration was done with simplified methodology as mentioned in following steps:

- Travel time for default model was generated.
- Travel times for the entire network segments were generated for each of the above said parameter with maximum and minimum values.
- Differences were recorded between these values and those calculated from travel times generated using the default values for all model parameters.
- If the difference is more than ±5% in any of the one travel time segment, then those parameters were considered for calibration.
- Only three parameters, minimum headway, average standstill distance and waiting time before diffusion met the criteria for the following values.
- Minimum headway 9.84 ft and 13.12 ft, average standstill distance 3.28 ft, 9.84 ft and the waiting time before diffusion 20 sec. The values were tabulated in Table 4-8.
- When other parameters were changed, only 0.1 to 0.3 sec of difference can be seen between the default value travel time and the parameter changed travel time.
- The travel times obtained from the values of these three parameters was compared with the travel times from the field.
- Then combinations were made between these five values of three parameters (named as ACE, ACD, BCE and BCD. refer A, B, C, D and E from Table 4-9) and model was calibrated with those combinations.

Table 4-8 Travel time of individual calibration parameters

Segment	Field Value (sec)	Avg. Standstill Distance (ft)		Waiting Time before Diffusion	Minimum Headway (ft)	
Northbound		13.12	3.28	20 sec	9.84	13.12
Naming for Combination		A	B	C	D	E
Tropicana – Elizabeth	13.6	11.9	11.8	11.8	12.0	12.2
Elizabeth – University Road	21.1	19.7	19.7	19.8	19.7	19.9
University Road – Del Mar	22.1	23.8	23.7	23.6	23.7	23.8
Del Mar - Harmon	27.0	29.8	29.7	29.9	29.9	29.9
Harmon – University East	18.9	24	24	23.9	23.9	24.2
University East - Rochelle	15.3	15.9	15.8	15.8	15.7	15.8
Rochelle – Cottage Grove	4.8	6.9	4.4	4.5	4.4	4.4
Cottage Grove – Flamingo	52.8	61.4	58.1	54.3	57.8	58.9
Southbound						
Flamingo – Cottage Grove	22.9	24.1	23.6	24.1	24.1	24.3
Cottage Grove - Rochelle	6.8	4.2	4.2	4.2	4.2	4.2
Rochelle – University East	17.5	21.8	21.7	21.8	21.9	22.1
University East - Harmon	19.9	23.4	23.2	23.2	23.3	24.3
Harmon – Del Mar	24.4	26.1	25.9	26.1	26.1	27.1
Del Mar – University Road	19.6	18.9	18.9	18.6	18.9	18.9
University Road - Elizabeth	27.2	29.0	26.6	26.9	26.9	27.9
Elizabeth - Tropicana	79.2	115.1	110.1	106.1	113.1	116.1

- The recorded travel time was shown in table 4-9. Almost in all the four combinations, the travel time segments are equal other than the last segment of Northbound and Southbound. So, the model which has travel time close to the field travel time for these segments was considered as calibrated model.
- The sum of the squares of the difference between the field travel time and calibrated time was found and the combination that has the minimum sum of squares of error was considered as final calibrated model. In this case Model BCD is considered as calibrated model.

Table 4-9 Travel Time of Combinations of Calibration Parameters

Segment	Field Values	ACE	ACD	BCE	BCD
Northbound	(Sec)	(sec)	(sec)	(sec)	(sec)
Tropicana – Elizabeth	13.6	12.2	12.1	12.1	12.1
Elizabeth – University Road	21.1	19.9	19.9	19.9	19.9
University Road – Del Mar	22.1	23.8	23.7	23.8	23.7
Del Mar - Harmon	27.0	29.9	29.9	29.8	29.9
Harmon – University East	18.9	24	24.1	24.1	24.0
University East - Rochelle	15.3	15.9	16.1	15.9	15.9
Rochelle – Cottage Grove	4.8	4.6	4.6	4.4	4.4
Cottage Grove – Flamingo	52.8	59.3	59.1	57.2	56.9
Southbound					
Flamingo – Cottage Grove	22.9	24.1	24.2	24.1	24.1
Cottage Grove - Rochelle	6.8	4.2	4.2	4.2	4.2
Rochelle – University East	17.5	21.8	21.8	21.9	21.8
University East - Harmon	19.9	23.2	23.2	23.2	23.1
Harmon – Del Mar	24.4	26.1	26.1	26.4	26.1
Del Mar – University Road	19.6	18.6	18.6	18.7	18.6
University Road - Elizabeth	27.2	27.5	27.4	27.2	27.1
Elizabeth - Tropicana	79.2	110.9	111.2	114.6	109.1

From the travel time standard deviation of twenty runs with 5% of travel time in sec was considered as error in travel time, the number of runs was calculated.

$$\text{Number of Runs} \geq \frac{Z^2 S^2}{\epsilon^2}$$

Where Z = Z value of 95% confidence interval

S = standard deviation of sample runs

ϵ = error of travel time in sec

With above equation, each travel time segment gives different number of runs and the maximum was 353 runs and the second maximum was 44 runs. Because of time limitations, the second maximum was considered. Each simulation run was executed 50 times, with each run using a different random seed value. The difference between the

Travel time from Field and travel time of calibrated model was shown in Table 4-10 and 4-11.

Table 4-10 Field Travel Times and Calibrated Travel Time of Maryland Parkway NB

Segment	Field Travel Time – Case 0 (sec)	Calibrated Travel Time – Case 2(sec)	Difference in Travel Time (sec)	% Difference
Tropicana - Elizabeth	13.6	12.1	-1.5	-11 %
Elizabeth – University Road	21.1	19.9	-1.2	-6 %
University Road – Del Mar	22.1	23.7	1.6	7 %
Del Mar - Harmon	27.0	29.9	2.9	11 %
Harmon – University East	18.9	24.0	5.1	27 %
University East - Rochelle	15.3	15.9	0.6	4 %
Rochelle – Cottage Grove	4.8	4.4	-0.4	-8 %
Cottage Grove – Flamingo	52.8	56.9	4.1	8 %

For most of the segments, the percentage difference in travel time between field and calibrated model is less than 15% except, Harmon-University East in northbound, Rochelle-University East in southbound, the percentage difference is around 25% and Elizabeth-Tropicana in northbound the percentage difference is 38%. As, it is really difficult to calibrate all the segments to obtain 0% difference, this model was accepted as calibrated model.

Table 4-11 Field Travel Times and Calibrated Travel Time of Maryland Parkway SB

Segment	Field Travel Time – Case 0 (sec)	Calibrated Travel Time – Case 2 (sec)	Difference in Travel Time (sec)	Difference in Travel Time
Flamingo – Cottage Grove	22.9	24.1	1.2	5 %
Cottage Grove - Rochelle	6.0	4.2	-1.8	-30 %
Rochelle – University East	17.5	21.8	4.3	25 %
University East - Harmon	19.9	23.1	3.20	16 %
Harmon – Del Mar	24.4	26.1	1.7	7 %
Del Mar – University Road	19.6	18.6	-1.0	-5 %
University Road - Elizabeth	27.2	27.1	-0.0	0 %
Elizabeth - Tropicana	79.2	109.1	29.9	38 %

From Figure 4-4, the difference in travel time was low for most of the travel time segments except in northbound, Harmon - University East. Harmon-University East in northbound is a crosswalk location and estimating more delays. From figure 4-5, in southbound, Cottage Grove-Rochelle is smaller segment hence, can be ignored. Rochelle-University East is a crosswalk location where the big difference can be seen in travel times. The crosswalk at University East as represented by the pseudo signal was producing higher delays in both northbound and southbound direction. The other crosswalk at Del Mar has 7% difference between field and calibrated travel time. Elizabeth – Tropicana is the last segment of the network, where travel time is much higher than the observed travel time which implies the over estimation of delays.

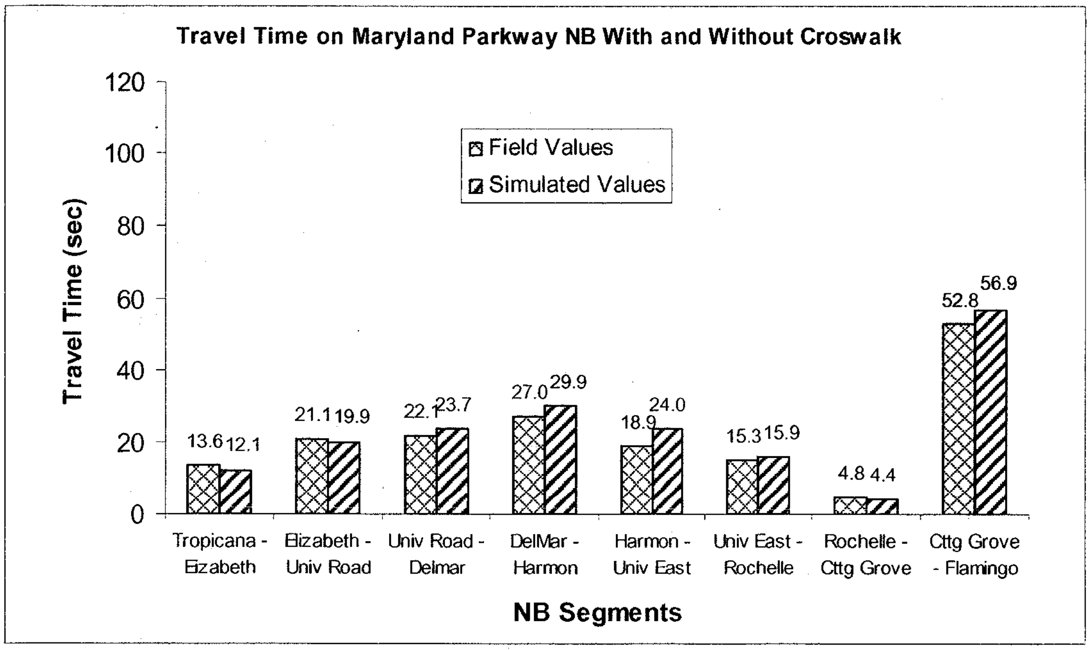


Figure 4-4 Travel Time on Maryland Parkway NB of Field and Calibrated Values

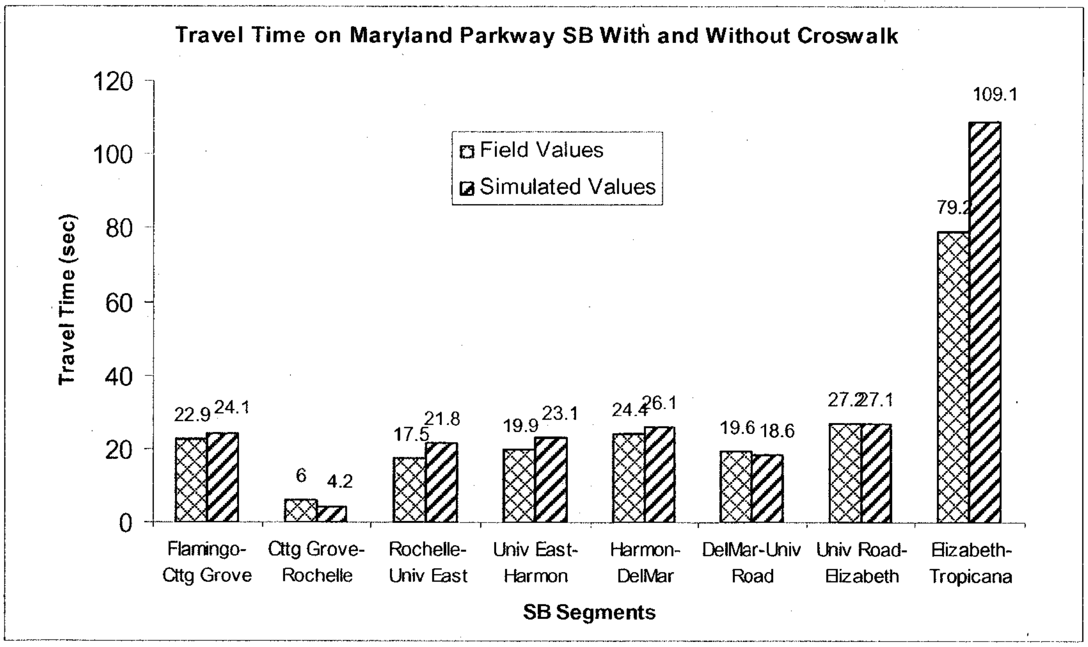


Figure 4-5 Travel Time on Maryland Parkway SB of Field and Calibrated Values

4.2.3 Statistical Analysis

One of the statistical tests used to see whether the difference between means of Field Travel Time and the simulated travel time was significant or not was Two Samples-Estimating the Difference between Two Means (Two-Sample Pooled T-Test). Since the data is from different experimental unit, it was considered as unpaired. The statistical software Minitab 15 was used to perform Two-Sample Pooled T-Test. The hypothesis testing was done with Null Hypothesis (H_0) as the difference in means was zero. If null Hypothesis was accepted then there was no significant difference between the field travel time and the simulated travel time. The alternate hypothesis (H_1) as the difference in means was not equal to zero. If alternate hypothesis was accepted then there was significant difference between the field travel time and the simulated travel time. Statistical testing was done for 95% confidence interval.

Null Hypothesis, H_0: $\mu_1 - \mu_2 = 0$

Alternate Hypothesis, H_1: $\mu_1 - \mu_2 \neq 0$

Two-Sample T-Test and CI

```
Sample    N        Mean    StDev    SE Mean
1         88       13.60    2.20     0.23
2         50       12.100   0.270    0.038


Difference = mu (1) - mu (2)
Estimate for difference:  1.500
95% CI for difference:  (1.028, 1.972)
T-Test of difference = 0 (vs not =): T-Value = 6.31 P-Value = 0.000 DF = 91
```

The sample test result from the Minitab 15 was shown above. Accept the alternate hypothesis if P-Value is less than 0.05, which indicates that the difference between the means is not equal to zero. In the above sample, the alternate hypothesis was accepted. Since alternate hypothesis was accepted, statistically there is difference between the field travel time and simulated travel time.

Measure(s) of effectiveness, travel time was collected from the field to calibrate the model. The model was calibrated in VISSIM and the simulations were run. The difference in mean field travel time and the mean simulated travel time was statistically tested and the results were shown in Table 4-12 and Table 4-13.

