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Vissim model development of a mid-block pedestrian crossing between signalized intersections

John Christopher Merrill
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VISSIM MODEL DEVELOPMENT OF A MID-BLOCK PEDESTRIAN CROSSING
BETWEEN SIGNALIZED INTERSECTIONS

by

John Christopher Merrill

Bachelor of Science in Engineering
Western Michigan University
2001

A thesis submitted in partial fulfillment
of the requirements for the

**Master of Science in Engineering Degree in Civil Engineering
Department of Civil and Environmental Engineering
Howard R. Hughes College of Engineering**

**Graduate College
University of Nevada, Las Vegas
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Master of Science in Engineering

Examination Committee Chair

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ABSTRACT

VISSIM model development of a mid-block pedestrian crossing between signalized intersections

by

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Dr. Mohamed S. Kaseko, Examination Committee Chair
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Pedestrian mid-block crossings through arterial roadways have significant impacts on vehicular level of service and safety. Past research has been conducted to simulate these crossings. However, a practical, realistic model and methodology have not been established for analysis and planning of mid-block pedestrian crossings. The objective of this study was to create a practical traffic microsimulation methodology for mid-block pedestrian crossings through a case study using a versatile transportation software package. VISSIM v3.70 was selected because of its ability to model individual vehicle and pedestrian interactions. The developed methodology provided a framework for model development at any pedestrian mid-block crossing through a case study. Input parameters for the case study model included an innovative field study that was designed using a Geometric distribution to estimate the proportion of drivers yielding to pedestrians at the crosswalk. Segment travel times and crosswalk approach vehicle queue lengths were used for calibration and validation, respectively. Level of service analyses were performed using the final model.

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CHAPTER 1

INTRODUCTION

Pedestrian mid-block crossings through arterial roadways have significant impacts on arterial level of service and safety. A mid-block pedestrian crossing can produce added delay on a roadway segment. These delays can have adverse effects on traffic signal coordination for the corridor, since the effective bandwidth of the corridor would be effectively reduced.

Past research has been conducted to simulate these crossings (Kaseko & Karkee, 2005). However, a practical, realistic model and methodology has not been established for the development, calibration, validation, and analysis of mid-block pedestrian crossing simulation models. Pedestrian crossing models have been generally difficult to develop previously because the available software packages were incapable of modeling the complex interactions between automobile and pedestrian movements.

The objective of this thesis was to develop a computer simulation model for a mid-block pedestrian crossing between two signalized intersections, and calibrate this model based on field observations and findings from previous pedestrian and vehicular research. This model was then used to evaluate the level of service at the crosswalk and the impact of the crosswalk on the arterial level of service.

This proposed model was based on the VISSIM microsimulation model concept developed by Kaseko and Karkee (2005). VISSIM v3.70, developed by Planung

Transport Verkehr AG in Karlsruhe, Germany, had the necessary features to model intricate traffic operations on a microscopic level. Other microsimulation models such as CORSIM and SimTraffic do not have the ability to model individual yielding decisions per lane. Kaseko and Karkee's model simulated an isolated marked mid-block pedestrian crossing. However, key variables for the model were not calibrated because of the lack of sufficient field data.

The model methodology, which is based on the yielding parameters developed in Kaseko and Karkee (2005), commenced in three key phases. First, a study location was identified for the proposed simulation model. Second, construction of the model was done in VISSIM, taking into consideration the geometry of the corridor. Third, data was collected to accurately simulate conditions at the study location. Fourth, measure of effectiveness data was used to calibrate and verify the operation of the model with respect to the study location. Finally, the model was analyzed to confirm the results and to determine effects of certain conditions on the accuracy of the model.

It was anticipated that this study would contribute to the understanding of pedestrian and vehicular operations with respect to the transportation engineering profession. It would also provide a practical methodology to implement similar models at other mid-block crossings. The results of this thesis presented several other opportunities for continued research.

1.1 Study Background

Mid-block pedestrian crossings are quite common, especially in certain metropolitan areas such as Los Angeles, California and Las Vegas, Nevada. At least 20 mid-block

pedestrian crossing facilities exist in the Las Vegas metropolitan area. Mid-block pedestrian crossings with heavy pedestrian traffic can produce significant control delays for vehicles. Conversely, corridors with heavy vehicular volumes present particular delay and safety concerns for pedestrians who wish to use the mid-block crossing. When signalized intersections are far apart (at least one mile), 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.

Few research publications addressed procedures for developing traffic microsimulation software models. Published research regarding mid-block pedestrian crossing operations has also been limited. Research regarding pedestrian safety statistics at mid-block crossings had been addressed in detail, but studies pertaining to vehicular and pedestrian behaviors, interactions, and attitudes were scarce. There were no published studies or procedures found that specifically intended to collect the proportion of drivers who yield for pedestrians at mid-block crossings

Therefore, it was determined that there was a decisive need to better understand the traffic operations at mid-block pedestrian crossings. Understanding the operation of any transportation facility is critical to conducting planning, design, and optimizing operations on similar segments. One way that this need could be addressed was to construct a procedure for the development, calibration, and validation of mid-block pedestrian crossing microsimulation models. Since mid-block pedestrian crossings are almost always within one mile from the nearest signalized intersections, a microsimulation model methodology that incorporates nearby signalized intersections was considered to be particularly useful for planning and designing similar facilities.

1.2 Study Objective

The objective of this study was to develop a practical traffic microsimulation model and procedure for the evaluation of mid-block pedestrian crossings through a case study using a versatile transportation software package. VISSIM v3.70 was selected because of its flexibility to model the interaction between vehicles and pedestrians at unsignalized locations. The methodology used was intended to provide a framework for model development at any pedestrian mid-block crossing. Traditional measures of effectiveness were used to calibrate and validate the model. For illustrative purposes, level of service analyses for vehicles and pedestrians were included.

1.3 Working Hypotheses

- A transportation microsimulation software model and process can be created that adequately reproduces actual operations of an arterial roadway with a mid-block pedestrian crossing
- The yielding behavior of drivers and pedestrians can be estimated from field data and modeled in a microsimulation model
- Once developed, calibrated, and validated, a microsimulation model can be used to analyze operations based upon generated measures of effectiveness

1.4 Study Expectations

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. The proportion of drivers yielding to pedestrians was an

essential parameter to estimate and enter into the model. Therefore, the methodology for designing a yielding study, calculating the sample proportion, and verifying the estimated proportion statistically became necessary. The process used was expected to be applicable to any mid-block pedestrian crossing. It also provided insight as to how determining yielding proportions might be used in other types of studies. Furthermore, the final calibrated model was expected to be capable of generating usable outcomes for arterial traffic operational and level of service analyses, as well as planning purposes, such as determining the effect of possible design alternatives on future levels of service.

CHAPTER 2

LITERATURE REVIEW

Upon searching for relevant literature regarding similar endeavors to simulate mid-block pedestrian crossings, it was discovered that very little research had been published regarding microsimulation of pedestrian mid-block crossings. Most published research models related to the performance of individual software packages, with case studies involving high-speed arterials. However, research had been uncovered that relates indirectly to the development of this type of simulation model. It was necessary to establish an accepted procedure to develop, calibrate, and validate a microsimulation model. Furthermore, some data were more easily obtained through previous research than by conducting studies specifically for this project (such as pedestrian walking speed distributions). Finally, it was necessary to establish a behavioral basis for vehicle-pedestrian interactions at crosswalk locations.

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) created an isolated mid-block pedestrian crossing without signalized intersections on either end. The case study site was a six-lane

arterial with raised median (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). Much of the research presented in this thesis will be a continuation of the research done by Kaseko and Karkee (2005), but in much greater detail and scope.

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.

2.2 Simulation Model Calibration Methodology

Hourdakis, Michalopoulos, and Kottommannil (2003) published a methodology for simulation model calibration and validation using the AIMSUN software developed by Transport Simulation Systems as a case study. The objective of this study was to develop a systematic and practical methodology for calibrating microsimulation models, and determine the feasibility of using automated calibration methods through optimization techniques. It was suggested that traffic simulation models be calibrated in three distinct steps. The authors believed that simulation variables that relate to vehicles, such as acceleration/deceleration rates, and speeds, such as reduced speed areas, should be the main calibration variables. These two categories were then subdivided into global and

local settings. Global settings imply that the variable applied to the entire model, where local settings were specific to a certain portion of a model. The authors also used an optional third stage for calibration based upon measures of effectiveness. These measures of effectiveness should relate directly to the purpose of creating the model. The example given in the research was that ramp queues should be the calibration variable if the purpose of the model was to evaluate the effectiveness of ramp metering.

Hourdakis et al. (2003) used three types of statistical calculations to evaluate the fit between the simulated model and actual field conditions. The first statistic, Root Mean Square Error Percentage (*RMSP*), was a general goodness-of-fit statistic. The calculation of *RMSP* as reported in Hourdakis et al. (2003) is shown in Equation 2-1. The second statistic, the correlation coefficient, represented the degree of linear correlation between simulated and actual field measurements. The calculation of the correlation coefficient, r , is shown in Equation 2-2. The third statistic was based on work done by Theil (1961) regarding economic forecasting models (as cited in Hourdakis et al., 2003). The Theil inequality coefficient was used because it was believed to be much more accurate and sensitive to differences in mean, standard deviation, and unsystematic error than both the *RMSP* and correlation coefficient. The full Theil inequality coefficient is presented in Equation 2-3.

The Theil inequality coefficient, U , was a conglomeration of three detailed statistical calculations. The first, U_m , was a quantity called “bias proportion”. This value represented the consistency of systematic errors due to variations, or bias, in the mean values which were being evaluated. The next statistical calculation, U_s , was called “variance proportion”. This value measured the ability of simulated measurements to

produce the same degree of variability as the actual field measurements. The final statistic, U_c , represented a quantity called “covariance proportion”, and was a measure of non-systematic errors. This final Theil coefficient component made use of the correlation coefficient mentioned previously. All three components of the Theil inequality coefficient are presented in Equations 2-4 through 2-6, respectively.

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

where :

n = number of observations

x_i = simulated measurement at time i

y_i = actual measurement at time i

$$r = \frac{1}{n-1} \sum_{i=1}^n \left(\frac{(x_i - \bar{x})(y_i - \bar{y})}{s_x s_y} \right) \dots\dots\dots (2-2)$$

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

$$U = \frac{\sqrt{\frac{1}{n} \sum_{i=1}^n (y_i - x_i)^2}}{\sqrt{\frac{1}{n} \sum_{i=1}^n y_i^2} + \sqrt{\frac{1}{n} \sum_{i=1}^n x_i^2}} \dots\dots\dots (2-3)$$

$$U_m = \frac{n(\bar{y} - \bar{x})^2}{\sum_{i=1}^n (y_i - x_i)^2} \dots\dots\dots (2-4)$$

$$U_s = \frac{n(s_y - s_x)^2}{\sum_{i=1}^n (y_i - x_i)^2} \dots\dots\dots (2-5)$$

$$U_c = \frac{2(1-r)ns_y s_x}{\sum_{i=1}^n (y_i - x_i)^2} \dots\dots\dots (2-6)$$

where :

r = correlation coefficient

Hourdakis et al. (2003) believed that the primary focus during model calibration should be to obtain satisfactory values for all three calibration statistics, including the three components of the Theil (1961) inequality coefficient. Table 2-1 contains favorable values recommended by Hourdakis et al. (2003), although the authors did not mention their basis for choosing these values. Unacceptable values for the *RMSP* and *r* statistics would usually mean that certain global parameters such as vehicle acceleration and deceleration rates need adjustment. The authors also made specific recommendations for correcting certain combinations of favorable values.

Table 2-1: Favorable test statistic values as recommended by Hourdakis et al. (2003)

<i>RMSP</i>	<i>r</i>	<i>U</i>	<i>U_m</i>	<i>U_s</i>	<i>U_c</i>
< 15%	> 0.8	< 0.3	≤ 0.1	≤ 0.1	≥ 0.9

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 process outlined in Park and Schneeberger (2003) was presented in nine distinct steps:

1) Determine of measures of effectiveness:

Identify performance measure(s) (e.g. travel times), uncontrollable parameters (e.g. corridor geometry and signal timings), and controllable parameters (e.g. minimum headway and minimum lane change distance)

2) Collect data:

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.

2.3 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 before continuing. According to the Nevada Revised Statutes (2004), Chapter 484, Section 325, Articles 1 and 3 pertain to marked mid-block pedestrian crossings:

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.

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. It is also important to understand that the proportion of drivers that do not yield to a pedestrian may not necessarily be the proportion of drivers that are disobeying these two laws.

2.4 Driver and Pedestrian Behavior

While many publications exist that address pedestrian safety (Cui & Nambisan, 2003; Pulugurtha & 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 profound 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, p. 28). 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.

2.5 Pedestrian Critical Gap Acceptance

The problem presented in El-Hakim (1999) was to develop a mathematical model to predict critical gap acceptance times for pedestrians in Greater Cairo (Egypt) by studying the varying pedestrian behaviors at crossings. It was hypothesized that critical gap acceptance was dependent on the road width, average vehicular speed, average pedestrian speed, traffic volume, and distance from the site to the nearest signalized crossing.

The research showed that the average pedestrian speed and gap acceptance/rejection data fit the respective generally accepted theories. That is, pedestrian speed was normally distributed, and gap acceptance/rejection followed a lognormal distribution. Pedestrian speeds were higher for crossing roads than sidewalks. Linear regression yielded a model to predict critical gap acceptance, with an R-squared value of 0.973 and a significance of 0.01. The equation developed by El-Hakim (1999) is shown in Equation 2-7.

$$G = 2.43 + wv + 0.06S + 42D^{-1} - 0.0018Q \quad \dots\dots\dots (2-7)$$

where :

G = Critical gap acceptance of pedestrians (50% accepting), or the
intersection of the acceptance and rejection curves (seconds)

w = Length for pedestrian to traverse roadway (meters)

v = Mean pedestrian crossing speed (meters per second)

S = Mean vehicle speed (kilometers per hour)

D = Distance to nearest signalized pedestrian crossing (meters)

Q = Hourly vehicular volume (vehicles per hour)

All five variables were found to be statistically significant, thus confirming the original hypothesis of the study. One drawback of this model is that it does not consider platooning of vehicles as a determining variable in critical gap acceptance. At an isolated location with random arrivals, pedestrians would be confronted with smaller gap times, and would be more likely to select an inappropriately small gap. Traffic signals create larger gap times after platoons pass, so pedestrians would be less likely to accept an inappropriately small gap.

Because of the unique nature of pedestrian and vehicular transportation in Cairo, this model presented by El-Hakim (1999) may not be applicable to other countries. This model assumes a random crossing point between two intersections, which in the United States, is regarded as illegal jaywalking. However, it is possible that this model may be applicable to pedestrians who are unaware that he or she has the right-of-way at a mid-block crossing.

El-Hakim (1999) also proposed that this model could be used for computing pedestrian delays, making it useful as a tool for operational analysis. The model may be applicable at other unsignalized mid-block pedestrian crossings worldwide, but platooning effects may need further consideration.

Since pedestrians have the right-of-way at unsignalized mid-block pedestrian crossings in Nevada, pedestrians should not need to choose gaps while queued at the crosswalk (assuming all drivers comply with the law). However, not all drivers yield lawfully to pedestrians. In this case, there is a combination of vehicles that comply and do not comply with Nevada pedestrian crossing laws. If pedestrians can perceive that an approaching vehicle will not yield to him/her, the pedestrian must decide the acceptable gap between non-lawful vehicles.

2.6 Pedestrian Speeds and Start-up Times

The amount of information on pedestrian kinematics at intersections is limited. Many previous studies have acknowledged the difference between crossing speeds of younger and older pedestrians, but little research exists to quantify these differences. Knoblauch, Pietrucha, and Nitzburg (1996) conducted a study to determine the effects of age and crossing locations on the distribution of pedestrian walking speeds. This study also addressed the distribution of start-up times for pedestrians at crosswalks. Their results showed that age was a significant factor in the mean and 15th percentile pedestrian crossing speeds. Age was also significant for the mean and 85th percentile start-up times. The research identified certain geometric and environmental conditions that significantly

affected crossing speeds. However, it was found that these conditions did not significantly impact crossing start-up times.

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 were recorded for each location, such as street widths, grade changes, presence of medians, and crosswalk channelization markers. Weather conditions were also noted, with particular information being collected on wind speeds. The observers collecting pedestrian information estimated the age range of each pedestrian. This methodology was verified prior to the study by sampling actual pedestrian ages and comparing them with the observers' estimates. Certain pedestrians were excluded from the study; among these were children (under 13 years old), pedestrians carrying or leading heavy objects, and disabled pedestrians. The observers were also careful to exclude pedestrians crossing diagonally through intersections and pedestrians running while entering the crosswalk.

Knoblauch et al. (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 was 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. Among the conditions that the authors found to be statistically significant were:

- Gender
- State/Urban area
- Day of week (weekdays)
- Group size (alone, grouped)
- Weather conditions (dry, drizzle, rain, and snow)
- Wind (low, medium, and high)
- Temperature (low, medium, and high)
- Street classification (arterial, collector/distributor, and local)
- Street parking (permitted, restricted, prohibited)
- Pedestrian signal type (none, words, picture)
- Median presence
- Crosswalk markings (“standard”, “highly visible”)
- Roadway width
- Number of lanes
- Vehicle volumes
- Walk times

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 et al. (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 pedestrians traveling as part of a group 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 et al. (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 et al. (1996) at pedestrian crossings. Table 2-3 shows start-up times recommended by Knoblauch et al. (1996).

Table 2-2: Speed measurements at pedestrian crossings for individual pedestrians crossing legally, as compiled from Knoblauch et al. (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 et al. (1996)

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

2.7 Pedestrian Level-of-service (LOS) Methodology

Beside the standard Highway Capacity Manual (Transportation Research Board [TRB] 2000) methodologies for computing levels of service, it was necessary to establish pedestrian level of service methodologies. Baltes and Chu (2002) developed a methodology for determining a level of service index for mid-block pedestrian crossings. It was hypothesized that the level of service could be modeled by various factors (determinants) that were important components of pedestrian crossing behavior.

This level of service model was partially created through the use of hired study participants, rather than random users of the crossings. The study participants (pedestrians) were hired through a staffing agency to answer level of service questionnaires. These participants observed crossing conditions for three minutes each, then ranked the site on a scale from A to F regarding their perception of the amount of difficulty they perceive if they were to cross at that crosswalk. These observations formulated the basis for developing the “quality of service” model using determinant characteristics.

According to Baltes and Chu (2002), some of these determinant factors were user age, gender, vehicular volumes, turning movements, average speed, crossing widths, median treatments and widths, presence of crosswalks, presence of pedestrian signals,

signal cycle length, and signal spacing. A linear relationship was found between the determinants and the “quality of service” ranking by study participants. The authors were able to develop two models that predict level of service for mid-block pedestrian crossings: a directional model and a combined model. The directional model predicted level of service for each side of a crossing when the variables for each direction of travel were significantly different. The combined model predicted level of service for a mid-clock pedestrian crossing that had relatively uniform variables for all directions of travel. Level of service is determined by a scale of 1 to 6, where a value of 1.5 corresponds to the threshold for LOS A and 5.5 for LOS F. The directional model developed by Baltes and Chu (2002) is shown in Equation 2-8.

The research showed that the level of service could be reasonably predicted using the various determinants as measured in the field, with an R-squared value of 0.34. While this error is acceptable and reasonable, Baltes and Chu (2002) stated that other similar studies at signalized intersections have yielded R-squared values of 0.85.

Directional Level of Crossing Difficulty =

$$\begin{aligned}
 & -2.4778 + 0.4937A - 0.1159B_1 + 0.2674B_2 + 0.0018C_1 + 0.0013C_2 \\
 & + 0.0107D - 0.0852E_1 + 0.1241E_2 - 0.0661F - 0.0712G \dots\dots\dots (2-8) \\
 & - 0.2762H - 0.4930J - 0.0326K_1 + 0.0610K_2 + 0.0007L
 \end{aligned}$$

where :

- A = Proportion of pedestrians 65 or over (as a decimal value, 0 - 1)
- B_1 = Near side vehicular volume (for every 1,000 vehicles per hour)
- B_2 = Far side vehicular volume (for every 1,000 vehicles per hour)
- C_1 = Near side turning movements between signalized intersections (vehicles per hour)
- C_2 = Far side turning movements between signalized intersections (vehicles per hour)
- D = Running speed (miles per hour)
- E_1 = Near side crossing width, excluding median (feet)
- E_2 = Far side crossing width, excluding median (feet)
- F = Width of raised median (feet)
- G = Width of painted median (feet)
- $H = \begin{cases} 1 & \text{if crosswalk is marked} \\ 0 & \text{if otherwise} \end{cases}$
- $J = \begin{cases} 1 & \text{if pedestrian signal present} \\ 0 & \text{if otherwise} \end{cases}$
- K_1 = Near side cycle length of two closest signalized intersections (seconds)
- K_2 = Far side cycle length of two closest signalized intersections (seconds)
- L = Spacing between closest signalized intersections (feet)

The research conducted by Baltes and Chu (2002) was part of an ongoing research effort by the Florida Department of Transportation regarding level of service analysis at mid-block and intersection pedestrian crossings. Because of this continuing research,

more information on quality of service at mid-block pedestrian crossings should be available in the next few years. One possible direction for continued research could be to determine other factors that might increase the regression fit (R-squared value). This model could be used on other mid-block crossing areas, but caution should be exercised because the study locations were all located within one urban area. Pedestrian laws and behaviors in other regions may differ from the study locations.

Milazzo, Rouphail, Hummer, and Allen (1999) conducted a literature synthesis to update walking speeds and pedestrian facility level of service standards for the year 2000 edition of the Highway Capacity Manual. Among the recommended updates were new measures of effectiveness strategies for signalized and unsignalized pedestrian crossings. These measures of effectiveness included methodologies for determining level of service at pedestrian facilities using space and delay as criteria.

Milazzo et al. (1999) recommended that space be retained as a measure of effectiveness, but not the main measure of effectiveness. The authors believed that pedestrian delay should be the main measure of effectiveness at signalized intersections. They also recommended the use of pedestrian delay as the main measure of effectiveness for unsignalized pedestrian crossings. However, queuing space was not included as a determining factor in level of service calculations at unsignalized pedestrian crossings. Table 2-4 contains the recommendations by Milazzo et al. (1999) for level of service criteria at unsignalized pedestrian crossings.

Table 2-4: Unsignalized pedestrian crossing level of service criteria. Source:
Milazzo et al. (1999, p.30)

Level of service	Average delay per pedestrian (seconds)*	Likelihood of pedestrian risk-taking behavior**
A	< 5	Low
B	5 – 10	Low
C	10 – 20	Moderate
D	20 – 30	Moderate
E	30 – 45	High
F	≥ 45	Very High
* Delay is defined as time waiting on one side of intersection plus time waiting in median		
** Likelihood implies chance pedestrian will accept an inappropriately small gap		

Kaseko and Karkee (2005)'s model presented useful concepts toward modeling vehicle and pedestrian interactions. Both Hourdakis et al. (2003) and Park and Schneeberger (2003) presented practical methodologies for calibrating and validating simulation models. However, the studies differed on the order in which certain steps should be conducted. A methodology to calibrate and validate mid-block pedestrian crossing models would likely benefit from a synthesis of these procedures. Redmon (2003) and Nevada Revised Statutes (2004) included motivations for vehicles to yield for pedestrians, but did not address actual behaviors. Redmon (2003) did provide justification for creating two types of drivers. The accepted gap estimates presented in El-Hakim (1999) were not applicable to the accepted gaps in the study situation. Knoblauch et al. (1996) was useful for developing pedestrian speed distributions, particularly for pedestrians traveling as a group. The pedestrian level of service methodologies presented in Baltes and Chu (2002) and Milazzo et al. (1999) were useful for demonstrating potential uses for the delays generated from simulation models.