Table 4-12 Statistical Results for the difference in Travel Time of Case 0 and Case 1 on Northbound

Segment	Field Travel Time – Case 0 (sec)	Calibrated Travel Time – Case 1(sec)	Difference in Travel Time	Statistical Result
Tropicana - Elizabeth	13.6	12.1	1.5	*Reject H_0*
Elizabeth – University Road	21.1	19.9	1.2	Accept H_0
University Road – Del Mar	22.1	23.7	-1.6	Accept H_0
Del Mar - Harmon	27.0	29.9	-2.9	Accept H_0
Harmon – University East	18.9	24.0	-5.1	*Reject H_0*
University East - Rochelle	15.3	15.9	-0.6	Accept H_0
Rochelle – Cottage Grove	4.8	4.4	0.4	Accept H_0
Cottage Grove – Flamingo	52.8	56.9	-4.1	Accept H_0

Table 4-13 Statistical Results for the difference in Travel Time of Case 0 and Case 1 on Southbound

Segment	Field Travel Time – Case 0 (sec)	Calibrated Travel Time – Case 1 (sec)	Difference in Travel Time (sec)	Difference in Travel Time
Flamingo – Cottage Grove	22.9	24.1	-1.2	*Reject H_0*
Cottage Grove - Rochelle	6.0	4.2	1.8	*Reject H_0*
Rochelle – University East	17.5	21.8	-4.3	*Reject H_0*
University East - Harmon	19.9	23.1	-3.20	*Reject H_0*
Harmon – Del Mar	24.4	26.1	-1.7	Accept H_0
Del Mar – University Road	19.6	18.6	1.0	Accept H_0
University Road - Elizabeth	27.2	27.1	0.0	Accept H_0
Elizabeth - Tropicana	79.2	109.1	-29.9	*Reject H_0*

In the northbound direction, other than Tropicana-Elizabeth and Harmon-University East, statistically there is no significant difference between field travel time and calibrated travel time. But in southbound, only for Harmon-Del Mar, Del Mar – University Road and University Road - Elizabeth segments statistically there is no significant difference in the mean field travel time and mean calibrated mean travel time. Other 5 segments have significant difference in mean travel time from field and calibrated mean travel time.

4.3 Case 2 – Model without Midblock Pedestrian Crosswalks

The model was built with no crosswalks and analyzed to see the effect of midblock pedestrian crossing on arterial traffic. The four legged intersection was changed into T-intersection at crosswalk locations which was as same as existing geometry. No crosswalks were given at crosswalk locations. Crosswalk locations were made as stop control. Minor streets, Del Mar and University East on Maryland Parkway were built as stop control. The simulation was ran for 50 runs and the travel time, total delay, stopped delay and number of stops were tabulated in Table 4-13 and 4-14 for the northbound and the southbound directions respectively.

4.4 Case 3 – Model with Crosswalks and Offsets Optimized only for Signalized Intersections

The offsets and splits optimized in SYNCHRO without crosswalk model were transferred to VISSIM pseudo signal model (Case 1). Pseudo signal design was kept same. Optimized splits and offsets were transferred to other four signalized intersections in VISSIM NEMA controller. This case implies offsets were optimized without considering pedestrians. In other words, the model was optimized in SYNCHRO without pedestrians and then pedestrians were introduced in VISSIM to see the effect of pedestrian crossing on arterial traffic. Travel time, total delay, stopped delay and number of stops was observed for northbound and southbound segments.

4.5 Case 4 – Model without Crosswalks and Offsets Optimized for only Signalized Intersections

The Same model was built in SYNCHRO 5.0 without crosswalk. The two crosswalk locations were built as unsignalized intersection with stop control. In Unsignalized intersection, the through vehicles in arterial were set free and minor street vehicles were made to stop. SYNCHRO was used only for optimizing splits and offsets. The model was optimized for splits and offsets. Then, the optimized splits and offsets were transferred to VISSIM Case 1 Model (Model with no Crosswalk). Travel time, total delay, stopped delay and number of stops was observed for the northbound and the southbound segments.

4.6 Case 5 – Model with Crosswalks and Offsets Optimized with Crosswalks as Actuated Uncoordinated Signals

The crosswalk locations in the study network were made as Actuated Uncoordinated control in order to account for pedestrians. The pseudo signal timings and cycle length was entered at actuated uncoordinated control. The offsets and splits were then optimized. The splits at actuated uncoordinated control were locked in order to use the pseudo signal timing which reflects the pedestrian activities. The optimized splits and offsets were then transferred to VISSIM pseudo signal model (Case 2). This case implies offsets were optimized with considering pedestrians. In other words, the model was optimized in SYNCHRO with taking pedestrians into account. Travel time, total delay, stopped delay and number of stops was collected for the northbound and the southbound segments.

4.7 Case 6 – Model with Crosswalks and Offsets Optimized with Crosswalks as Actuated Coordinated Signal

The crosswalk locations in the study network were made as Actuated Coordinated control which is nothing but act as an actual signal to account for pedestrians. The pseudo signal timings and cycle length was entered at actuated coordinated control. In order to coordinate the offsets, the actuated signal cycle time should be in multiples of other signals cycle time. The Del Mar cycle time was changed from 52 seconds to 70 seconds and the difference was added to Northbound and Southbound maximum green time. The University East Cycle time was changed from 66 seconds to 70 seconds and the difference was added to northbound and southbound maximum green time. The splits at the pseudo signals were kept same to account for pedestrians. Other intersection splits were optimized individually and then only offsets were optimized with the existing splits option in offset optimization window. The optimized splits and offsets were then transferred to VISSIM pseudo signal model with midblock crosswalk. This case implies offsets were optimized with considering pedestrians crossing at an actual actuated signal which is designed to account for pedestrian activity rather than for vehicles at Minor Street. In VISSIM NEMA controllers the offsets of pseudo signals were coordinated with other signal offsets. Travel time, total delay, stopped delay and number of stops was collected for Northbound and Southbound segments.

CHAPTER 5

ANALYSIS OF RESULTS AND DISCUSSION

5.1 Effect of Midblock Pedestrian Crossing on Arterial Traffic

5.1.1 Evaluation Based on Calibrated Model

Calibrated model with midblock pedestrian crosswalk (Case 1) was compared with the model with no crosswalks (Case 2) and analyzed to see the effect of midblock pedestrian crossing on arterial traffic. The travel time, delay, and Number of stops were tabulated in Table 5-1 and 5-2 for Northbound and Southbound segments respectively for Case 1 and Case 2.

Table 5-1 Travel Time, Delay and # of Stops on Maryland NB for Case 1 and Case 2

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	1	2	1	2	1	2	1	2
Tropicana - Elizabeth	12.1	11.7	1.7	1.3	0.1	0.1	51	43
Elizabeth – University Road	19.9	19.4	4.3	3.8	1.5	1.4	259	248
University Road – Del Mar	23.7	12.7	11.8	0.9	5.5	0.0	1124	0
Del Mar - Harmon	29.9	30.3	16.4	16.7	9.7	11.3	1315	921
Harmon – University East	24.0	13.9	11.1	1.0	4.9	0.0	270	24
University East - Rochelle	15.9	14.1	3.2	1.4	0.3	0.1	0	66
Rochelle – Cottage Grove	4.4	4.3	0.4	0.3	0.0	0.0	762	0
Cottage Grove – Flamingo	56.9	55.6	32.7	31.6	24.2	0.8	5125	701

Table 5-2 Travel Time, Delay and Number of Stops on SB for Case 1 and Case 2

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	1	2	1	2	1	2	1	2
Flamingo – Cottage Grove	24.1	23.9	2.5	2.10	0	0.0	0	0
Cottage Grove - Rochelle	4.2	4.2	0.4	0.4	0	0.0	0	0
Rochelle – University East	21.8	14.0	9.1	1.2	3.7	0.0	764	0
University East - Harmon	23.1	21.8	9.4	8.2	5.1	4.3	530	401
Harmon – Del Mar	26.1	14.6	12.7	1.0	6.0	0.0	1088	0
Del Mar – University Road	18.6	16.8	7.1	5.3	3.6	3.1	606	283
University Road - Elizabeth	27.1	17.5	10.7	1.6	4.0	0.0	1300	0
Elizabeth - Tropicana	109.1	84.3	95.7	71.4	79.3	61.4	7453	1295

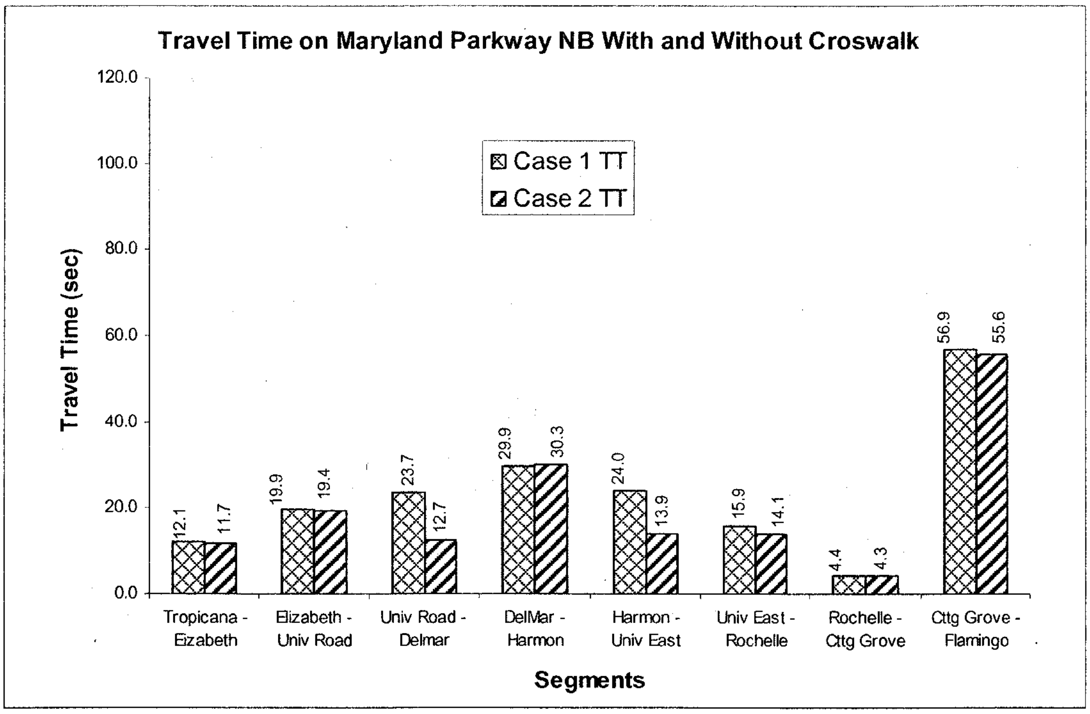
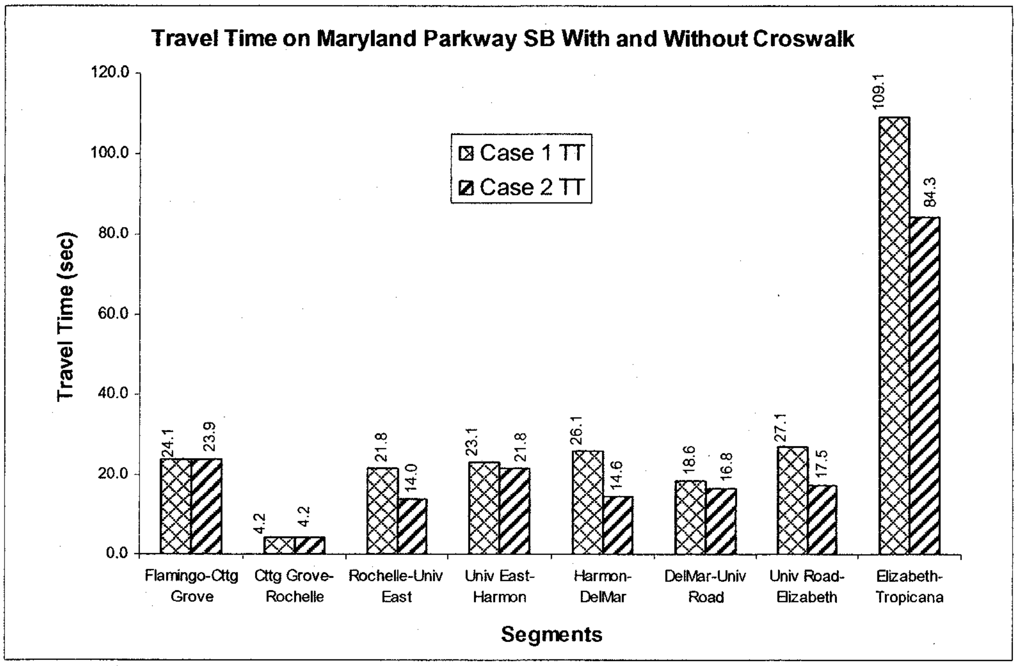


Figure 5-1 Travel Time on Maryland Parkway NB for Case 1 and Case 2



Key: Case 1 – Travel Time from Model with Midblock Pedestrian Crossing
Case 2 – Travel Time from Model without Midblock Pedestrian Crossing

Figure 5-2 Travel Time on Maryland Parkway SB for Case 1 and Case 2

The Figure 5-1 and Figure 5-2 was the graph between the mean travel time of arterial network with and without crosswalk in Northbound and Southbound direction. There is significant difference between the travel times of both cases.

- The two crosswalk segments University Road-Del Mar and Harmon-University E in Northbound (NB) and Rochelle-University East and Harmon-Del Mar in Southbound (SB) have big difference in travel time. This difference is due to the delays caused by crosswalks.
- And eventually due to delays at midblock pedestrian crosswalks, the vehicles messed up with the signal coordination and the segments of Case 1 at downstream of midblock pedestrian crosswalks would cause more delays than that of Case 2.