CHAPTER 3

METHODOLOGY

3.1 Methodology Development

The methodology that has been developed for this study was a synthesis of techniques. Most traffic data collection methodologies were similar to those recommended by the Manual of Traffic Engineering Studies (Institute of Transportation Engineers [ITE] 1994). Other data collection techniques, such as site inventories, utilized accepted practice methods from professional experience and knowledge of the levels of accuracy necessary. Many of the crosswalk yield parameters were based upon the original work done by Kaseko and Karkee (2005). However, new yield conditions and parameters were added for this study based upon empirical observations at the crosswalk. The calibration and validation methodologies developed were similar to those outlined in Park and Schneeberger (2003). The simulation values generated for calibration were compared using the Theil (1961) non-parametric goodness-of-fit tests as discussed in Hourdakis et al. (2003). After reviewing the applicable literature, it was decided that local parameters also needed calibration. Since the global parameters varied in the first calibration step (“major calibration”) affected each segment differently, variables such as the vehicle/pedestrian yielding parameters needed adjustment. Thus, a second calibration step (“minor calibration”) was added to the methodology, and local parameters adjusted manually to achieve desired travel time distributions for each segment. Analyses and

discussions regarding sensitivity, level-of service, and potential implications will be addressed in the Case Study Results (Chapter 4), and Conclusions (Chapter 5).

3.2 Case Study Location

The study location was a mid-block pedestrian crossing on Maryland Parkway adjacent to the University of Nevada, Las Vegas campus, which is approximately halfway between Harmon Avenue and University Road. This pedestrian crossing is a “Danish offset” type crossing. The roadway segment has a posted speed limit of 30 miles per hour. According to Nevada Department of Transportation (2003), the average annual daily traffic count on Maryland Parkway near the study location was roughly 35,200 vehicles. This portion of Maryland Parkways is also currently being considered for major redevelopment (UNLV 2004). Figure 3-1 shows a map of the area for orientation purposes, with the project location specified. Figure 3-2 is a photograph of the case study mid-block crossing on Maryland Parkway.

Almost all observations for this study were conducted during the midday period, 11:00 AM to 1:00 PM. The intent was to sample vehicles and pedestrians from midday traffic. This period reflects the daily peak for pedestrian traffic at this crosswalk. However, the proportion of yielding vehicles study showed that collecting data between 11:00 AM and 1:00 PM would be difficult, and result in considerably fewer suitable observations. The proportion of yielding drivers data was collected on two subsequent days, between 10:00 AM through 11:00 AM, and 1:00 PM through 2:00 PM. The midday period typically extends from 9:00 AM through 2:00 PM, and so it was determined that observations during this time should adequately represent midday traffic behaviors.

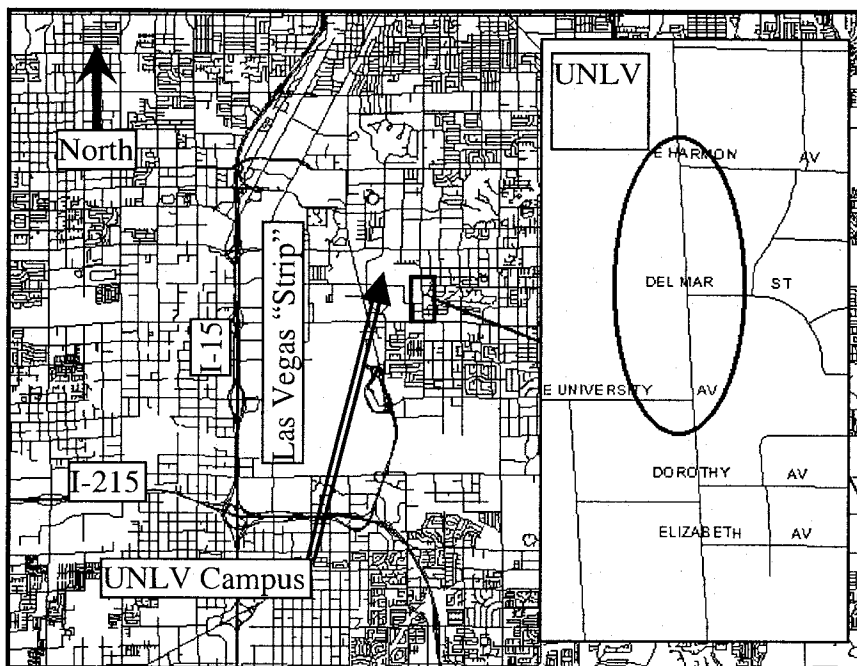


Figure 3-1: Vicinity map of the project location on Maryland Parkway



Figure 3-2: Case study mid-block pedestrian crossing on Maryland Parkway

3.3 Methodology Overview

The methodology developed for this project was divided into three stages: model development, calibration, and validation. Model development was perhaps the most crucial stage, since any omissions in the model design could severely impact the calibration and validation stages. Care was taken to ensure that the level of accuracy required for each data collection effort would be the minimum necessary to allow for proper model function, but that the data collection process was resourceful and not overly cumbersome. Model calibration and validation was conducted only after the model design had been finalized. Figure 3-3 explains these steps graphically in a flow chart of these three steps, with each minor step included.

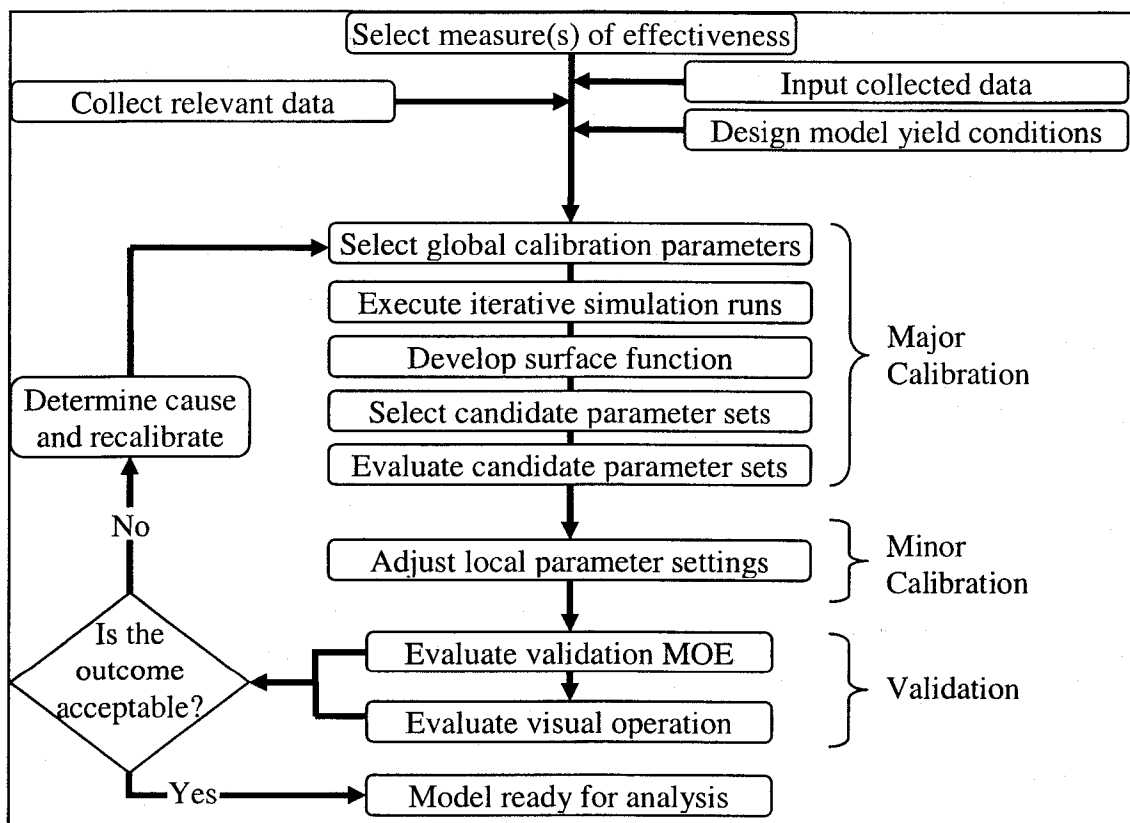


Figure 3-3: Flow chart of model development, calibration, and validation process

3.4 Model Development

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 previous section. Results produced after applying the methodology as outlined can be found in the Case Study Results (Chapter 4).

3.4.1 Measures of effectiveness selection

The calibration measures of effectiveness used were the northbound and southbound travel times between the stop bars between Harmon Avenue and the crosswalk, and between the crosswalk and University Road (refer to Figure 3-1). Only passenger cars were included in both field and model travel time measurements. 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. If the travel times were only measured for the entire segment or network, it could lead to discrepancies in the control delays incurred at the crosswalk, and produce incorrect control delays at the downstream intersection.

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). The number of vehicles queued in the middle lane for both directions of traffic were used for model validation.

3.4.2 Data collection

Several data collection efforts were necessary along the Maryland Parkway study corridor. Most of these collection efforts were routine studies 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 intersection volume studies provided critical vehicular and pedestrian information within the study area. One field study, the proportion of drivers yielding to pedestrians, was independently developed. Sample data collection sheets are included in Appendix I for reference. The following is a description of site investigation and observation studies conducted for this project. Detailed results of these studies can be found in the Case Study Results (Chapter 4).

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. A field drawing was necessary to record intricate dimensions such as the width of each lane, distance between the yield bar and crosswalk, and location of the crosswalk centerline.

Determining the vertical grade was also important, since acceleration and deceleration of heavier vehicles can be affected by changes in elevation. Generally, this information was obtainable through the respective transportation authority, or through the United States Geological Service. Grades of less than 1% would likely not affect heavy vehicles over short distances.

The routes and timetables of public transit vehicles were also considered. Bus routes, especially those which board/alight in the rightmost lane, were expected to introduce significant travel time delays to the corridor.

It was also necessary to determine signal timings for the two signalized intersections in the study corridor. Since average values were desired outcomes of this study, all signals were assumed to be pretimed (not actuated), using nominal green split times.

Intersection volume studies were conducted at two signalized intersections and one unsignalized intersection. These volume counts were designed so that count durations complemented the signal cycles. Each of these studies differentiated between passenger cars/motorcycles, heavy vehicles, and buses. U-turns were counted separately from left turns.

Pedestrian counts were taken at the mid-block crosswalk. Pedestrians were differentiated into four groups: those who appeared age 65 and under; appeared over 65 years old; were disabled; and were children under age 14. Groups of pedestrians were also noted, as well as how many pedestrians were in the group.

Vehicle occupancies were sampled for vehicles traveling through the corridor. Vehicles were selected randomly during a certain time period, and the number of occupants was tallied. This information was intended to be used for calculating average delays per person, and was an optional step.

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".

Desired speeds, or free flow speeds, were sampled on both directions of traffic. A standard law-enforcement radar gun was used to sample speeds. 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. It was intended to also sample actual average speeds at four places along the study corridor. However, due to obstructions and equipment limitations, only one location was sampled.

The proportion of drivers yielding to a pedestrian at the crosswalk was a necessary input variable in VISSIM. Unfortunately, no previous methodologies on how to collect this information were found at the time this study was conducted. As a result, a new procedure was developed to collect and estimate this proportion.

It should be noted that there are several reasons why a driver would not yield to a pedestrian. There are three reasons why a driver would fail to yield to a pedestrian in this situation:

- Ignorance of Nevada State Law: the driver is unaware that failing to yield to a pedestrian waiting at or in a crosswalk in the State of Nevada is unlawful
- Inability to recognize the condition to yield: the driver does not recognize the area as being a legal crosswalk either by lack of proper signage or markings, or the driver's view of a pedestrian or stopped vehicle is obstructed
- Willful non-compliance: the driver wantonly disregards Nevada State Law (defined as pedestrian-aggressive behavior in this study)

A driver that possesses at least one of these three reasons, would by definition, be a non-yielding driver. The two reasons "Ignorance of Nevada State Law" and "Willful non-

compliance” are both unlawful behaviors. However, “Inability to recognize the condition to yield” is a legitimate defense, and could be considered lawful. Therefore, the proportion of non-yielding drivers does not necessarily represent the proportion of pedestrian-aggressive drivers, nor does it accurately represent unlawful behavior. A yielding driver could not possess any of the three previously mentioned reasons.

Unlike a standard stop compliance study or yield compliance study at an unsignalized intersection, the condition to yield only exists when a pedestrian is present at the crosswalk or in the crosswalk. This proportion, therefore, must be extracted from observations. As discussed in Redmon (2003), there was a significant difference in motives of drivers (defined as pedestrian-aggressive and pedestrian-friendly motives in this thesis) concerning pedestrian use of the roadway. Redmon (2003) also discussed that certain drivers were ignorant of the respective laws regarding yielding to pedestrians. Kaseko and Karkee (2005) defined aggressive and non-aggressive drivers as a difference in motivation. To develop the methodology for estimating the proportion of drivers who will yield to pedestrians, it was necessary to define a condition in which non-yielding driver behavior would be obvious enough to be distinguished from a yielding driver. Random occurrences of these incidents would produce a frequency distribution in which the proportion of yielding drivers could be estimated. Furthermore, it was necessary to define differences between the previously mentioned behaviors and motivations, to develop an understanding of what this proportion represented.

The only possible way to observe yielding and non-yielding drivers was to observe behaviors at the crosswalk in question. Yielding drivers, by definition, would yield to a pedestrian, except when the vehicle was not able to stop within the stopping sight

distance. Non-yielding drivers, however, would not yield unless the pedestrian was blocking the travel lane. Distinguishing between drivers that would and would not yield could only occur when a pedestrian was present. These behaviors could be observed in any lane. Since the condition no longer existed once a vehicle stopped, the vehicles upstream of the yielding driver could not be distinguished. If pedestrian arrivals are random, this situation fit the description of a Geometric distribution. Therefore, the working hypothesis was that the distribution of yielding drivers could be represented as a Geometric distribution of vehicles that yield to a pedestrian waiting to cross, and the proportion could be estimated from the frequency of these individual occurrences. The general probability formula for a Geometric distribution is shown in Equation 3-1.

$$f(x) = p(1 - p)^{x-1} \dots\dots\dots (3-1)$$

where :

$f(x)$ = Probability distribution of trials before first positive outcome

p = Bernoulli (raw) proportion of obtaining a positive outcome

$x - 1$ = Total number of trials before first positive outcome

In this case, the proportion p would represent the proportion of yielding drivers, which was unknown. To estimate population proportion, observations producing an entire probability histogram where the number of trials, or total number of vehicles observed, was used to work backward (via the Method of Moments) to calculate the sample proportion. This proportion would represent the vehicles that yield to pedestrians at the crosswalk; vehicles that do not yield (for any reason) would be represented by $1 - p$.

Figure 3-4 illustrates the procedure used to collect yielding and non-yielding vehicle data. Observations were taken upon the arrival of a pedestrian at the crosswalk queue. Drivers who do not yield, “NY”, were counted after waiting an initial time to clear the stopping distance area, “SSD” (determined to be about 3 seconds at 30 mph). This count continued until a vehicle, “Y”, yielded to the pedestrian. This set of events corresponded to one total observation. For instance, if two cars failed to yield after the initial clearance time before the third car stopped, the number failing, $x - 1$, will be 2, x will be 3, and would consist of one total observation. Observations were only taken when a vehicle queue did not exist before the crosswalk area before the event, since these clearance intervals would be shorter than calculated at the mean vehicular speed. Observations also did not include incidents where the pedestrian had already entered the roadway prior to a vehicle stopping. A sample field data collection sheet can be found in Appendix I.

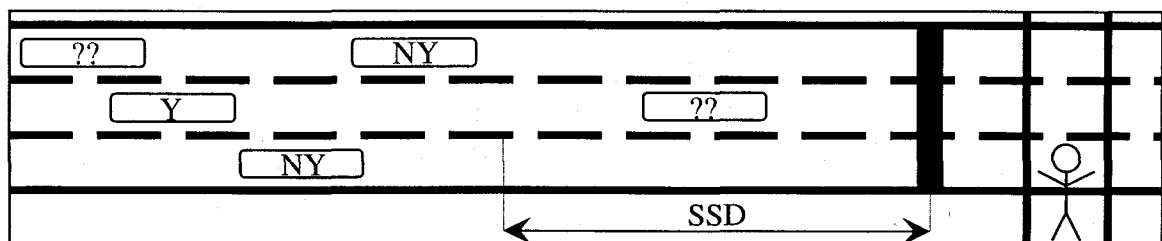


Figure 3-4: Illustration of the data collection procedure to estimate the proportion of yielding drivers

Since the mean of a Geometric distribution is simply the inverse of the sample proportion, the mean was calculated from the observed frequency histogram. As mentioned previously, this calculation utilized the Method of Moments. The result was the sample proportion. The formula for calculating sample proportion for a Geometric

distribution is shown in Equation 3-2. A standard Chi-squared goodness-of-fit test was used to verify that the estimated proportion produced a Geometric distribution.

Since $\bar{x} = \frac{1}{\hat{p}}$ for a Geometric distribution,

then :

$$\hat{p} = \frac{1}{\bar{x}} = \frac{N}{\sum_i (x_i * \hat{f}(x_i))} \dots\dots\dots (3-2)$$

The confidence interval for the yielding proportion estimate should be calculated to determine whether the error in the sample proportion is appropriate for its intended use. Unfortunately, an exact solution has not been discovered at the time of this publication for confidence intervals of Geometric distributions where p is unknown. Although an exact solution cannot be determined analytically, the confidence interval can be estimated using the bootstrap theory, from the percentile bootstrap method summarized in Efron and Tibshirani (1986). This method relied on creating random data using the field proportions collected for each number of trials during the actual experiment to produce several simulated experiments based on these distributions. Random data can be generated using a standard statistical software package such as Minitab. The number of simulated experiments must be sufficiently large to produce standard errors that would only increase nominally when the number of simulations is greatly increased. Efron and Tibshirani (1986) stated that 128 simulations seemed sufficient. The mean values for each simulation would then be calculated. After ranking each mean by percentile, the mean at the desired confidence interval percentile would represent an estimate of the

respective lower and upper confidence interval bound. For instance, if 100 simulations were executed, the 5th percentile and 95th percentile mean values produced would represent a 90% two-tailed confidence interval estimate. Since the mean of a Geometric distribution is the inverse of the proportion, the upper and lower confidence interval proportions could then be calculated from the respective percentile means.

3.4.3 Input collected data

The aerial photography obtained from Clark County Department of Public Works was loaded and scaled as a background in the VISSIM model. All general roadway geometries (connectors), as well as turning movements (links), were then defined from the aerial photography. General accuracy for defining roadway geometries can vary, but for this study, an accuracy of about ± 2 feet was acceptable. Because of this lax requirement, aerial photographs were best suited to draft the connectors and links for most of the corridor.

At the crosswalk, however, a higher level of accuracy was needed. Specifically, it was necessary to input the crosswalk location with respect to the centerline of Del Mar Street. Also, lane widths were checked and input into the model. Finally, it was necessary to input the distance between the edges of the crosswalk and the yield bars on Maryland Parkway.

The two bus transit routes, Maryland Parkway and Harmon Avenue via Maryland Parkway, were input into VISSIM by defining two bus transit routes with average headways of ten minutes. Stations were then input using approximate locations from the Clark County Department of Public Works bus route GIS dataset (2004). The results

were two transit lines with buses operating at a headway published on the respective CAT route timetables (RTC 2004).

Signals were input into the VISSIM model by using the information provided by FAST. Although the actual signals were semi-actuated, the simulation signals were non-actuated. Since average travel times through the corridor are the measurable outcomes of calibration, the nominal signal phase lengths were used to reduce the complexity of the model. Turning movements at the three intersections were then input into the model as static route choices. The raw turning movements were used since VISSIM automatically calculates the probability of a vehicle taking a certain route.

The traffic counts for northbound and southbound Maryland Parkway at Del Mar Street were used for the vehicle volume inputs for VISSIM. These volumes were counted from pre-recorded video. This allowed the observer to replay a missed segment if necessary to obtain the most accurate count. 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 Del Mar Street intersection, the vehicles generated per hour would need adjustment at each source.

As will be mentioned in the Model Design section, two types of passenger cars were defined: yielding and non-yielding. These traffic compositions were adjusted to include the correct proportion of yielding passenger vehicles.

The pedestrian counts at the crosswalk showed that very few pedestrians were age 65 and over. However, many pedestrians traveled in groups through the crosswalk. Prototype simulation models suggested that the presence of pedestrians traveling in groups significantly affected the vehicular delay at the crosswalk. The likely reason for this was

that since pedestrian groups travel much closer together, generating every individual pedestrian adversely increased the number of yielding incidents. Thus, two different pedestrian compositions were defined: single pedestrians and pedestrian groups. Both compositions had different speed distributions, using the respective values reported by Knoblauch et al. (1996). Pedestrian groups were modeled as walking in pairs of two. Therefore, if a group consisted of three people, that group would have a length of approximately twice a pedestrian. The average person-occupancy of the observed groups resulted in an average length assigned to the pedestrian group model definition.

Exiting and entering driveways for the upstream and downstream segments were created in the model to simulate the mid-block turning movements. Each driveway was located approximately halfway between the respective intersections. It would be impractical and too time-consuming to input every driveway for each segment. The intent of this procedure was to simulate in and out turning movements at the busiest driveway in each segment, using the entire volume counted in that segment.