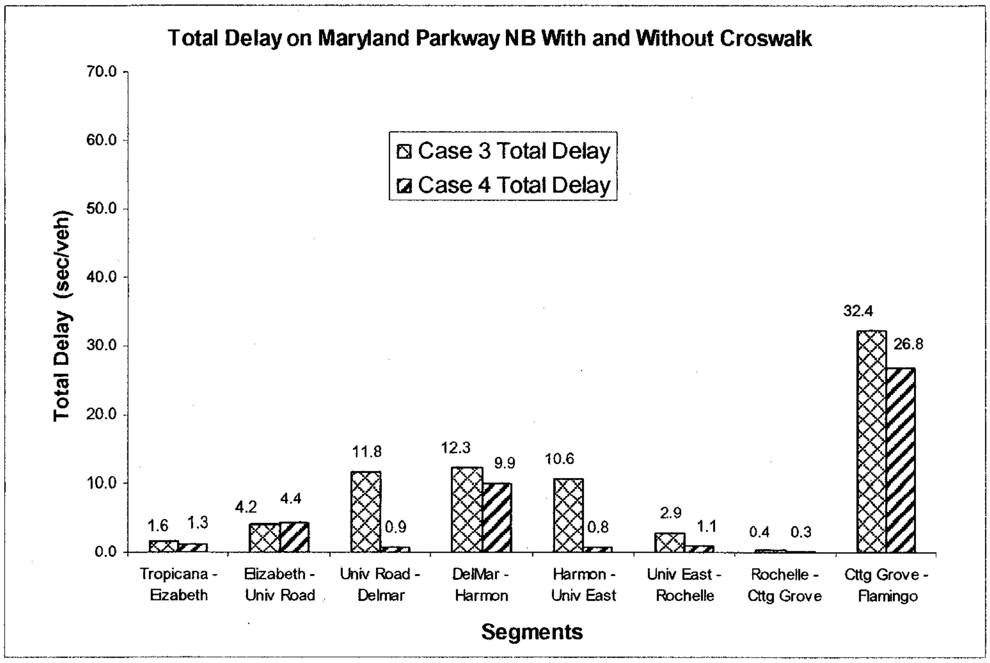
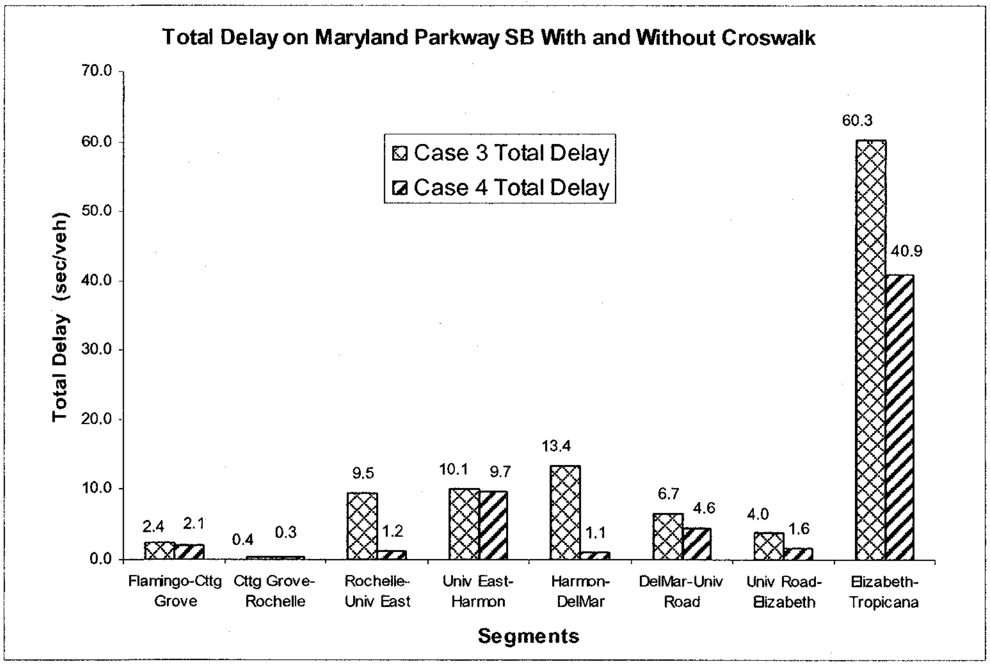


Figure 5-3 Total Delay on Maryland Parkway SB for Case 1 and Case 2

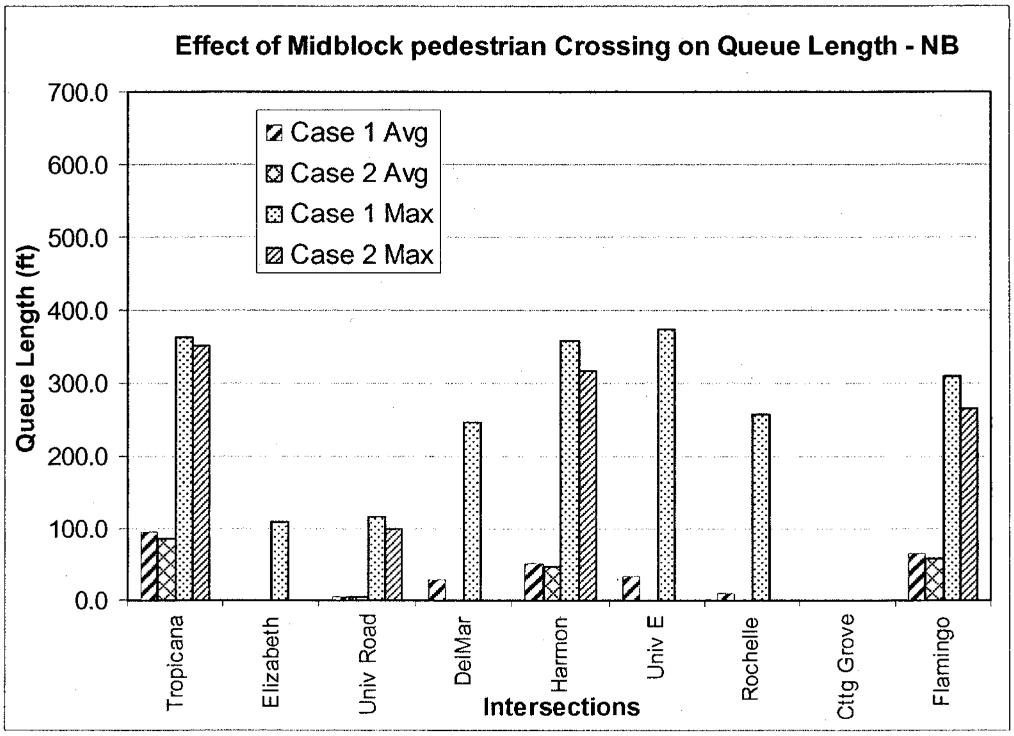


Key: Case 1 – Model with Midblock Pedestrian Crossing
Case 2 – Model without Midblock Pedestrian Crossing

Figure 5-4 Total Delay on Maryland Parkway SB for Case 1 and Case 2

The Total delay caused by with and without crosswalk cases was shown in Figure 5-3 and 5-4. The observations were as follows:

- In the northbound direction, only midblock pedestrian crosswalk locations have significant difference in delays. The University Road-Del Mar with midblock pedestrian crossing has 11.8 sec/veh of total delay and without midblock pedestrian crossing has only 0.9 sec/veh
- The other midblock pedestrian crossing at Harmon-Del Mar, for Case 1 it has 11.1 sec/veh and for Case 2 it has only 1.0 sec/veh
- In the southbound direction also, midblock pedestrian crossing locations has high delays for with Case 1and much low delays for Case 2
- In southbound, Elizabeth-Tropicana segment experienced the much higher delays. On field also, this segment was most of the times under grid lock which was experienced during License Plate travel time survey. Due to the grid lock at this segment, the previous segment University Road-Elizabeth also experiences some stopped delay.
- When Case 2 and Case 1 were compared, it is visually evident from the graphs; the downstream segments of with crosswalk conditions have higher delays than the downstream segments of without crosswalk condition.



Key: Case 1 Avg– Average Queue length of Model with Midblock Crossing
Case 2 Avg – Average Queue length of Model without Midblock Crossing
Case 1 Max – Maximum Queue length of Model with Midblock Crossing
Case 2 Max – Maximum Queue length of Model without Midblock Crossing

Figure 5-5 Queue length on Maryland Parkway NB for Case 1 and Case 2

Effect of midblock pedestrian crossing on Queue length was shown in the Figure 5-5 and 5-6 for Maryland parkway Northbound and Southbound direction respectively. Case 1 Avg. represents the average queue length and Case 1 Max. represents the maximum queue length of intersections without crosswalks scenario and Case 2 Avg. represents the average queue length and case 2 max. represents the maximum queue length of Case 2, with crosswalk scenario. The observation made were

- At both crosswalks, Del Mar and University East, the maximum queue length is equal to queue length at signalized intersections.

- Some unsignalized intersections like Rochelle in northbound direction and Elizabeth in southbound direction experiences maximum queue length in simulation with crosswalk scenario (Case2). The reason for this queue length could be these intersections located at downstream of the crosswalks.

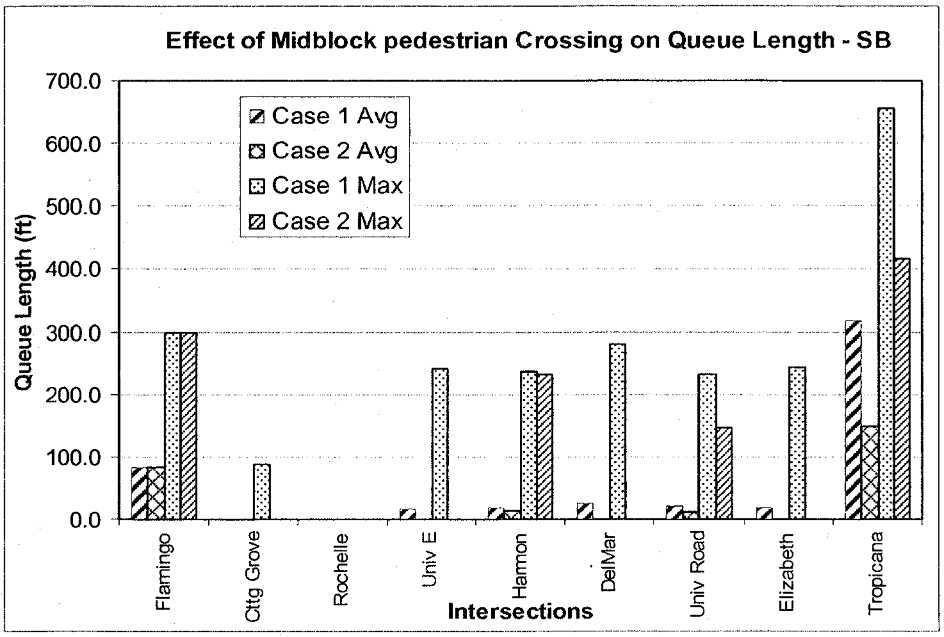


Figure 5-6 Queue Length on Maryland Parkway SB for Case 1 and Case 2

Effect on Number of stops with and without midblock pedestrian crossing was shown in Figure 5-7 and Figure 5-8 for Northbound and Southbound directions.

- There was high difference in number of stops than other MOEs between Case 1 and Case 2 at all signalized intersections and at midblock crosswalk locations
- Due to the more number of stops in Case 2, the vehicle emissions are high.
- The fuel consumptions will be higher in Case 2 than that of Case 1

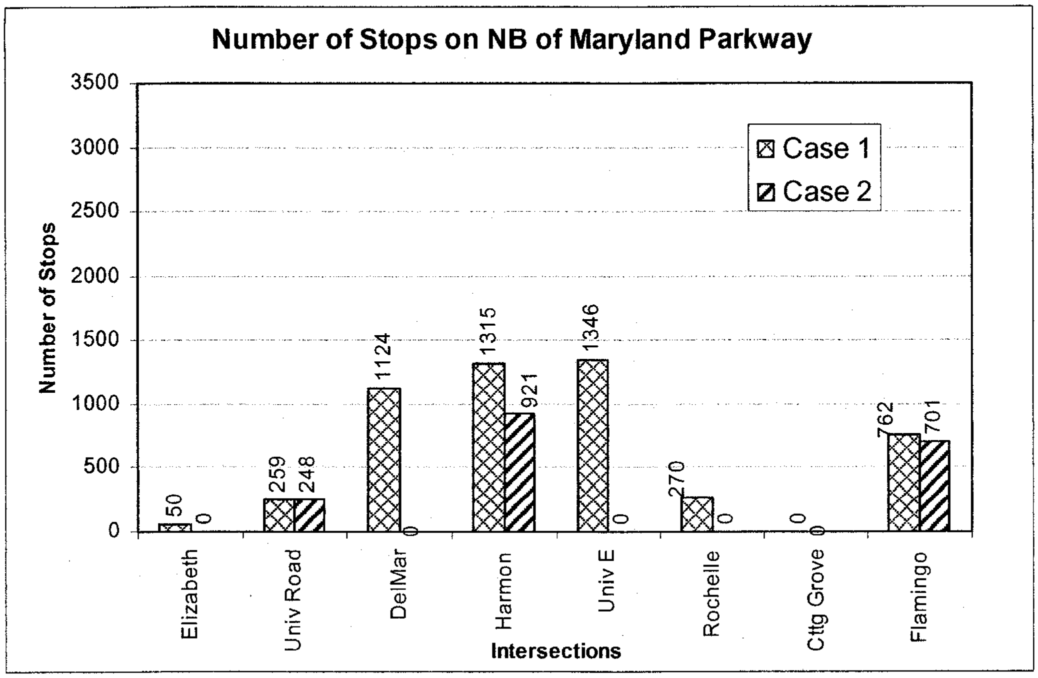
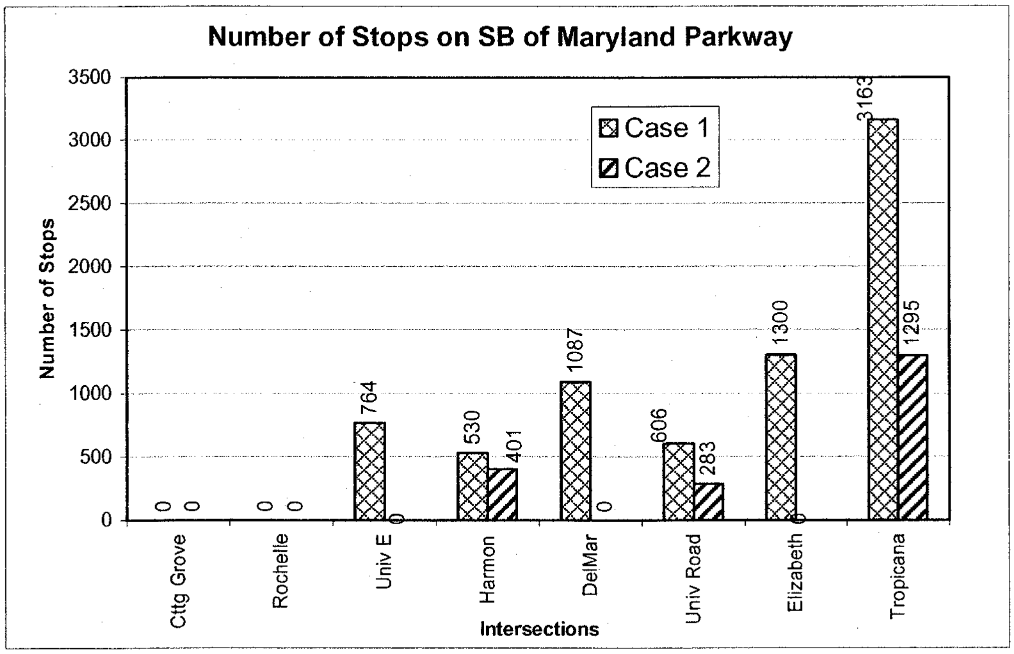


Figure 5-7 Number of stops on Maryland Parkway NB for Case 1 and Case 2



Key: Case 1 – Model with Midblock Pedestrian Crossing
Case 2 – Model without Midblock Pedestrian Crossing

Figure 5-8 Number of Stops on Maryland Parkway SB for Case 1 and Case 2

The significant difference between with and without midblock pedestrian crossing scenarios implies the effect of pedestrian midblock crossing on arterial traffic. Measures of Effectiveness, travel time, total delay, queue length and number of stops were increased due to the presence of pedestrian midblock crossings. Not only the segments with midblock pedestrian crossings caused more delay, travel time, queue length, the segments at downstream of midblock pedestrian crossings also have an adverse impact due to the presence of midblock pedestrian crossings. The Comparison between Case 1 and Case 2 for entire network was shown in Table 5-3.

Table 5-3 MOEs of Entire Network for Case 1 and Case 2 on NB and SB

		Northbound				Southbound			
		Case 1	Case 2	Diff. 1-2	% Diff	Case 1	Case 2	Diff 1-2	% Diff
Mid Day	Travel Time (v-h)	71	57	14	24.6	78	55	23	41.8
	Total Delay(v-h)	30	18	12	66.7	43	22	21	95.5
	Stopped Delay(v-h)	16	11	5	45.5	28	16	12	75.0
	Number of Stops	5125	1870	3255	174.1	7421	1979	5442	275.0

The following observations were made from the above table:

- When, midblock crossing was considered, the travel time is increased by 14 vehicle hours in northbound direction and 23 vehicle hours in southbound direction.
- There is significant increase in percentage of MOEs when pedestrian midblock crossing was considered.

- The percentage increase of number of stops when midblock pedestrian crossing considered was huge in both directions which directly increase the fuel consumption and emission of vehicles.

5.1.2 Effect of Optimized Offsets for with and without Midblock Crossing Models

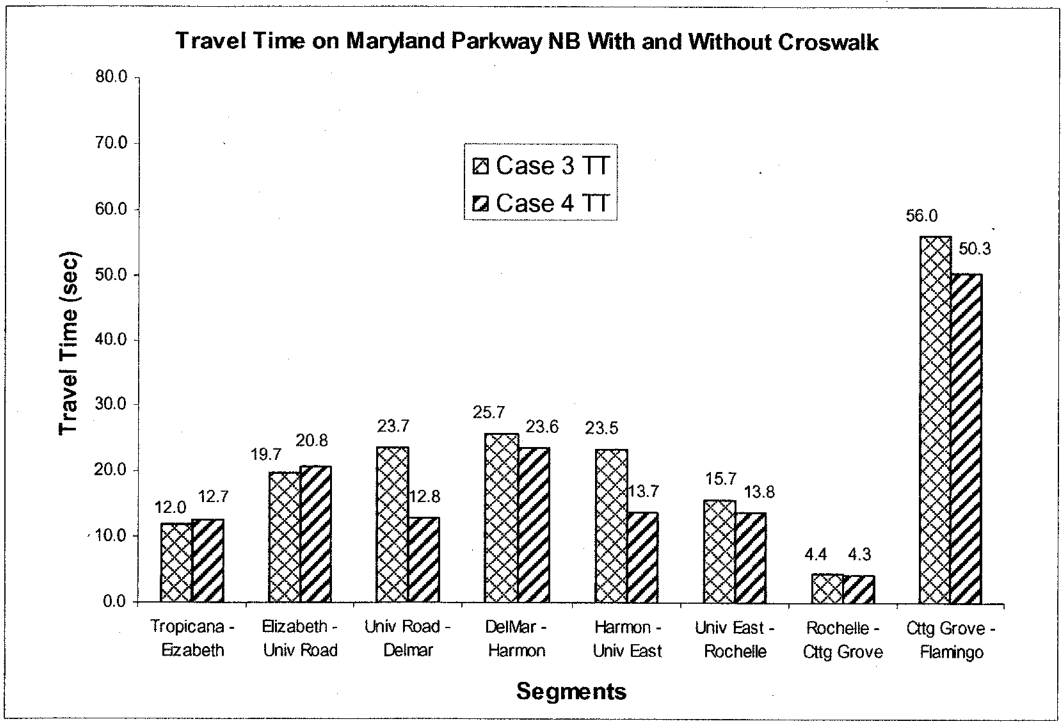
SYNCHRO was used to optimize the offsets and splits. The model without midblock pedestrian crossing was optimized for offsets and splits. These offsets and splits were then transferred to VISSIM Model with midblock pedestrian crossing (Case 3) and without midblock pedestrian crossing (Case 4). The model with midblock pedestrian crosswalk (Case 3) was compared with the model without crosswalks (Case 4) and analyzed to see the effect of midblock pedestrian crossing on arterial traffic after optimization of offsets and splits. The travel time, total delay, stopped delay and number of stops was tabulated in Table 5-4 and 5-5 for northbound and southbound directions respectively.

Table 5-4 Travel Time, Delay and # of Stops on Maryland NB for Case 1 and Case 2

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	3	4	3	4	3	4	3	4
Tropicana - Elizabeth	12.0	12.7	1.6	1.3	0.1	0.1	40	38
Elizabeth – University Road	19.7	20.8	4.2	4.4	0.9	1.2	263	360
University Road – Del Mar	23.7	12.8	11.8	0.9	5.1	0.0	1200	0
Del Mar - Harmon	25.7	23.6	12.3	9.9	7.2	6.1	832	535
Harmon – University East	23.5	13.7	10.6	0.8	4.6	0.0	1225	0
University East - Rochelle	15.7	13.8	2.9	1.1	0.3	0.0	234	0
Rochelle – Cottage Grove	4.4	4.3	0.4	0.3	0	0.0	0	0
Cottage Grove – Flamingo	56.0	50.3	32.4	26.8	24.6	19.9	684	592

Table 5-5 Travel Time, Delay and # of Stops on Maryland SB for Case 1 and Case 2

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	3	4	3	4	3	4	3	4
Flamingo – Cottage Grove	24.0	23.8	2.4	2.1	0.0	0.0	0	0
Cottage Grove - Rochelle	4.2	4.2	0.4	0.3	0.0	0.0	0	0
Rochelle – University East	22.2	14.6	9.5	1.2	4.1	0.0	790	0
University East - Harmon	23.6	24.5	10.1	9.7	5.1	4.7	629	546
Harmon – Del Mar	26.9	16.3	13.4	1.1	6.4	0.1	1157	49
Del Mar – University Road	18.0	17.9	6.7	4.6	3.2	2.5	578	253
University Road - Elizabeth	19.7	17.4	4.0	1.6	0.7	0.0	265	19
Elizabeth - Tropicana	72.9	53.0	60.3	40.9	48.3	32.2	1803	963



Key: Case 3 – Model with Midblock Pedestrian Crossing using optimized offsets
Case 4 – Model without Midblock Pedestrian Crossing using optimized offsets

Figure 5-9 Travel Time on Maryland Parkway NB for Case 3 and Case 4

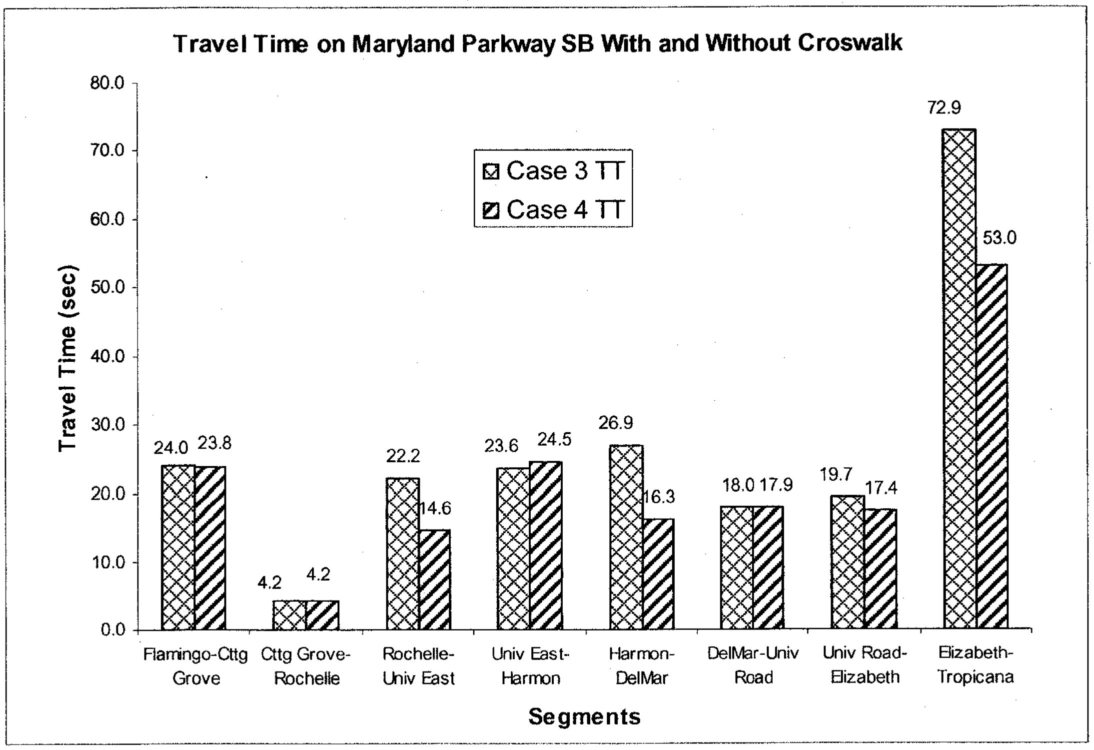


Figure 5-10 Travel Time on Maryland Parkway SB for Case 3 and Case 4

The Figure 5-9 and Figure 5-10 was the graph between the mean travel time of arterial network with and without crosswalk in northbound and southbound directions after optimization of offsets. There is significant difference between the travel times of both cases.

- After optimization of offsets and splits, there is no big difference in link travel times before and after optimization except for the segments Del Mar-Harmon in the northbound direction and Elizabeth-Tropicana in the southbound direction
- The segments with midblock pedestrian crossing case have high travel times than the segments without midblock pedestrian crossing case.

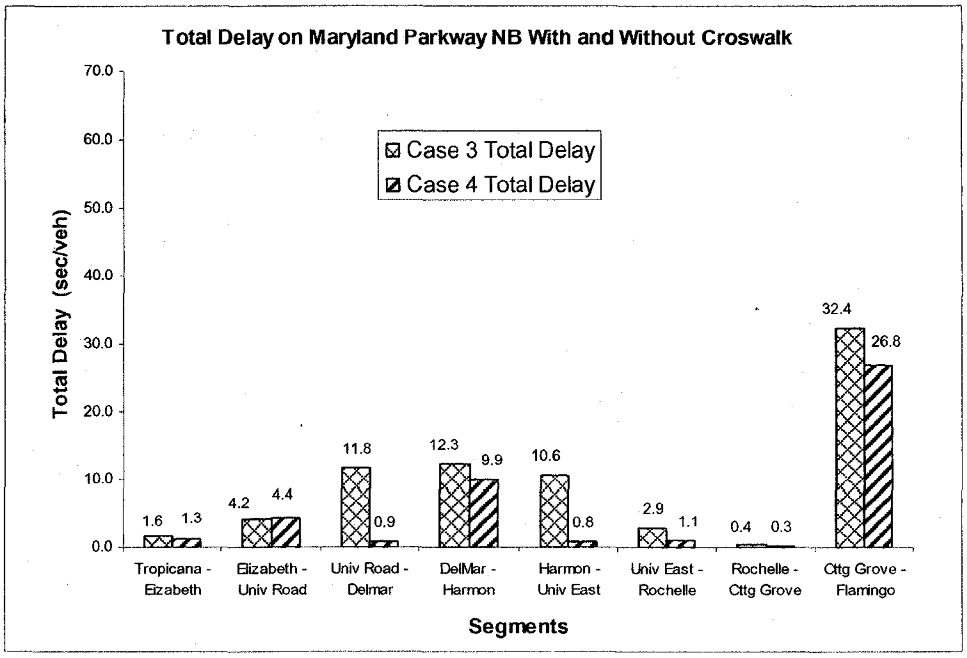
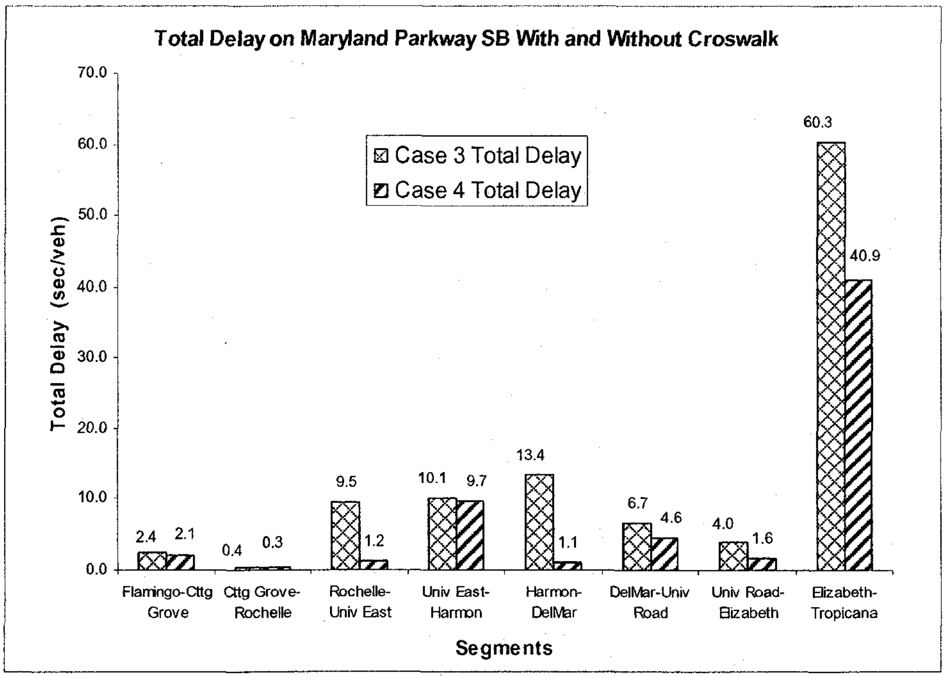
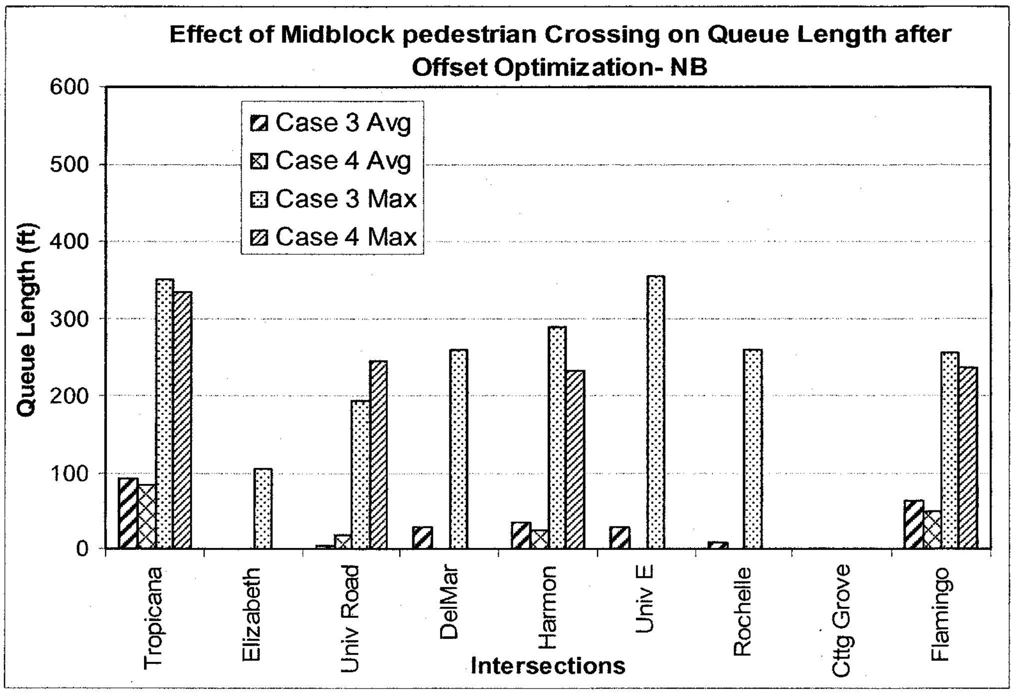