3.4.4 Model design

The basic yielding parameter concepts for model operation at the crosswalk were largely based on those defined by Kaseko and Karkee (2005). However, general operational observations at the crosswalk location showed that some of these yielding concepts needed further consideration. For instance, Kaseko and Karkee (2005) did not assume that a pedestrian would yield to a non-yielding vehicle while already crossing. As a result, the conditions in Kaseko and Karkee (2005) were supplemented by adding yielding conditions for pedestrians yielding to non-yielding drivers. Figures 3-5 and 3-6

illustrate the “minimum gap” and “minimum headway” concepts, respectively, as defined in the simulation. The following yielding parameters were defined in the VISSIM model for each direction of vehicular and pedestrian traffic:

- H_{yp} : Minimum headway for yielding vehicles yielding to pedestrians
- G_{yp} : Minimum gap for yielding vehicles yielding to pedestrians
- H_{npx} : Minimum headway for non-yielding vehicles yielding to pedestrians for each lane x
- G_{npx} : Minimum gap for non-yielding vehicles yielding to pedestrians for each lane x
- H_{py} : Minimum headway for pedestrians yielding to yielding vehicles
- G_{py} : Minimum gap for pedestrians yielding to yielding vehicles
- H_{pnx} : Minimum headway for pedestrians yielding to non-yielding vehicles for each lane x
- G_{pnx} : Minimum gap for pedestrians yielding to non-yielding vehicles for each lane x

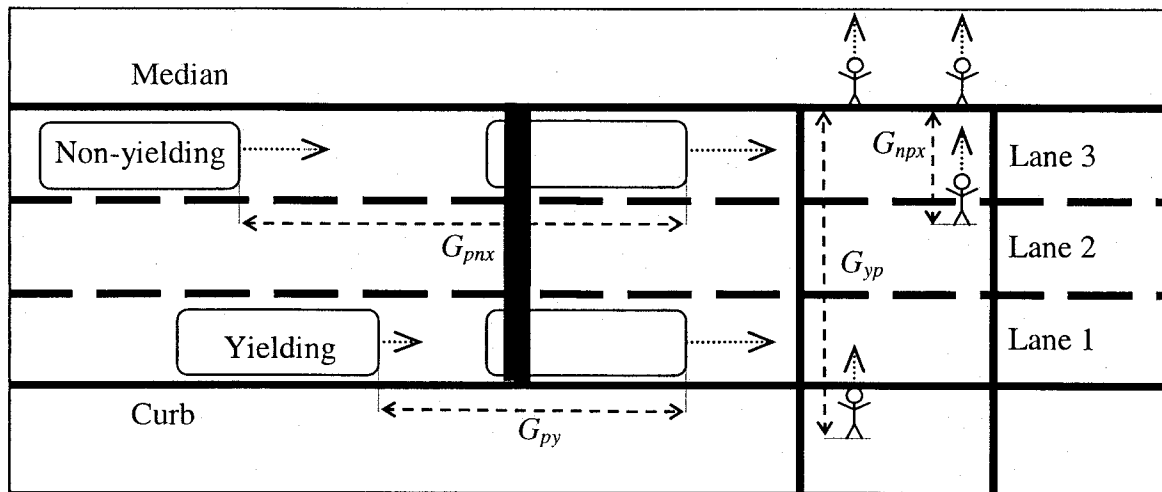


Figure 3-5: Illustration of “minimum gap” times in simulation model

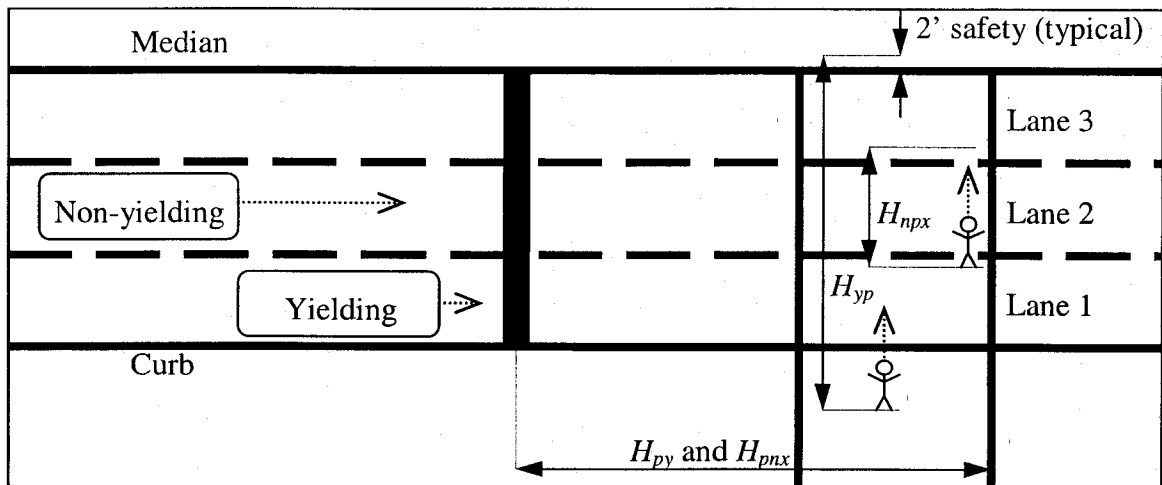


Figure 3-6: Illustration of “minimum headway” dimensions in simulation model

Two vehicle types were created: yielding and non-yielding. The proportion of yielding and non-yielding vehicles as determined in the field study were input into VISSIM as traffic compositions. Heavy vehicles and buses were assumed to all be yielding vehicles, and thus part of the yielding vehicle composition. Yielding vehicles would yield to a pedestrian when the pedestrian was within a certain distance from the

end of the crosswalk plus an additional safety distance (“minimum headway” in VISSIM), H_{yp} . Yielding vehicles would also yield to a pedestrian when the gap time between the pedestrian and the end of the crosswalk was less than a certain amount (“minimum gap time” in VISSIM), G_{yp} . When either of these conditions existed, the vehicle would yield. Once the pedestrian left the crosswalk, the yielding vehicle continued through the crosswalk, provided that another pedestrian was not present. These concepts are more clearly explained in Figures 3-5 and 3-6. Both conditions were intended to simulate these actual conditions observed in the field:

- yielding to pedestrians already in the crosswalk
- yielding to pedestrians within a distance from the crosswalk queue area

In a similar manner, non-yielding vehicles would only yield to a pedestrian that had already entered the lane in which the vehicle is traveling (see Figures 3-5 and 3-6). “Minimum headway”, H_{npx} , and “minimum gap time”, G_{npx} were defined for each lane, with the lane number corresponding to x . Non-yielding vehicles would yield to a pedestrian when the pedestrian was within a certain distance from the edge of the travel lane plus an additional safety distance. Non-yielding vehicles would yield to a pedestrian when the gap time between the pedestrian and the edge of the travel lane was less than a certain amount. Once the pedestrian left the lane of travel, the non-yielding vehicle continued through the crosswalk, provided that another pedestrian was not present. Both conditions were intended to simulate these actual conditions in the field:

- yielding only when pedestrians were in the lane of travel
- yielding only when pedestrians were too close to safely continue through

According to general field observations at the study location, pedestrians were able to tell if a vehicle intended to yield or continue through. Thus, pedestrians knew whether an approaching vehicle was a yielding or non-yielding vehicle. Pedestrians would only yield to a yielding vehicle if the vehicle was within the stopping sight distance and could not stop (see Figures 3-5 and 3-6). Pedestrians would yield to a yielding vehicle when the vehicle was within the “minimum headway”, H_{py} , from the far edge of the crosswalk, which included the distance between the crosswalk and the yield bar. Pedestrians would also yield to a yielding vehicle when the gap time between the vehicle and the far edge of the crosswalk was less than the “minimum gap time”, G_{py} . When either of these conditions existed, the pedestrian would yield. Once the yielding vehicle left the conflict area, the pedestrian continued through the crosswalk, provided that another vehicle was not present. Both conditions were intended to simulate these actual conditions observed in the field:

- only yielding to yielding vehicles already in the conflict area
- only yielding to yielding vehicles unable to stop within the stopping sight distance

Pedestrians would always yield in the adjacent lane to a non-yielding vehicle unless the non-yielding vehicle had already stopped for another pedestrian (see Figures 3-5 and 3-6). “Minimum headway”, H_{pnx} , and “minimum gap time”, G_{pnx} were defined for each lane, with the lane number corresponding to x . Pedestrians would yield to a non-yielding vehicle when the vehicle was within the “minimum headway” from the far edge of the crosswalk, which included the distance between the crosswalk and the yield bar.

Pedestrians would also yield to a non-yielding vehicle when the gap time between the vehicle and the far edge of the crosswalk was less than the “minimum gap time”. When either of these conditions existed, the pedestrian would yield. Once the non-yielding vehicle left the conflict area, the pedestrian continued through the crosswalk, provided that another vehicle was not present. Both conditions were intended to simulate these actual conditions observed in the field:

- yielding to non-yielding vehicles in the adjacent lane when the vehicle was present in the conflict area
- yielding to non-yielding vehicles in the adjacent lane when the vehicle would arrive before the pedestrian could cross the lane safely

These eight conditions provided a total of sixteen yield conditions for each direction of traffic. With two directions of vehicular and pedestrian traffic, this means that there were 64 separate yield conditions which needed to be input into VISSIM (see Appendix III). The synthesis of these 64 interactions creates a uniquely complicated situation at these types of pedestrian crossings.

3.5 Model Calibration

The calibration procedure for this model consisted of two parts: major and minor calibration. In the major calibration portion, global parameters were varied to study the relationship between each parameter and the four travel times. Local parameters were collectively varied in major calibration, although they actually represented distinct vehicular yielding decisions that regard pedestrians approaching from four different

situations. In minor calibration, only local parameters were individually varied to more closely match all four travel time segments.

3.5.1 Selection of calibration parameters

Once the model design was completed, variables within VISSIM had to be selected for calibration. 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. The pedestrian/vehicle yield conditions as mentioned in the Model Design section were also potential calibration variables.

To determine which of these variables had the greatest impact on travel times, all reasonably possible variables were identified and the default values recorded. These yielding parameters were assumed to be uniform for each similar variable in the model. Next, each parameter was assigned a feasible minimum and maximum value. The simulation was run ten times with all default variables entered into the model. Each of the four simulated travel times was recorded. 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 once, 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.

3.5.2 Major calibration

Park and Schneeberger (2003) used a Latin hypercube experimental design to optimally select parameter values to determine the effect on the measurable outcome. One major disadvantage of using the Latin hypercube design was that it assumed that the probability distribution of parameter values being adjusted was known. Another disadvantage was that it generally required numerous runs, especially for an experiment with six parameters.

The intent of the “major calibration” phase was to generate parameter sets that produced measure(s) of effectiveness that were reasonably close to those measured in the field. Thus, the general relationships between parameters would be the most important discovery in this step. For this reason, a simpler and more practical experimental design method was chosen. The 2^k factorial experimental design was chosen (k being the number of parameters), with an additional $2 \times k + 3$ experiments. The 2^k factorial experimental design tests the minimum and maximum (or low and high) values for each parameter, to determine the interactions between parameters on the measurable outcome. The additional $2 \times k$ experiments were included to determine the effects of a minimum or maximum value of one parameter on intermediate values of every other parameter. Generally, this intermediate value is the value that is halfway between the high and low values. However, since there are default values for the parameters in VISSIM, and since these default values were relatively close to the median values, the default values were used in these additional experiments. The final 3 additional experiments were run using the default values for every parameter, but using different random seed values. These

final experiments were intended to estimate the variance produced by VISSIM when different combinations of random seeds were used.

Once travel times were generated from the factorial experiments, surface functions representing the calibration parameters could be constructed to aid in estimating possible candidate parameter sets. One method for obtaining these candidate parameter sets using the surface functions generated was to use an optimization model to minimize the total difference from produced by the surface functions and the actual travel times measured in the field. After obtaining candidate parameter sets, each set was evaluated to determine which produced the closest simulated travel time distributions to the actual travel time distributions for each segment. Methods for comparing the two distributions included calculating the test statistics mentioned in Hourdakakis et al. (2003), comparing the sample means and standard deviations, and comparing the two proportion histograms.

3.5.3 Minor calibration

Using the selected set as a base for comparison, local parameters at or near the crosswalk location were varied to determine the differences in travel time distributions, in an effort to make the simulated proportion distribution match more closely with the actual distribution. Local parameters to vary include adjusting any of the 64 individual vehicle/pedestrian yielding parameters as discussed in Section 3.4.4, introducing reduced speed areas in the model, and adjusting emergency stop and maximum lane change distances on roadway links. The process for choosing which variable to vary and the amount by which it should be varied was subjective. Adjusting these variables was an iterative process, until either a more favorable output was obtained, or it was determined

that the output by altering a certain variable would only produce more unfavorable outcomes. The same test statistics as used in the Major calibration portion were used for comparison in minor calibration. Also, frequency histograms were used to visually compare scenarios, as well as the simulated average and standard deviation travel time values.

3.6 Model Validation

Model validation was conducted in two stages. First, the model validation measure of effectiveness was compared to the actual observed measure of effectiveness. Next, the visual operation of the model was observed to ensure proper operations. If either of these validation measures were determined to not be acceptable, the model would be evaluated to determine possible reasons why the validation measures were not acceptable. Then, the model would be referred back to the appropriate stage to re-calibrate and re-validate.