Figure 5-11 Total Delay on Maryland Parkway SB for Case 3 and Case 4



Key: Case 3 – Model with Midblock Pedestrian Crossing using optimized offsets
Case 4 – Model without Midblock Pedestrian Crossing using optimized offsets

Figure 5-12 Total Delay on Maryland Parkway SB for Case 3 and Case 4



Key: Case 3 – Model with Midblock Pedestrian Crossing using optimized offsets
Case 4 – Model without Midblock Pedestrian Crossing using optimized offsets

Figure 5-13 Queue length on Maryland Parkway NB for Case 3 and Case 4

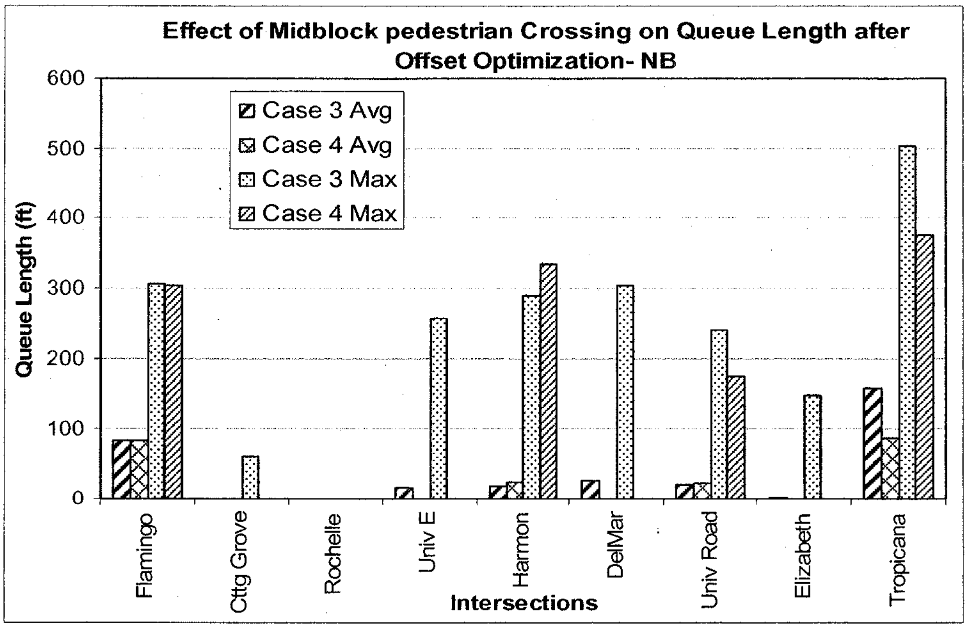


Figure 5-14 Queue length on Maryland Parkway NB for Case 3 and Case 4

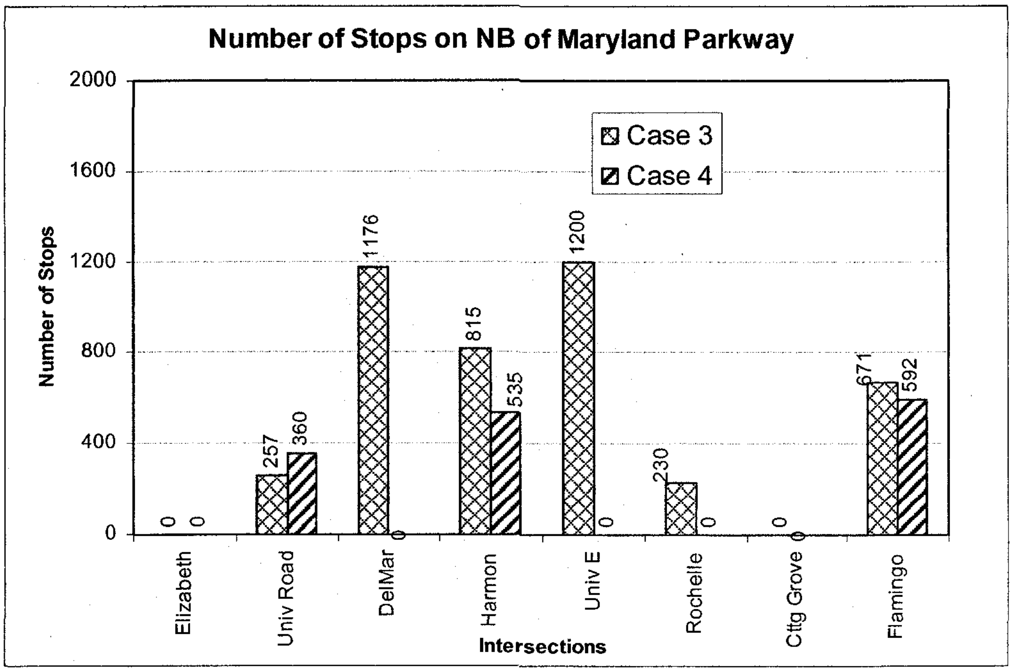
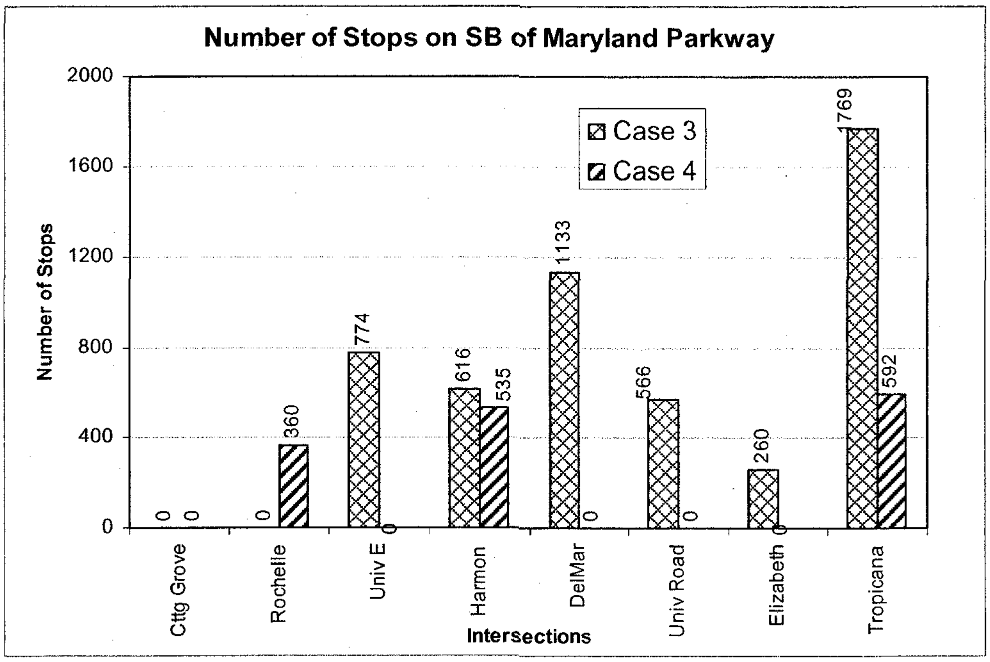


Figure 5-15 Number of stops on Maryland Parkway NB for Case 3 and Case 4



Key: Case 3 – Model with Midblock Pedestrian Crossing using optimized offsets
Case 4 – Model without Midblock Pedestrian Crossing using optimized offsets

Figure 5-16 Number of Stops on Maryland Parkway SB for Case 3 and Case 4

On above graphs, it was clearly shown the model with midblock pedestrian crossing has the higher travel times, delays, queue lengths and number of stops than the model without midblock pedestrian crossing.

The model was built for AM Peak Traffic and PM Traffic with AM traffic volumes and PM traffic volumes. The offsets and splits were also calculated for both AM and PM networks. But due to unavailability of pedestrian flow data, the same modeled pseudo signal for mid day traffic, representing the midblock pedestrian crossing was used in AM and PM traffic networks.

Table 5-6 MOEs of Entire Network of Case 3 and Case 4 for NB and SB

		Northbound				Southbound			
		Case 3	Case 4	Diff.	% Diff	Case 3	Case 4	Diff	% Diff
Mid Day	Travel Time (v-h)	68	54	14	25.9	67	50	17	34.0
	Total Delay(v-h)	27	14	13	92.9	31	16	15	93.8
	Stopped Delay(v-h)	14	8	6	75.0	19	9	10	111.1
	Number of Stops	4,388	1,581	2,807	177.5	5,118	1,581	3,537	223.7
PM	Travel Time(v-h)	76	55	21	38.2	135	68	67	98.5
	Total Delay(v-h)	34	15	19	126.7	90	25	65	260.0
	Stopped Delay(v-h)	18	8	10	125.0	50	15	35	233.3
	Number of Stops	5,797	1,690	4,107	243.0	31,633	3,646	27,987	767.6
AM	Travel Time(v-h)	40	32	8	25.0	37	32	5	15.6
	Total Delay(v-h)	19	7	12	171.4	19	9	10	111.1
	Stopped Delay(v-h)	10	4	6	150.0	12	6	6	100.0
	Number of Stops	3,112	789	2,323	294.4	2,818	969	1,849	190.8

The travel time, total delay, stopped delay and number of stops for entire network from Tropicana downstream to Flamingo downstream in northbound direction and flamingo downstream to Tropicana downstream in southbound direction were tabulated in Table 5-6. The following observations were made:

- Ignoring the pedestrians at midblock crossing and designing the arterial would increase the total delay by 92.9% for NB and 93.8% for southbound during mid day period.
- The number of stops has an adverse impact on arterial traffic if we ignore the midblock pedestrian crossing. The percentage increase in number of stops was more than 100%.
- Mid day pedestrian data flow was used for AM and PM networks which would overestimate the delays and stops at midblock pedestrian crossing locations because of overestimated pedestrian flow data.
- In AM and PM networks also, there is significant increase in MOEs if pedestrian midblock crossing was considered.

5.1.3 Statistical Analysis

The effect of midblock pedestrian crossing was tested statistically by paired one sample t-test. Since both the networks were same and the only difference is midblock pedestrian crossing, the paired t-test was done. Case 1 is the model without midblock pedestrian crossing and Case 2 is the model with midblock pedestrian crossing. If the difference in mean travel time of Case 2 and Case 1 was greater than zero then it was implied that mean travel time of model with midblock pedestrian crossing (Case 2) is more than the mean travel time of model without midblock pedestrian crossing (Case 1). The Hypothesis testing was done by considering,

Null Hypothesis, H_0: $\mu_1 - \mu_2 = 0$

Alternate Hypothesis, H_1: $\mu_1 - \mu_2 > 0$

Paired T-Test and CI

```
             N    Mean    StDev  SE Mean
Difference  50  0.3300  0.3200   0.0453

95% lower bound for mean difference: 0.2541
T-Test of mean difference = 0 (vs > 0): T-Value = 7.29  P-Value = 0.000
```

The sample test result from the Minitab 15 was shown above. Alternate Hypothesis was accepted, if the P-Value was less than 0.05 and it indicates that, the mean travel time of Case 1 is greater than the mean travel time of Case 2. Alternate Hypothesis was rejected, if the P-Value was greater than 0.05 for 95% lower bound mean difference and it indicated that the mean travel time of Case 2 is equal to mean travel time of Case 1.

The Same Statistical test was done to see the difference between travel time of Case 3 and Case 4. Case 3 is the model with optimized offsets and splits and without midblock pedestrian crossing. Case 5 is the model with optimized offsets and splits and with midblock pedestrian crossing.

The statistical results were shown in Table 5-12. Except Del Mar-Harmon segment in Northbound, alternate hypothesis has been accepted for all other segments in both northbound and southbound directions. The mean travel time of segments of Case 2 is greater than that of Case 1.