3.6.1 Validation measure of effectiveness evaluation

The number of queued vehicles at each approach to the crosswalk were used for the model validation measure of effectiveness. Queues were recorded every 30 seconds, and only for the middle through lane. The middle through lane was chosen for several reasons. First, it was the least likely to experience unreasonably high queues because of right turns, and because the actual queues in the median lane tended to be shorter. The number of observers available at that time would have made a full queue count impossible and inaccurate. This study was conducted at the same time as the License plate study used for travel times, but with different observers. From this data, two

frequency distributions were created for the actual observations, representing the number of occurrences that a vehicle queue would involve a certain number of vehicles in the middle through lane. For the simulation, VISSIM was only capable of generating average queue lengths (in feet) every 30 seconds. The VISSIM default minimum speed requirements were used for recording queue lengths, but the default maximum headway to consider part of the same queue was reduced from 65.6 to 15 feet. Since the VISSIM output was in actual lengths, it was necessary to translate these lengths into average number of vehicles in the queue. VISSIM vehicles tend to be evenly distributed in all three lanes of a queue, and so the queue length conversion was intended to produce the average number of vehicles in one lane. This measurement was compared to the actual number of vehicles in the middle through lane queue.

The data obtained for the actual and simulated vehicles in queue was a discrete sample, and so it was decided that a standard Chi-squared test would be sufficient to test for goodness-of-fit. The actual vehicles in queue were considered the observed frequency, and the simulated vehicles in queue were the hypothesized frequency. Detailed results of this calculation and analysis are included in Case Study Results (Chapter 4).

3.6.2 Visual evaluation

Generally, the model should function similarly to the actual location. However, there are certain common model behaviors that do not occur in the actual corridor. The collision of vehicles and/or pedestrians at the crosswalk location is an unlikely event, but the model may show several more occurrences of this. Care should be taken to observe

these areas and determine if this is a result of incorrect yielding parameters or incorrect geometry. If these possibilities are ruled out, then the collisions are most likely a result of the simulation program. It can be assumed that a simulation model is not capable of modeling every possible behavior, and that many of these collisions would be avoided at the actual location because of various intricate corrective maneuvers.

3.6.3 Model revision, recalibration, and revalidation

. If either of these validation measures were determined to not be acceptable, the model would be evaluated to determine possible reasons why the validation measures were not accepted. Then, the model would be referred back to the appropriate stage to re-calibrate and re-validate. This process would be iterative, until either the model conformed to both calibration and validation measures of effectiveness, or it was determined that the model could not be accepted as being an accurate representation of the operations at the study area.

CHAPTER 4

CASE STUDY RESULTS

Using the methodology as explained in Chapter 3, data was collected and the model was developed, calibrated, and validated to adequately reproduce actual operations at the study location during the midday period. The outcome for determining the proportion of yielding drivers at the crosswalk was also evaluated. Also, control delays for both vehicular and pedestrian traffic at the mid-block crossing were determined from the model simulation. Levels of service were determined for the Maryland Parkway corridor, using methods published in the Highway Capacity Manual (TRB 2000) and those addressed in the Literature Review (Chapter 2). Finally, sensitivity analysis was performed on the final calibrated model to determine how certain parameters affected measures of effectiveness.

4.1 Model Development

To determine actual travel times for calibrating the simulation model, a standard license plate survey was conducted on November 17, 2004 (Wednesday) on northbound and southbound Maryland Parkway between the hours of 11:00 AM and 1:00 PM. Observers were stationed at Harmon Avenue, the mid-block crossing, and University Road. Since vehicles in the curb and median lanes were most affected by turning movements and weaving, only passenger cars traveling in the center lane (of three lanes)

were included in the study. The observers recorded the first three characters on each license plate, as well as the time at which the vehicle completely passed over the stop bar. The recorded times between matching license plates produced average travel times at each of the four segments (Harmon to crosswalk, crosswalk to University, University to crosswalk, and crosswalk to Harmon). Average travel times from Harmon to University and University to Harmon were also produced. Table 4-1 shows the calculated travel times on each segment.

Table 4-1: Travel times calculated from Maryland Parkway license plate study

	Northbound (seconds)		Southbound (seconds)	
	University-Crosswalk	Crosswalk-Harmon	Harmon-Crosswalk	Crosswalk-University
Average travel times	19.4	48.3	28.7	23.4
Standard deviation	10.4	25.8	9.9	14.1
Sample size	46	42	40	72

The geometry of Maryland Parkway was necessary to obtain for an accurate model. In lieu of a field study to determine the appropriate dimensions, digital aerial photography from spring 2004 was obtained from the Clark County Department of Public Works. Furthermore, a detailed field drawing was made at the crosswalk site on November 11, 2004. The actual field drawing is included in Appendix I.

The elevation differences on the centerline of Maryland Parkway between Harmon Avenue and University Road were obtained from the National Elevation Database, made available on The National Map webpage on November 24, 2004, (United States Geological Survey [USGS] 2004). The elevation difference between the intersections of Harmon Avenue and University Road was only about 5 feet. This means that the grade of

Maryland Parkway between these two intersections is about 0.4%, which was not significant enough to impact heavy vehicles over such a short distance.

Bus timetables along Maryland Parkway and Harmon Avenue were accessed on the website for Citizen's Area Transit (CAT) on November 29, 2004. Bus stop locations were determined through the use of GIS information published by Clark County Department of Public Works (2004).

Signal timing and offset data 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. These details included green times, cycle lengths, offsets, all-red and yellow times, cycle phasing, and signal actuation design.

Intersection volume studies were conducted at Harmon Avenue, Del Mar Street, and University Road. These studies took place on November 30 (Tuesday), December 1 (Wednesday), and November 23, 2004 (Tuesday), respectively, between the hours of 11:00 AM and 1:00 PM. The study at Del Mar Street included videotaping through traffic on Maryland Parkway, and counting it later in the office. Table 4-2 shows a summary of the through volumes as measured in the field.

Table 4-2: Through volumes from intersection volume studies

Roadway	Through volumes			
	NB	SB	EB	WB
Maryland Parkway	1,606	1,402	---	---
Harmon Avenue	---	---	275	390
University Road	---	---	255	297
Del Mar Street	---	---	141	107

Pedestrian counts were taken at the mid-block crosswalk on November 17, 2004 (Wednesday) between 11:00 AM and 1:00 PM. Table 4-3 shows a summary of the pedestrian counts at the mid-block crossing location.

Table 4-3: Pedestrian counts at the mid-block crossing

Eastbound pedestrians	Westbound pedestrians
160	135

The vehicle occupancy was observed on November 9, 2004 (Tuesday). Table 4-4 shows a summary of the vehicle occupancy measurements at the mid-block crossing location.

Table 4-4: Vehicle occupancy as measured at the crosswalk

Number of samples	Average vehicle occupancy
133	1.21

Driveway turning movements were counted on the northbound portion of Maryland Parkway on November 15, 2004 (Monday) between 11:00 AM and 1:00 PM. Since there is a raised median and are no curb cuts on southbound Maryland Parkway between Harmon Avenue and University Road, no turning movements were possible except at intersections (refer to Figure 3-1). All northbound right turning movements in and out of driveways upstream and downstream of the crosswalk were counted (except for those at Harmon Avenue, Del Mar Street, and University Road). Table 4-5 shows a summary of the driveway turning movement counts at the mid-block crossing location.

Table 4-5: Driveway turning movement counts as measured at the crosswalk

Location	Entering	Exiting
Northbound upstream	124	74
Northbound downstream	41	55

Desired speeds, or free flow speeds, were measured on northbound and southbound Maryland Parkway on December 11, 2004 (Saturday) between 11:00 AM and 12:00 PM. Fifty random samples of vehicle speeds were taken downstream of the crosswalk in each direction during the green through phase for the downstream intersection. In accordance with the procedure outlined in the Manual of Transportation Engineering Studies (ITE 1994), only vehicles that were at least 500 feet apart from the upstream vehicle were selected. In addition, only platoon leaders were included in the data sets. Speeds of passenger vehicles from all three through lanes were used to construct the speed distribution. An analysis of variance (ANOVA) test on northbound and southbound measurements (Table 4-6) showed that the two data sets could not be rejected as being from the same population, with a P-value of 0.964. The data collected was adjusted to correct for a measured incidence angle of approximately 20 degrees. Table 4-7 includes the average and standard deviation of free flow speeds on Maryland Parkway for both directions. The actual speed distribution was compared graphically with the theoretical distribution. The speed frequency and cumulative frequency distributions are found in Appendix II.

Table 4-6: ANOVA results for free flow speeds, northbound and southbound

Source of variation	SS	df	MS	F	P-value	F crit.
Between groups	0.04	1	0.04	0.002	0.964	3.938
Within groups	1907	98	19.459	---	---	---
Total	1907.04	99	---	---	---	---

Table 4-7: Measured free flow speeds on Maryland Parkway

Average speed, mph	33.7
Standard deviation, mph	4.67

The actual speed distribution for northbound Maryland Parkway upstream of the crosswalk was also obtained. The data was reduced in a similar manner as the desired speed data, as shown in Table 4-8.

Table 4-8: Measured actual speeds on northbound Maryland Parkway, upstream of the crosswalk

Average speed, mph	25.8
Standard deviation, mph	4.73

Observations regarding the proportion of yielding drivers were made at the case study crosswalk on November 17 and 18, 2004 (Wednesday and Thursday) from 10:00 AM to 11:00 AM, and 1:00 PM to 2:00 PM. During a pilot study of this procedure, taking observations during the actual study period (11:00 AM to 1:00 PM) proved to be difficult. Since this was a peak travel time for both vehicles and pedestrians, many queues were not cleared, which led to few usable observations within the two-hour timeframe. Since the proportion of yielding vehicles in midday traffic was thought to be uniform

through the midday period (10:00 AM to 3:00 PM), the data collection was split into two sets of observable hours per day.

The sample proportion was calculated using the method as explained in the Methodology (Chapter 3). The number of vehicles observed was multiplied by the frequency of each occurrence and summed. The total number of observations was then divided by this sum to obtain the sample proportion, as shown in Table 4-9.

Table 4-9: Calculation of sample yielding driver proportion

Number of vehicles failing $x - 1$	Number of vehicles observed x	Frequency observed $f(x)$	
0	1	61	61
1	2	26	52
2	3	17	51
3	4	6	24
4	5	5	25
5	6	3	18
6	7	4	28
7	8	0	0
8	9	0	0
9	10	0	0
10	11	0	0
11	12	1	12
12	13	0	0
13	14	1	14
Total		124	285
$p = 124/285 = 0.435$			

Table 4-10 shows the Chi-square evaluation test that was used to verify that the sample proportion was represented by a Geometric distribution. It was determined that the hypothesized frequency was acceptable with $\alpha = 0.05$. Figure 4-1 shows a histogram between the probability of each occurrence and the number of drivers failing to yield

before one driver yields to a pedestrian(s), using the data collected during the study and hypothesized data for the estimated proportion ($p = 0.435$).