Table 5-7 Statistical Results for Effect of Midblock Pedestrian Crossings

	Existing offsets and splits (Case 1 and Case 2)			Optimized offsets and splits (Case 3 and Case 4)		
	Difference in Mean TT	P-Value	Statistical Results	Difference in Mean TT	P-Value	Statistical Results
Northbound						
Tropicana - Elizabeth	0.33	0.000	Accept H_1	0.25	0.000	Accept H_1
Elizabeth – University Road	0.43	0.000	Accept H_1	-0.25	0.999	*Reject H_1*
University Road – Del Mar	10.98	0.000	Accept H_1	10.87	0.000	Accept H_1
Del Mar - Harmon	-0.41	0.923	*Reject H_1*	2.16	0.000	Accept H_1
Harmon – University East	10.13	0.000	Accept H_1	9.77	0.000	Accept H_1
University East - Rochelle	1.83	0.000	Accept H_1	1.89	0.000	Accept H_1
Rochelle – Cottage Grove	0.12	0.000	Accept H_1	0.12	0.000	Accept H_1
Cottage Grove – Flamingo	1.31	0.000	Accept H_1	5.70	0.000	Accept H_1
South Bound						
Flamingo – Cottage Grove	0.25	0.000	Accept H_1	0.20	0.000	Accept H_1
Cottage Grove - Rochelle	0.06	0.000	Accept H_1	-0.01	0.557	*Reject H_1*
Rochelle – University East	7.81	0.000	Accept H_1	8.23	0.000	Accept H_1
University East - Harmon	1.31	0.000	Accept H_1	0.32	0.000	Accept H_1
Harmon – Del Mar	11.54	0.000	Accept H_1	12.19	0.000	Accept H_1
Del Mar – University Road	1.75	0.000	Accept H_1	1.87	0.000	Accept H_1
University Road - Elizabeth	9.61	0.000	Accept H_1	2.23	0.000	Accept H_1
Elizabeth - Tropicana	24.81	0.000	Accept H_1	19.92	0.000	Accept H_1

After optimizing offsets and splits, Elizabeth-University Road in Northbound and Cottage Grove-Rochelle in southbound, the alternate hypothesis was reject which implies there is no significant difference in travel time of Case 3 and Case 5. All other segments have significant difference in mean travel time of Case 3 and Case 5. Hence statistically, there was significant effect of midblock pedestrian crossing on arterial traffic.

5.2 Evaluation of Arterial Traffic with Optimized Network

To see the effect of optimization of offsets and splits, Case 3, Case 5 and Case 6 of mid day network were compared. Case 3, Case 5 and Case 6 have midblock pedestrian crossing which was modeled as pseudo signal. Case 3 was the network which was not accounted for pedestrians during optimization in SYNCHRO. In real life, the engineers design the arterial with midblock pedestrian crossing as in Case 3. In order to account for pedestrians during optimization of offsets, the midblock pedestrian crossing was modeled as actuated-uncoordinated signal. This optimized network was considered as Case 5. Case 5 MOEs were expected to be less than Case 3 since the pedestrians were taken into account while optimizing the offsets. Case 6 was the network with actual actuated signals at midblock pedestrian crossing locations and offsets and splits were optimized. MOEs for Case 3 and Case 5 were tabulated in Table 5-5 and Table 5-6

Table 5-8 Travel Time, Delay and # of Stops on Maryland NB for Case 3 and Case 5

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	3	5	3	5	3	5	3	5
Tropicana - Elizabeth	12.0	12.0	1.6	1.6	0.1	0.1	40	0
Elizabeth – University Road	19.7	19.5	4.2	4.0	0.9	0.8	263	239
University Road – Del Mar	23.7	24.1	11.8	12.2	5.1	5.5	1200	1202
Del Mar - Harmon	25.7	31.2	12.3	17.6	7.2	10.0	832	1591
Harmon – University East	23.5	23.7	10.6	10.8	4.6	4.5	1225	1428
University East - Rochelle	15.7	16.3	2.9	3.6	0.3	0.3	234	348
Rochelle – Cottage Grove	4.4	4.4	0.4	0.4	0	0.0	0	0
Cottage Grove – Flamingo	56.0	65.8	32.4	42.1	24.6	33.4	684	809

Table 5-9 Travel Time, Delay and # of Stops on Maryland SB for Case 3 and Case 5

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	3	5	3	5	3	5	3	5
Flamingo – Cottage Grove	24.0	24.0	2.4	2.4	0.0	0.0	0	0
Cottage Grove - Rochelle	4.2	4.2	0.4	0.4	0.0	0.0	0	0
Rochelle – University East	22.2	21.9	9.5	9.2	4.1	3.9	790	741
University East - Harmon	23.6	33.3	10.1	19.4	5.1	12.3	629	1184
Harmon – Del Mar	26.9	27.1	13.4	13.7	6.4	6.2	1157	1356
Del Mar – University Road	18.0	22.8	6.7	11.4	3.2	5.9	578	1378
University Road - Elizabeth	19.7	20.3	4.0	4.6	0.7	0.7	265	310
Elizabeth - Tropicana	72.9	79.6	60.3	66.8	48.3	53.5	1803	2090

- The travel time, delays and number of stops in Case 5 is higher than that of Case 3 which was not expected.

- Both in the northbound and the southbound directions of travel, all segments from the first crosswalk location, has higher travel time, delays and number of stops.
- In the northbound direction, Del Mar-Harmon, Cottage Grove-Flamingo has higher difference when compared to other travel time segments.
- In the southbound direction, University East-Harmon, Del Mar-University Road and Elizabeth-Tropicana have higher travel time than other segments of the network.
- Offsets might not be well coordinated at downstream of the crosswalks of Case 5 in both the directions.

The MOEs for Maryland Parkway arterial was shown in the Table 5-7 and 5-8 for Case 3 and Case 6 and in the Table 5-10 and 5-11 for Case 5 and Case 6.

Table 5-10 Travel Time, Delay and # of Stops on Maryland NB for Case 3 and Case 6

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	3	6	3	6	3	6	3	6
Tropicana - Elizabeth	12.0	12.0	1.6	1.6	0.1	0.1	40	43
Elizabeth – University Road	19.7	19.4	4.2	3.9	0.9	0.9	263	219
University Road – Del Mar	23.7	23.5	11.8	11.6	5.1	4.7	1200	1241
Del Mar - Harmon	25.7	25.4	12.3	11.8	7.2	6.7	832	870
Harmon – University East	23.5	22.1	10.6	9.2	4.6	4.2	1225	1061
University East - Rochelle	15.7	15.1	2.9	2.4	0.3	0.2	234	143
Rochelle – Cottage Grove	4.4	4.4	0.4	0.4	0	0.0	0	0
Cottage Grove – Flamingo	56.0	55.5	32.4	31.9	24.6	23.7	684	700

Table 5-11 Travel Time, Delay and # of Stops on Maryland SB for Case 3 and Case 6

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	3	6	3	6	3	6	3	6
Flamingo – Cottage Grove	24.0	24.1	2.4	2.4	0.0	0.0	0	0
Cottage Grove - Rochelle	4.2	4.2	0.4	0.4	0.0	0.0	0	0
Rochelle – University East	22.2	18.5	9.5	5.7	4.1	2.8	790	402
University East - Harmon	23.6	21.9	10.1	8.4	5.1	4.6	629	411
Harmon – Del Mar	26.9	22.3	13.4	8.9	6.4	3.5	1157	791
Del Mar – University Road	18.0	19.2	6.7	7.7	3.2	4.1	578	688
University Road - Elizabeth	19.7	19.0	4.0	3.2	0.7	0.3	265	137
Elizabeth - Tropicana	72.9	66.2	60.3	52.8	48.3	41.3	1803	1612

Table 5-12 Travel Time, Delay and # of Stops on Maryland NB for Case 5 and Case 6

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	5	6	5	6	5	6	5	6
Tropicana - Elizabeth	12.0	12.0	1.6	1.6	0.0	0.1	0	43
Elizabeth – University Road	19.5	19.4	4.0	3.9	0.2	0.9	239	219
University Road – Del Mar	24.1	23.5	12.2	11.6	0.8	4.7	1202	1241
Del Mar - Harmon	31.2	25.4	17.6	11.8	1.1	6.7	1591	870
Harmon – University East	23.7	22.1	10.8	9.2	0.8	4.2	1428	1061
University East - Rochelle	16.3	15.1	3.6	2.4	0.2	0.2	348	143
Rochelle – Cottage Grove	4.4	4.4	0.4	0.4	0.0	0.0	0	0
Cottage Grove – Flamingo	65.8	55.5	42.1	31.9	0.8	23.7	809	700

Table 5-13 Travel Time, Delay and # of Stops on Maryland SB for Case 5 and Case 6

Segment	Travel Time (sec)		Total Delay (veh/sec)		Stopped Delay (veh/sec)		Number of Stops	
Case	5	6	5	6	5	6	5	6
Flamingo – Cottage Grove	24.0	24.1	2.4	2.4	0.0	0.0	0	0
Cottage Grove - Rochelle	4.2	4.2	0.4	0.4	0.0	0.0	239	0
Rochelle – University East	21.9	18.5	9.2	5.7	0.5	2.8	1202	402
University East - Harmon	33.3	21.9	19.4	8.4	0.9	4.6	1591	411
Harmon – Del Mar	27.1	22.3	13.7	8.9	1.0	3.5	1428	791
Del Mar – University Road	22.8	19.2	11.4	7.7	0.9	4.1	348	688
University Road - Elizabeth	20.3	19.0	4.6	3.2	0.2	0.3	0	137
Elizabeth - Tropicana	79.6	66.2	66.8	52.8	2.4	41.3	809	1612

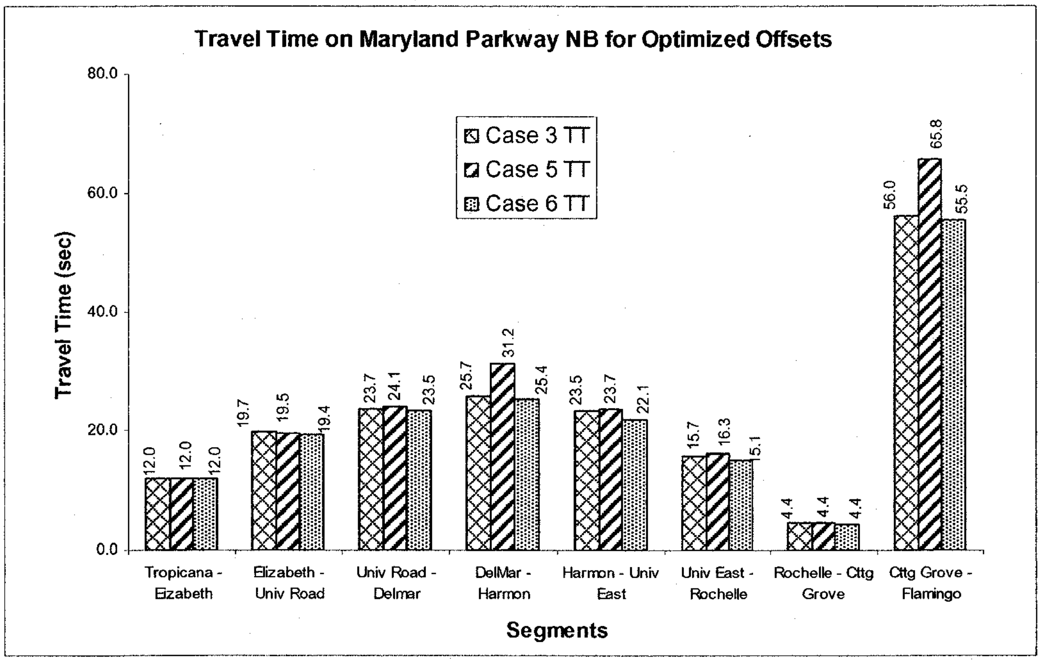


Figure 5-17 Travel Time on Northbound Direction for Case 3, Case 5 and Case 6.

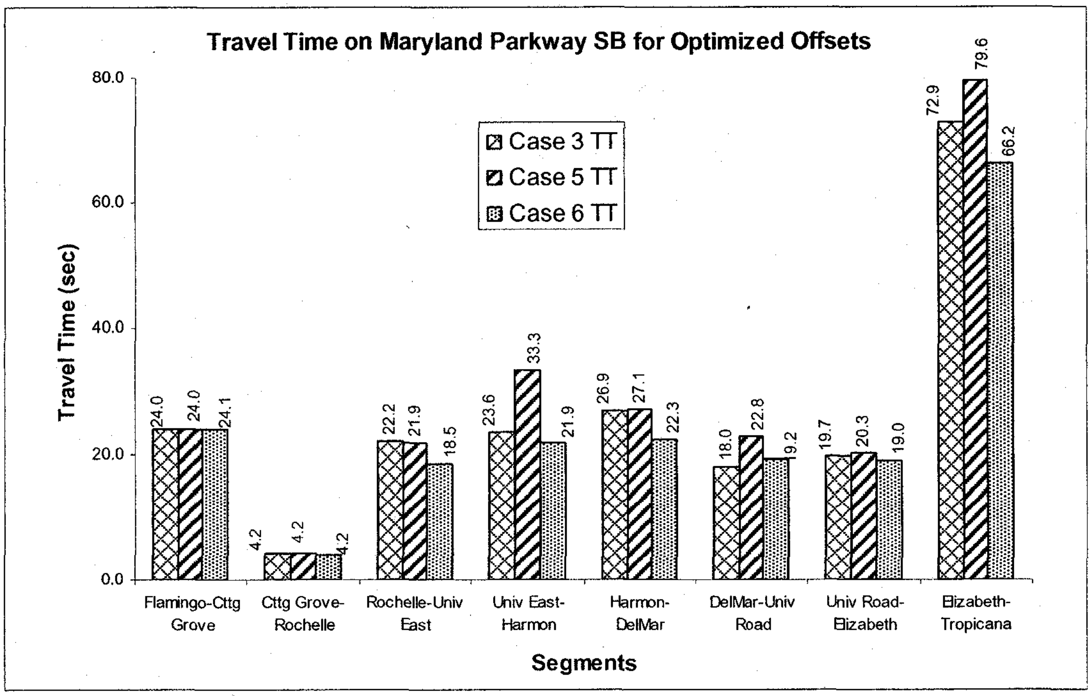


Figure 5-18 Travel Time on Southbound Direction for Case 3, Case 5 and Case 6

The travel time, delays and number of stops were compared for Case 3, Case 5 and Case 6. For Case 6, actual signals were produced better results for all segments than Case 3 and Case 5. Del Mar-Harmon and Cottage Grove-Flamingo travel times of northbound direction in Case 5 were higher than Case 3 and Case 6 which was unexpected results. In other segments also Case 3 and Case 5 travel times were nearly equal. But in all segments, Case 6 has lower travel times than Case 2 and Case 5. This implies that the actual signals, actuated-coordinated at midblock pedestrian crossing (reflecting pedestrian activities) will yield better results than the Case 5 Conditions. In southbound direction also, Case3 travel times and Case 5 travel times are nearly equal except University East-Harmon and Del Mar-University Road. The comparison of delays, queue length and number of stops were shown in graphs for Case 3, Case 5 and Case 6.

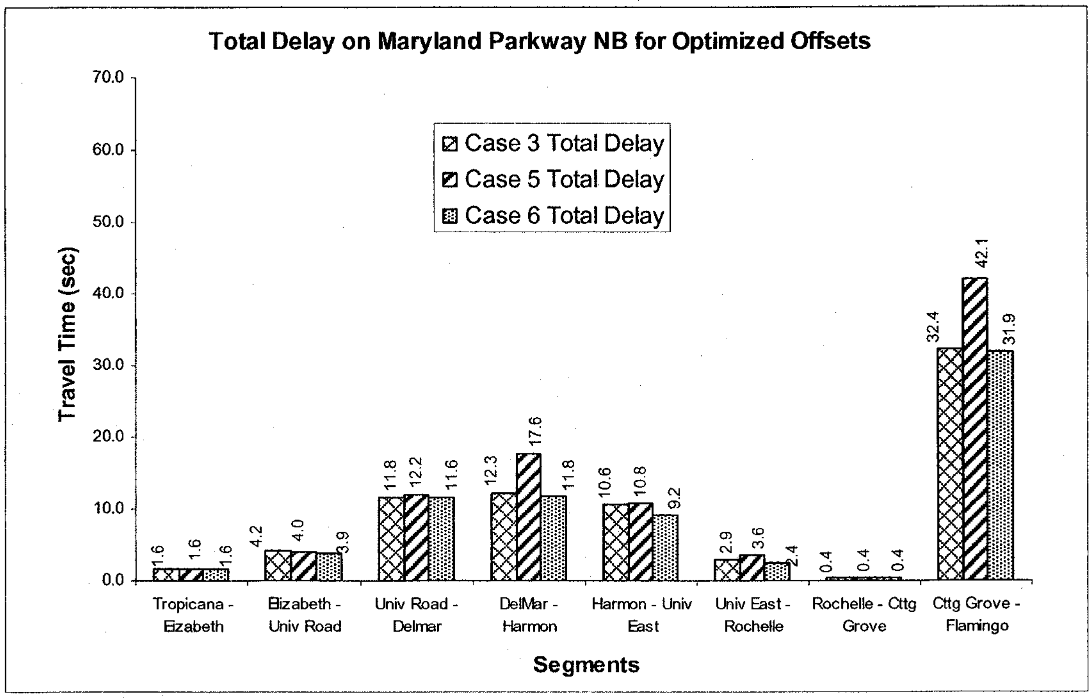


Figure 5-19 Total Delay on Northbound Direction for Case 3, Case 5 and Case 6

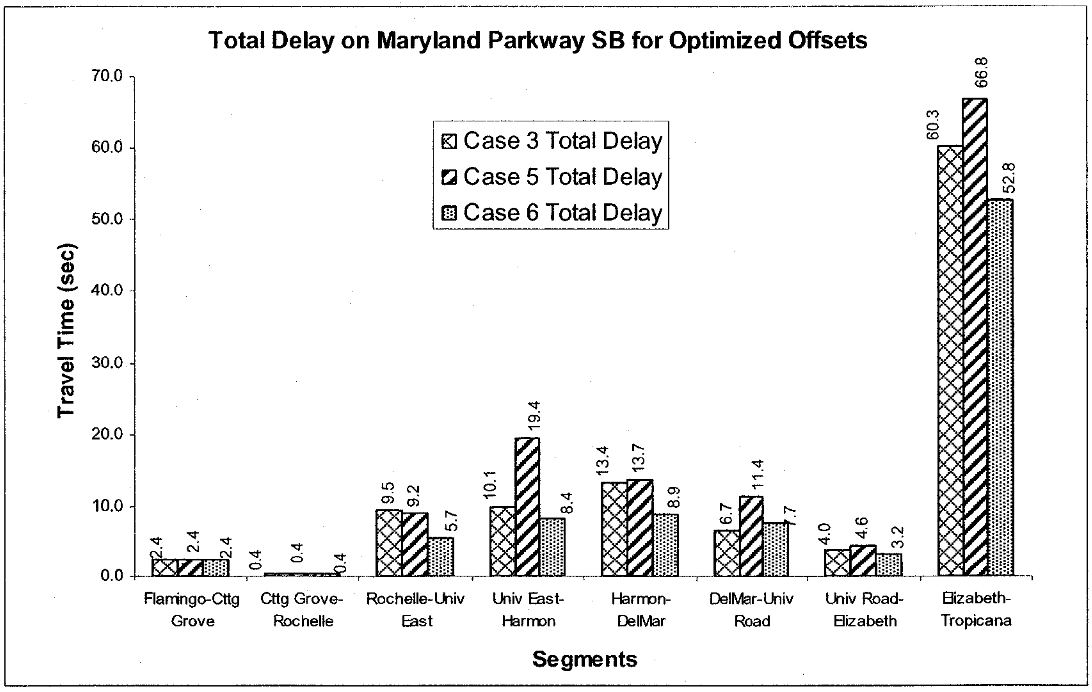


Figure 5-20 Total Delay on Southbound for Case 3, Case 5 and Case 6

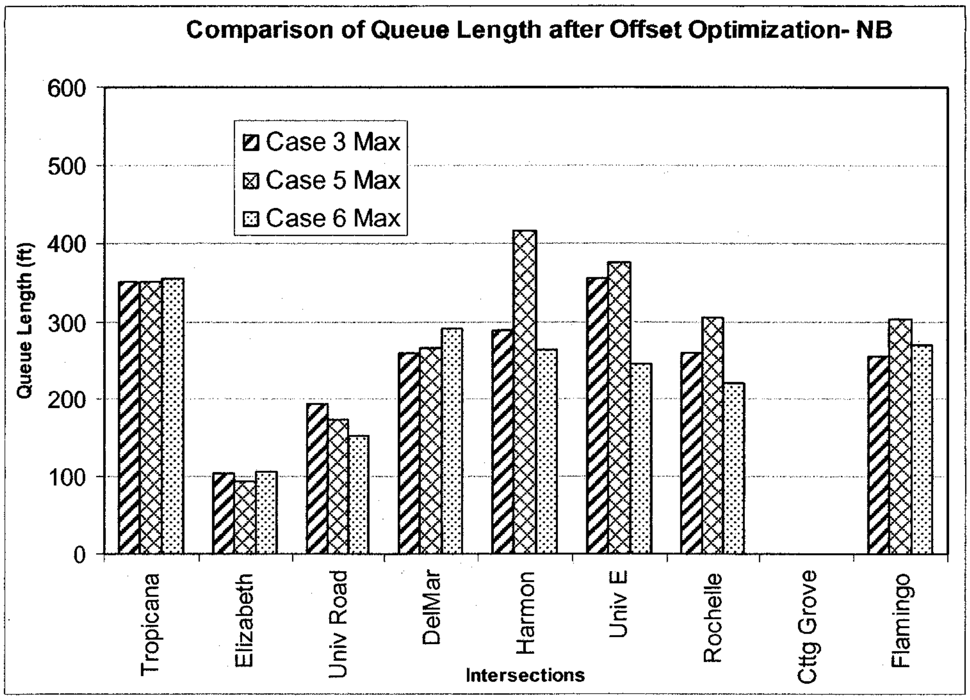


Figure 5-21 Maximum Queue Length on Northbound for Case 3, Case 5 and Case 6

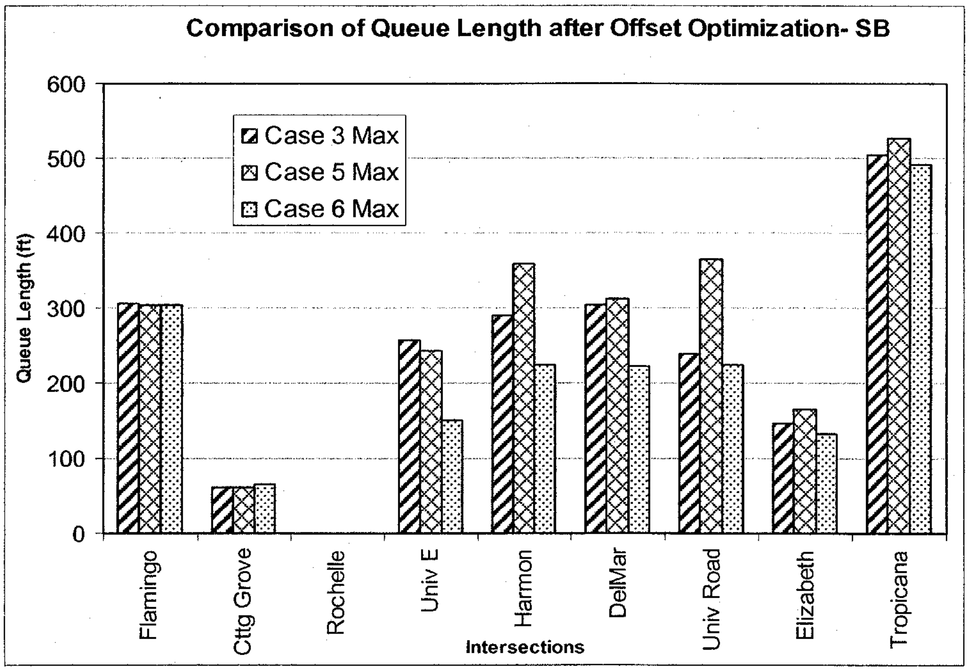


Figure 5-22 Maximum Queue length on Southbound for Case 3, Case 5 and Case 6

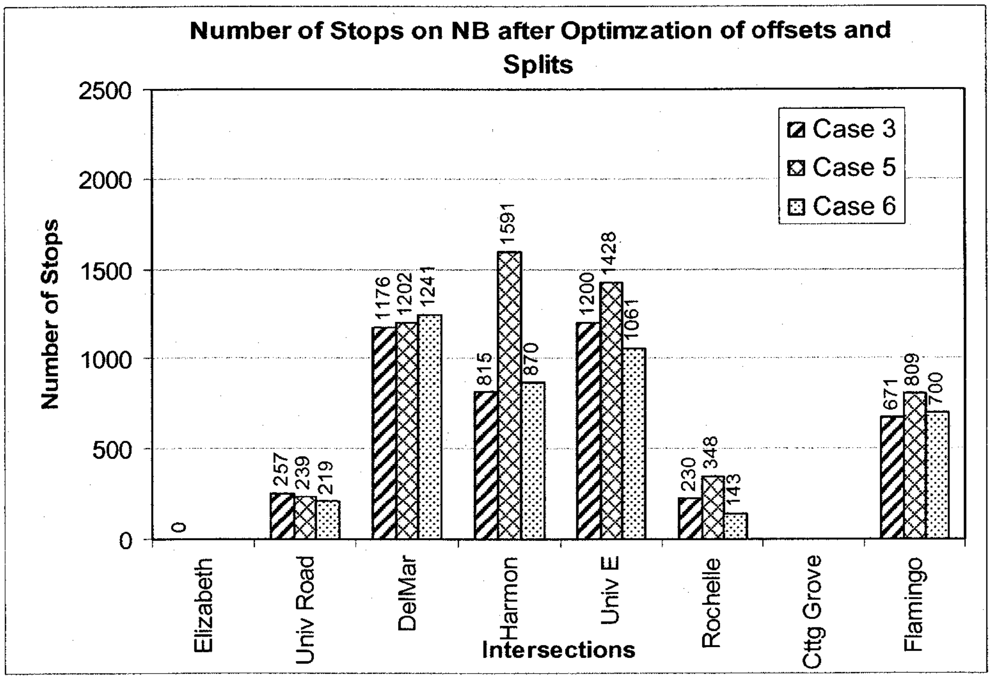


Figure 5-23 Number of stops on Maryland Parkway NB for Case 3, Case 5 and Case 6

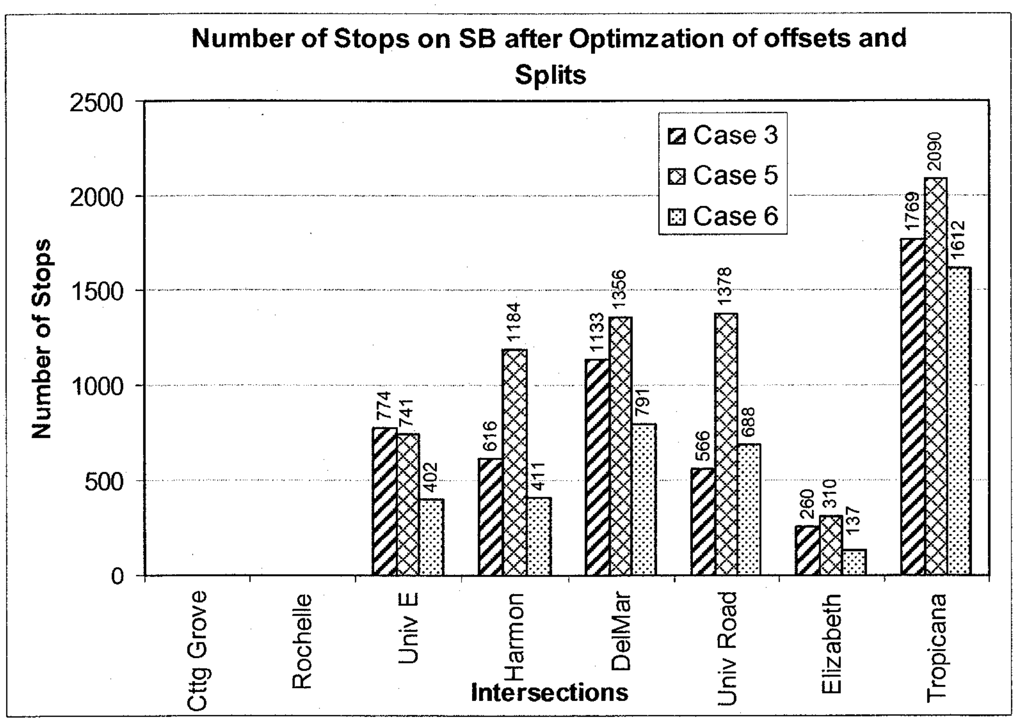


Figure 5-24 Number of Stops on Maryland Parkway SB for Case 3, Case 5 and Case 6

Table 5-14 MOEs of Arterial Network for Case 3, Case 5 and Case 6 on NB

		Case 3	Case 5	Case 6	% Difference	% Difference	% Difference
					Case 5 - 3	Case 6 - 3	Case 6 - 5
Mid Day	Travel Time (v-h)	68	74	67	8.8	-1.5	-9.5
	Total Delay(v-h)	27	33	26	22.2	-3.8	-21.2
	Stopped Delay(v-h)	14	18	14	28.5	0	-22.2
	Number of Stops	4,388	5,658	4,276	28.9	-2.6	-24.1
PM	Travel Time (v-h)	76	81	71	6.5	-6.6	-12.3
	Total Delay(v-h)	34	39	29	14.7	-14.7	-25.6
	Stopped Delay(v-h)	18	22	15	22.2	-16.7	-31.8
	Number of Stops	5,797	6,559	5,093	13.1	-12.1	-22.4
AM	Travel Time (v-h)	40	44	37	10.0	-7.5	-15.9
	Total Delay(v-h)	12	19	14	58.3	-26.3	16.7
	Stopped Delay(v-h)	6	10	7	66.7	-30.0	16.7
	Number of Stops	1,879	3,112	2,380	65.6	-23.5	26.7

The entire networks MOEs for Midday, AM and PM traffic were tabulated in Table 5-14 and 5-15 for NB and SB directions respectively and the following observations were made:

- Case 5 has higher travel times, delays and number of stops than Case 3 which was unexpected during midday and PM.
- For the AM Traffic, Case 5 travel time is higher than the Case 3.