Table 4-10: Chi-squared test for sample yielding vehicle proportion ($p = 0.435$)

Number of vehicles failing	Number of vehicles observed	Frequency (observed)	Proportion (observed)	Frequency (hypothesized)	Proportion (hypothesized)	Chi-squared value
$x - 1$	x	$f(x)$		$f(x)$		
0	1	61	0.492	54.0	0.435	0.92
1	2	26	0.210	30.5	0.246	0.66
2	3	17	0.137	17.2	0.139	0.00
3	4	6	0.048	9.7	0.078	1.43
4	5	5	0.040	5.5	0.044	0.16
5	6	3	0.024	3.1	0.025	
6	7	4	0.032	1.8	0.014	
7	8	0	0.000	1.0	0.008	
8	9	0	0.000	0.6	0.005	
9	10	0	0.000	0.3	0.003	
10	11	0	0.000	0.2	0.001	
11	12	1	0.008	0.1	0.001	
12	13	0	0.000	0.1	0.000	
13	14	1	0.008	0.0	0.000	
Total		$N = 124$	1.000	124.0	1.000	3.17
$\alpha = 0.05$					$\chi^2 = 9.49 > 3.17$	
Degrees of freedom = $5 - 1 = 4$					Acceptable	

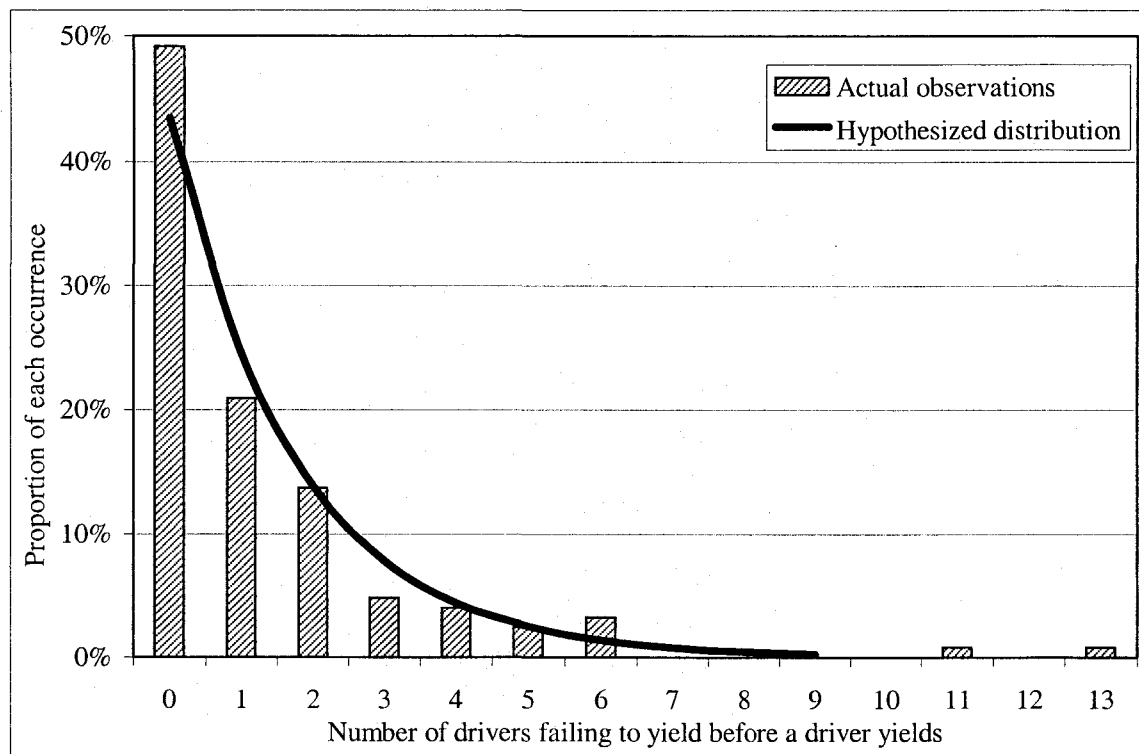


Figure 4-1: Proportion histogram of yielding drivers, actual and hypothesized values for sample yielding proportion ($p = 0.435$)

Once the sample proportion was calculated, a confidence interval could be constructed around the proportion estimate. Using the percentile bootstrap method presented in Efron and Tibshirani (1986), 100 simulated experiments were generated from the observed proportions in Table 4-10. Minitab 12 statistical software was used to generate random discrete numbers based on each of the fourteen “Number of vehicles observed” (number of trials) categories based on the field proportions to produce these simulated experiments. From this information, means were calculated for each simulation. The means were then sorted from lowest to highest, which produced one mean per percentile rank. To construct a two-tailed 95% confidence interval, the mean values at the 2.5th percentile and 97.5th percentile were necessary. The 2nd and 3rd

percentile means were interpolated to determine the upper bound, and the 97th and 98th percentile means for the lower bound. The inverse of the confidence interval means were then calculated to determine approximate lower and upper bounds for the sample proportion at 95% confidence. Table 4-11 shows the mean and proportion values calculated for several percentile ranks. The 95% confidence interval for the field data was estimated to be $0.378 \leq p \leq 0.509$, where 0.435 was the hypothesized sample proportion.

Table 4-11: Bootstrap percentiles generated using field data distribution

Percentile rank	Mean	Proportion
1	1.935	0.517
2.5	1.964	0.509
5	2.008	0.498
10	2.056	0.486
25	2.194	0.456
40	2.258	0.443
50	2.323	0.430
60	2.379	0.420
75	2.427	0.412
90	2.556	0.391
95	2.629	0.380
97.5	2.645	0.378
100	2.718	0.368

Next, the 64 vehicle/pedestrian yielding parameters were calculated and entered into the model (refer to Figures 3-5 and 3-6). Table 4-12 shows the default values for the variables as calculated from the geometries and/or corresponding speeds. Appendix III contains the default values calculated for each of the 64 yield condition parameters.

Table 4-12: Default values as calculated for vehicle/pedestrian interactions

Parameter	Calculation used to determine default value	Calculated value
H_{yp}	Width of all lanes (35 ft.) + “2 ft. safety” + “look ahead distance”	43 ft.
G_{yp}	(“2 ft. safety” + Width of all lanes (35 ft.) + “2 ft. safety”) / Pedestrian speed (3.0 ft/sec)	13 sec.
H_{npx}	“2 ft. safety” + Lane x width (11 to 13 ft.) + Previous lane(s) width + “2 ft. safety”	15-39 ft.
G_{npx}	Lane x width (12 ft.) / Pedestrian speed (3.0 ft/sec)	4.0 sec.
H_{py}	Crosswalk width (10 ft.) + distance from near side of crosswalk to yield bar (20 ft.)	30 ft.
G_{py}	Time to clear H_{py} @ 30 mph: 30 ft. / (30 mph * 1.47)	1 sec.
H_{pnx}	Crosswalk width (10 ft.) + distance from near side of crosswalk to yield bar (20 ft.)	30 ft.
G_{pnx}	Time to stop before yield bar @ 30 mph and 15.0 ft/s ² : (30 mph * 1.47) / (15.0 ft/s ²)	3 sec.

4.2 Model Calibration

Using the process described in Section 3.5.1, the following candidate global VISSIM parameters were chosen for calibration, followed by a brief definition (PTV 2003):

- Minimum Headway: Minimum distance necessary from the subject vehicle to the vehicle in front while stopped for a lane change
- Maximum Deceleration, Own Vehicle: Maximum deceleration rate of the subject vehicle that the subject vehicle will make a lane change
- Maximum Deceleration, Trailing Vehicle: Maximum deceleration rate of the ahead vehicle that the subject vehicle will make a lane change
- Accepted Deceleration, Own Vehicle: Accepted deceleration rate of the subject vehicle that the subject vehicle will make a lane change
- Accepted Deceleration, Trailing Vehicle: Accepted deceleration rate of the ahead vehicle that the subject vehicle will make a lane change

- Number of Observed Vehicles: Number of vehicles perceived by a subject vehicle
- Look-ahead Distance, Minimum: Minimum distance a subject vehicle will be able to predict movements of other vehicles
- Look-ahead Distance, Maximum: Maximum distance a subject vehicle will be able to predict movements of other vehicles
- Average Standstill Distance: Average distance between stopped vehicles
- Additive Part of the “Weidemann 74” Car-following Model: Factor involved in the computation of desired safety distance
- Multiplicative Part of the “Weidemann 74” Car-following Model: Factor involved in the computation of desired safety distance

The following vehicle/pedestrian yielding parameters were also included in the evaluation (refer to Figures 3-5 and 3-6): H_{yp} , G_{yp} , G_{npx} , G_{py} , and G_{pnx} . The yielding parameters H_{npx} , H_{py} , and H_{pnx} were not included as candidate calibration variables for the following reason. In VISSIM, the “Minimum headway” distance defined for yield behaviors (not the “Minimum Headway” global parameter setting) is only applicable when the vehicle or pedestrian yielding had already yielded. This distance is defined in VISSIM as the length of the conflict area (see Figure 3-6). For the three parameters not included, this distance should, in theory, be fixed. However, H_{yp} would represent the minimum distance between an approaching pedestrian and the curb that a yielding vehicle would no longer continue to yield. In other words, it is the distance that yielding vehicles would look upstream of the pedestrian crossing for other approaching

pedestrians while already yielding. This distance cannot be easily observed. Therefore, it was considered in the list of candidate parameters to calibrate.

Of the sixteen candidate parameters chosen for the process, six produced significant differences in travel times when minimum and maximum values were varied. Travel times for all four segments were generated for each combination. These travel time differences were averaged over ten runs for each parameter combination. Differences were recorded between these values and those calculated from travel times generated using default values for all model parameters. Candidate parameter combinations that produced travel time differences in each segment larger than 2 seconds were noted. Sets which had at least four incidents where travel times had changed 2 seconds or more were determined to be ideal for the calibration process. One interesting finding was that these travel times were not necessarily linearly related. Also, each segment was affected differently by a change in each parameter. The chosen calibration parameters, default values, feasible minimum, maximum, and the difference from the default travel times are shown in Table 4-13.

The selected parameter sets were then varied with an experimental design as described in Major Calibration (Section 3.5.2). Each simulation run as determined from the experimental design was executed 20 times, with each run using a different random seed value. A total of 79 different parameter combinations were executed 20 times each, yielding a total of 1,580 simulation runs.

Table 4-13: Parameters selected for calibration, and difference from default travel times

ID	Parameter	Unit	Default	Min.	Max.	Travel time change using minimum				Travel time change using maximum			
						NB		SB		NB		SB	
						U-x	x-H	H-x	x-U	U-x	x-H	H-x	x-U
x_1	Number of observed vehicles	each	2	1	4	-4.0	-6.5	-5.3	-4.1	-0.3	1.3	-0.4	-0.6
x_2	Average standstill distance	ft.	6.56	3	15	-1.8	-2.5	-3.3	-2.2	3.8	5.6	-0.8	-0.7
x_3	Additive part of Weidemann 74	---	2	0.5	3	-2.6	-2.2	-3.8	-2.3	2.4	3.0	-1.4	-1.4
x_4	Multiplicative part of Weidemann 74	---	3	0.5	4	-1.8	-1.8	-3.2	-2.9	-0.3	1.6	-3.3	-2.5
x_5	Yielding vehicle yield to pedestrian, minimum gap	sec.	13	9	18	-5.1	-2.2	-5.4	-3.6	3.1	0.2	-0.2	0.1
x_6	Yielding vehicle yield to pedestrian, minimum headway	ft.	43	40	48	-6.0	-4.3	-7.6	-4.7	1.7	1.6	-1.9	-1.4
U-x: University Road to the crosswalk (travel time of 23.2 seconds using default values)													
x-H: Crosswalk to Harmon Avenue (travel time of 54.2 seconds using default values)													
H-x: Harmon Avenue to the crosswalk (travel time of 34.1 seconds using default values)													
x-U: Crosswalk to University Road (travel time of 29.4 seconds using default values)													

After averaging the four travel time values generated from the 20 iterative runs for each experiment combination, linear regression surface functions were developed for each of the four output travel times using Minitab 12 statistical analysis software. The six individual parameters were treated as the predictor, or independent variables (x_1 through x_6). Each travel time was the response, or dependent variable (y_1 through y_4). Table 4-14 shows the linear regression surface functions for each of the four travel times. Table 4-15 includes the regression analysis and ANOVA results on each model developed.