- When Case 6 was compared with Case 3 and Case 5, the travel times, delays and queue lengths were reduced significantly.
- This can be occurred because of good coordination of signalized intersections of the network.
- The southbound direction also had the same trend was as in the northbound direction.

Table 5-15 MOEs of Arterial Network for Case 3, Case 5 and Case 6 on SB

		Case 3	Case 5	Case 6	% Difference	% Difference	% Difference
					Case 5 - 3	Case 6 - 3	Case 6 - 5
Mid Day	Travel Time (v-h)	67	74	62	10.4	-8.9	-8.1
	Total Delay(v-h)	31	39	26	25.8	-16.1	-33.3
	Stopped Delay(v-h)	19	24	16	26.3	-15.7	-33.3
	Number of Stops	5,118	7,060	4,039	37.9	-21.1	-42.8
PM	Travel Time (v-h)	135	142	129	5.1	-4.4	-9.1
	Total Delay(v-h)	90	97	85	7.8	-5.5	-12.3
	Stopped Delay(v-h)	50	54	47	8.0	-16.0	-13.0
	Number of Stops	31,633	33,681	30,734	6.5	-2.9	-8.7
AM	Travel Time (v-h)	37	41	35	10.8	-5.4	-14.6
	Total Delay(v-h)	13	19	15	46.1	-21.1	-15.3
	Stopped Delay(v-h)	8	12	9	50.0	-25.0	-12.5
	Number of Stops	1,791	2,818	2,145	57.3	-23.8	-19.7

5.3 Summary of Findings

From the above analysis, the summary of findings is as follows:

- When Case 1 and Case 2 was compared to evaluate the effect of midblock crossing on arterial performance with existing offsets and splits, the link travel time, delays and stops were high for Case 1 than Case 2. It shows there is effect of midblock pedestrian crossings on arterial performance.
- When Case 3 was compared with Case 4 to evaluate the effect of midblock crossing on arterial performance with optimized offsets and splits by ignoring pedestrians, the measure(s) of effectiveness link travel time, delays and stops were high for Case 3 than Case 4.
- When Case 3 was compared with Case 5 to evaluate the optimal design of offsets and splits with considering pedestrians, the measures of effectiveness of Case 3 was better than that of Case 5.
- When Case 3 and Case 5 was compared with Case 6 to evaluate the optimal design of offsets and splits, the measures of effectiveness of Case 6 was better than that of Case 3 and Case 5.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Effect of Midblock Pedestrian Crossing on Arterial Traffic

There was significant impact on arterial traffic due to midblock pedestrian crossing. There was large difference of travel times, total delays, queue length, number of stops at midblock pedestrian crossing segments in the network. Other segments also has the impact but not as equal to midblock pedestrian crossing segments. Overall, when it comes to entire arterial traffic in northbound and southbound direction, there is large difference in MOEs between with and without midblock pedestrian crossing networks. This study demonstrated the potential effects if midblock pedestrian crossing was ignored in design and operational analysis.

6.2 Pseudo Signal Design and Network Calibration

In Summary, the methodology that was developed for designing pseudo signal has functioned well. The designed pseudo signal represented the pedestrian activities at midblock pedestrian crossing. The randomness of arriving and crossing pedestrians was reflected by modeling the pseudo signal as an actuated-uncoordinated signal with vehicles entering the minor streets at crosswalk locations. However, there are a few areas which could improve the methodology of calibration. It is essential that selecting the measure(s) of effectiveness commence before data is collected. In this study, only the travel time was used as measure(s) of effectiveness. The delay and queue length data collection at pedestrian crosswalk locations would have been better measure(s) of effectiveness calibration. The process used for calibration must be performed in the exact sequence as outlined in the Literature Review (Chapter 2). Due to eighteen segments in the network, the calibration procedure was simplified.

The simulation model did not take into consideration erratic actions of drivers or pedestrians. Conflicts between vehicle and pedestrian movements were not considered in this model, since micro simulations are not intended to simulate these events. Since the purpose of this study was to develop the methodology to model pseudo signal, to find the effect of midblock pedestrian crossing on arterial traffic, to optimize the network for better signal coordination, the calibration of network was done.

6.3 Optimized Offsets for Signal Coordination

When optimized in SYNCHRO, better signal coordination was expected for actuated uncoordinated signal (as midblock pedestrian crossing). But the results from VISSIM explicitly shown the MOEs of actuated-uncoordinated optimized offset model was higher than the MOEs of without pedestrian crossing optimized offset model (with midblock pedestrian crossing). In other words, Case 5 MOEs was higher than the Case 3 MOEs which implies there will be more delays, number of stops and queue length if the pedestrians were taken into account while optimizing the offsets. Travel time, delays, queue length and number of stops was reduced when actuated coordinated signals considered at midblock pedestrian crossing locations. With the help of this study, actuated coordinated signal can be recommended at midblock pedestrian crossing. But the distance between the signalized intersections will become less in our study area if actuated signals were recommended.

6.4 Recommendations

The research conducted in this thesis presented numerous additional research topics. Studies confirming the methodology for constructing the pseudo signal design by testing at other case study location would be one obvious direction for future research. With different traffic software, comparison can be done by using pseudo signal design for midblock pedestrian crossing. Mathematical model can be developed to optimize the offsets with actuated uncoordinated signal in between the signalized intersections because in this study SYNCHRO was used and the pseudo signal vehicles was also taken into account while offset optimization.

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APPENDIX I

DATA COLLECTION SHEETS

License Plate Survey

Name of the Surveyor: City:
Intersection: Date:
Direction: Time:

Time	License Plate Numbers	Time	License Plate numbers	Time	License Plate Numbers

Sample of License Plate Study Survey Sheet

Pedestrian Travel Time Study

Name of Observer:Nitin
Site:Maryland-University E
Direction:Southbound

City: Las Vegas
Date:11/09/2006
Time:11:00-11:45 AM

# of Peds	Peds Arrival Time	Start Time	End Time	Start Time-2	End Time-2	Initial Delay	Delay-2	Total Delay	Peds Crossing Time
	0:00:00								
	0:00:00								
	0:00:00								
	0:00:00								
	0:00:00								
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Sample Survey Sheet of Pedestrian Crossing Time at Midblock Crosswalk

Vehicle Yielding Study at Pedestrian Crosswalk

Location ______________________ Day/Date ______________________

Arterial direction ______________________ Start time ______________________

Observer ______________________

	Ped arrival Time	No. veh arriving within 3 secs	No. veh passing before 1st stop	No veh passing in front of ped in xwalk	No. of peds in xwalk
Occurrence 1					
Occurrence 2					
Occurrence 3					
Occurrence 4					
Occurrence 5					
Occurrence 6					
Occurrence 7					
Occurrence 8					
Occurrence 9					
Occurrence 10					
Occurrence 11					
Occurrence 12					
Occurrence 13					
Occurrence 14					
Occurrence 15					
Occurrence 16					
Occurrence 17					
Occurrence 18					
Occurrence 19					
Occurrence 20					
Occurrence 21					
Occurrence 22					
Occurrence 23					
Occurrence 24					
Occurrence 25					
Occurrence 26					

Sample Survey Sheet of Crossing Occurrences of Pedestrians at Crosswalk

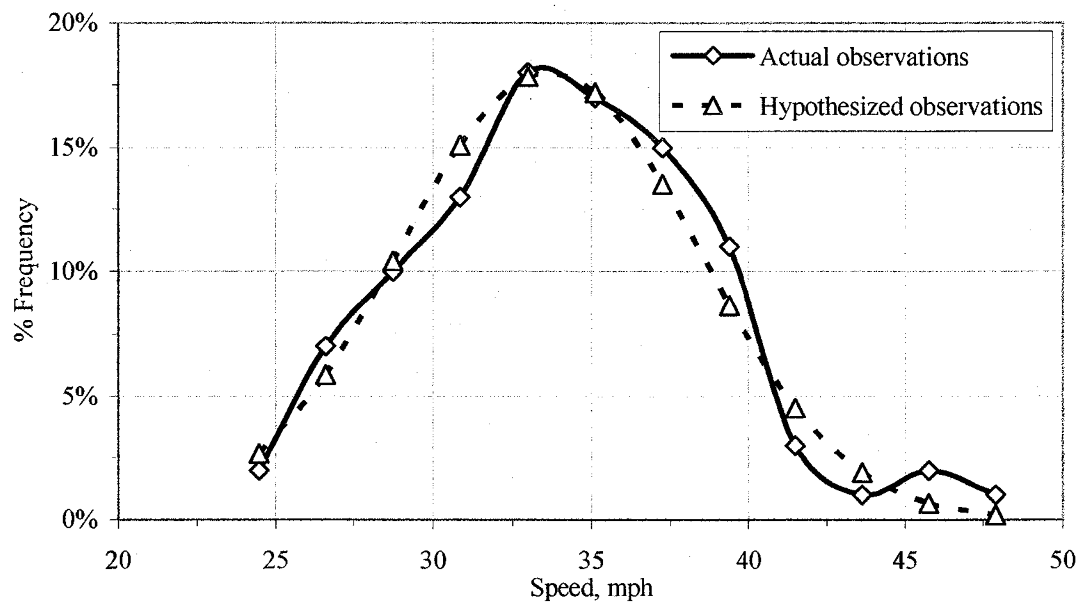
APPENDIX II

SPEED DISTRIBUTION DATA

Speed group, lower (mph)	Speed group, upper (mph)	Speed group, lower (mph)	Speed group, upper (mph)	Median of range (mph)	Number of observed vehicles	Percent frequency in group	Cumulative percent frequency
		(adjusted for α = 20°)	(adjusted for α = 20°)	S	n_j	$\%_j = n_j/N$	$\%_{j-1} + (n_j/N)$
22	24	23.41	25.54	24.48	2	2.0%	2.0%
24	26	25.54	27.67	26.60	7	7.0%	9.0%
26	28	27.67	29.80	28.73	10	10.0%	19.0%
28	30	29.80	31.93	30.86	13	13.0%	32.0%
30	32	31.93	34.05	32.99	18	18.0%	50.0%
32	34	34.05	36.18	35.12	17	17.0%	67.0%
34	36	36.18	38.31	37.25	15	15.0%	82.0%
36	38	38.31	40.44	39.37	11	11.0%	93.0%
38	40	40.44	42.57	41.50	3	3.0%	96.0%
40	42	42.57	44.70	43.63	1	1.0%	97.0%
42	44	44.70	46.82	45.76	2	2.0%	99.0%
44	46	46.82	48.95	47.89	1	1.0%	100.0%

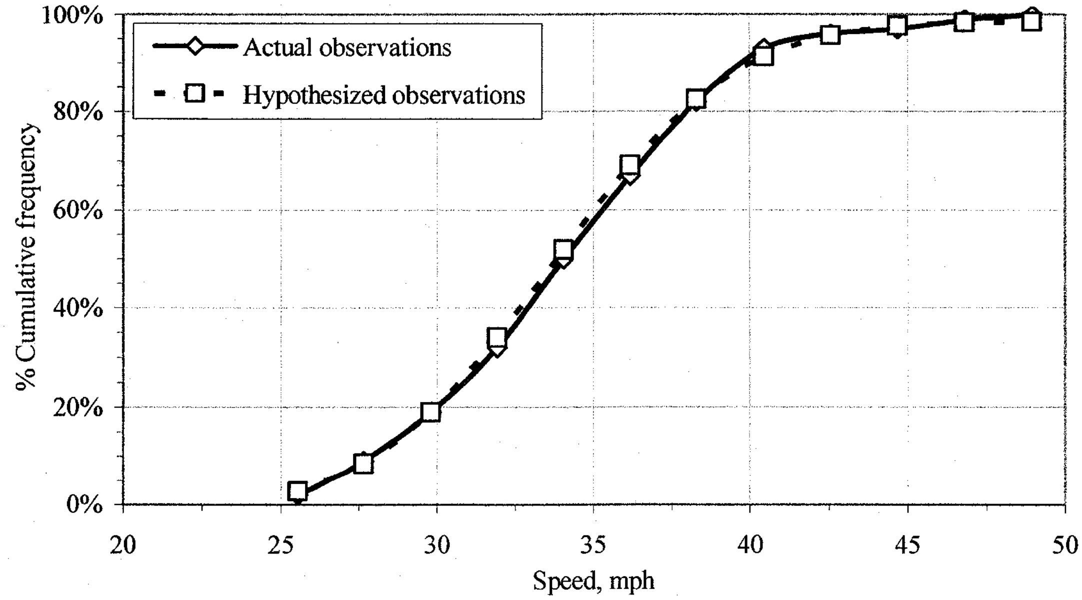
Summary of free flow speed distribution table

Source: John Merrill (2005)



Frequency distributions for measured and hypothesized free flow speeds

Source: John Merrill (2005)



Cumulative frequency distributions for measured and hypothesized free flow speeds

Source: John Merrill (2005)

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