Table 4-14: Linear regression surface functions for each calibration travel time

Travel time section	Surface function equation
University to crosswalk	$y_1 = -37.1256 + 0.9276 x_1 + 0.1449 x_2 + 1.585 x_3 + 0.858 x_4 + 0.8306 x_5 + 0.8575 x_6$
Crosswalk to Harmon	$y_2 = 13.3595 + 1.0116 x_1 + 0.317 x_2 + 1.8803 x_3 + 1.0318 x_4 + 0.1137 x_5 + 0.4934 x_6$
Harmon to crosswalk	$y_3 = -31.3398 + 0.9461 x_1 + 0.2039 x_2 + 1.9256 x_3 + 1.0932 x_4 + 0.7447 x_5 + 0.9048 x_6$
Crosswalk to University	$y_4 = -5.00916 + 0.57231 x_1 + 0.1592 x_2 + 1.22869 x_3 + 0.68678 x_4 + 0.33222 x_5 + 0.4164 x_6$

Table 4-15: Regression analysis and ANOVA results on surface functions

		University to crosswalk	Crosswalk to Harmon	Harmon to crosswalk	Crosswalk to University
Adjusted R^2		82.6%	82.1%	75.2%	77.0%
F		62.79	60.65	40.42	44.64
P		< 0.001	< 0.001	< 0.001	< 0.001
T	Constant	-10.44	5	-6.82	-2.16
	x_1	4.62	6.7	3.64	4.37
	x_2	2.9	8.44	3.15	4.88
	x_3	6.54	10.33	6.14	7.78
	x_4	5.01	8.02	4.94	6.15
	x_5	12.31	2.24	8.53	7.55
	x_6	11.34	8.69	9.25	8.45
P	Constant	< 0.001	< 0.001	< 0.001	0.034
	x_1	< 0.001	< 0.001	0.001	< 0.001
	x_2	0.005	< 0.001	0.002	< 0.001
	x_3	< 0.001	< 0.001	< 0.001	< 0.001
	x_4	< 0.001	< 0.001	< 0.001	< 0.001
	x_5	< 0.001	0.028	< 0.001	< 0.001
	x_6	< 0.001	< 0.001	< 0.001	< 0.001

These four surface models were then used to estimate different value combinations of the six calibration parameters that would produce the average travel times as collected from the field observations. The optimization goal was to minimize the sum of square

errors between actual and predicted travel times for each segment. Equation 4-1 explains this concept mathematically.

$$\text{Minimize } \sum_{i=1}^4 (y_i - \hat{y}_i)^2 \quad \dots\dots\dots (4-1)$$

subject to :

$$\hat{y}_1 = -37.1256 + 0.9276x_1 + 0.1449x_2 + 1.585x_3 + 0.858x_4 + 0.8306x_5 + 0.8575x_6$$

$$\hat{y}_2 = 13.3595 + 1.0116x_1 + 0.317x_2 + 1.8803x_3 + 1.0318x_4 + 0.1137x_5 + 0.4934x_6$$

$$\hat{y}_3 = -31.3398 + 0.9461x_1 + 0.2039x_2 + 1.9256x_3 + 1.0932x_4 + 0.7447x_5 + 0.9048x_6$$

$$\hat{y}_4 = -5.0092 + 0.5723x_1 + 0.1592x_2 + 1.2287x_3 + 0.6868x_4 + 0.3322x_5 + 0.4164x_6$$

$$x_1 = \text{integer value}$$

$$L_n \leq x_n \leq U_n$$

where :

i = number of travel time segments to calibrate (1 to 4)

y_i = actual average travel time observed in field

$\hat{y}_i$ = predicted average travel time as output by simulation model

n = number of calibration parameters (refer to Table 4-12)

x_n = value of calibration parameter n

L_n = feasible lower limit value of calibration parameter n

U_n = feasible upper limit value of calibration parameter n

The optimization equation was translated into Microsoft Excel, and What'sBest! 7.0, developed by Lindo Systems, Inc. was used to conduct minimization of the objective function. The WhatsBest! software utilized a procedure that involves converting non-linear optimization models into smaller linear programming problems. Although no

method can guarantee global minimization, the software used a “branch and bound” algorithm to converge on a solution after 337 trials. The optimization model was executed four additional times using different starting values for each parameter. Each WhatsBest! run converged on the same optimal solution for all parameters. Since the purpose of this procedure is to estimate parameter values which will produce simulated travel time averages that are reasonably close to actual times, sensitivity analysis was performed on the optimization model to generate additional candidate parameter sets. Strategies employed included holding one or two variables at the default values and allowing the program to adjust the others. A technique that was used for creating sets 1-6, 1-7, and 1-8 was to generate optimal solutions for all six parameters with respect to two of the four travel times, and then re-run the optimization by only adjusting the two minimum gap and headway parameters (5 and 6) to obtain the minimum error for the other two travel times. The sum-of-squares errors for most parameter sets were only slightly larger than the optimized solution. Set 1-7 had a higher sum-of-squares error, because it was optimized for equations y_1 and y_2 only. In all, eight sets were derived using the above four surface functions and the optimization equation, as shown in Table 4-16.

Table 4-16: Candidate parameter set values as determined from surface function minimization (refer to Table 4-4)

Set number	Calibration parameters						Predicted travel times				SSE
	x_1	x_2	x_3	x_4	x_5	x_6	y_1	y_2	y_3	y_4	
1-1	3	5.64	0.62	4	9	48	19.5	48.2	28.3	24.1	0.593
1-2	2	5.54	1.15	4	9	48	19.4	48.1	28.4	24.2	0.638
1-3	3	6.56	0.50	3.95	9.1	47.9	19.5	48.2	28.3	24.1	0.599
1-4	3	5.27	2	1.59	9	48	19.6	48.2	28.3	24.1	0.636
1-5	3	5.57	1.18	3	9	48	19.5	48.2	28.3	24.1	0.609
1-6	3	5.64	0.62	4	9	48	19.5	48.2	28.3	24.1	0.600
1-7	1	15.00	2.42	3.07	15.2	40	19.3	48.3	28.2	24.7	1.958
1-8	2	6.56	2	3	9.9	46.3	19.4	48.3	28.3	24.3	0.863

After entering all eight candidate parameter sets selected and running each simulation 20 times, it was discovered that the predicted average travel times differed from the new simulation average travel times produced. Most of these times differed by less than 1.5 seconds. The logical conclusion from this observation was that travel times were not linearly related to one or all of the calibration parameters. The lower R-squared values for each of the surface functions also suggested that the relationship was not linear. However, the regression models were determined to be sufficient for producing travel times that were relatively close to those simulated, since the residual values generally differed only slightly near actual travel time averages. The residual values for each linear regression model are plotted in Figures 4-2 through 4-5. Most of these residual values were below 6 seconds in either direction.

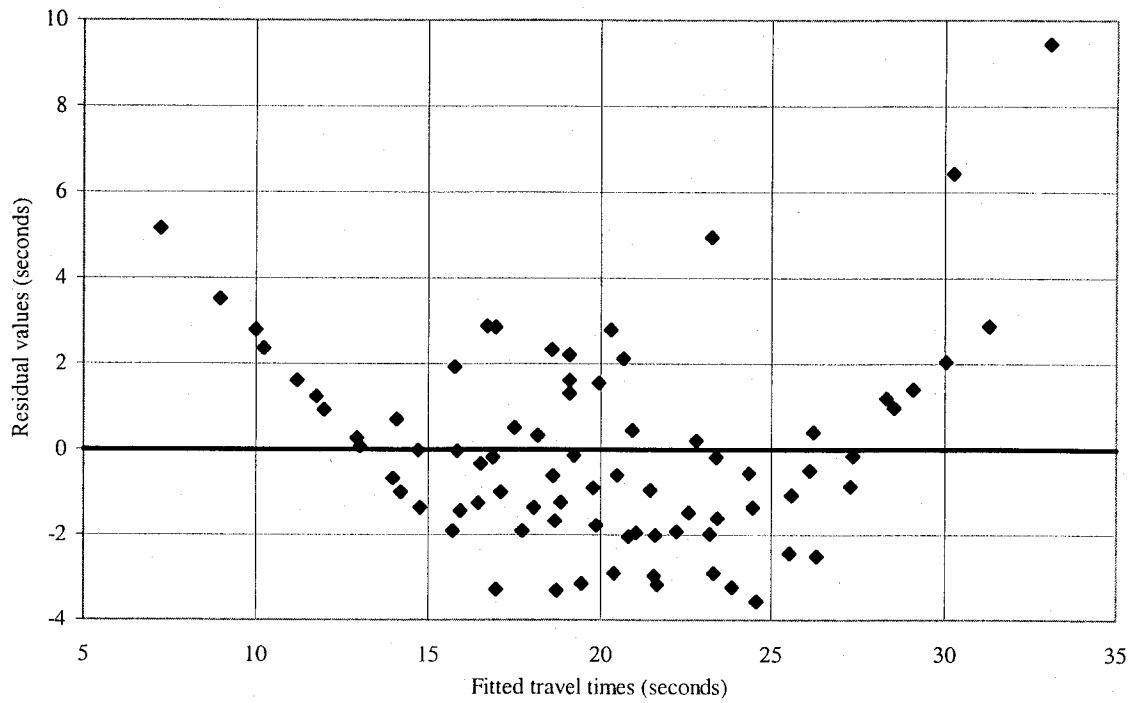


Figure 4-2: Residual travel times versus fitted from University Road to the crosswalk

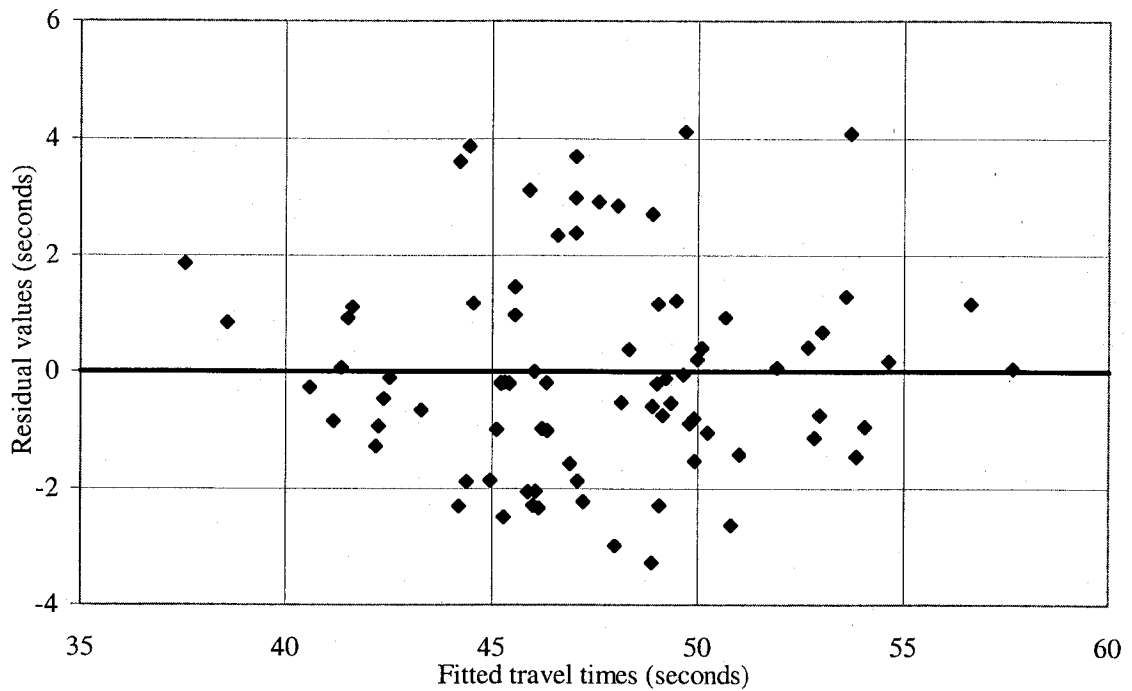


Figure 4-3: Residual travel times versus fitted from the crosswalk to Harmon Avenue

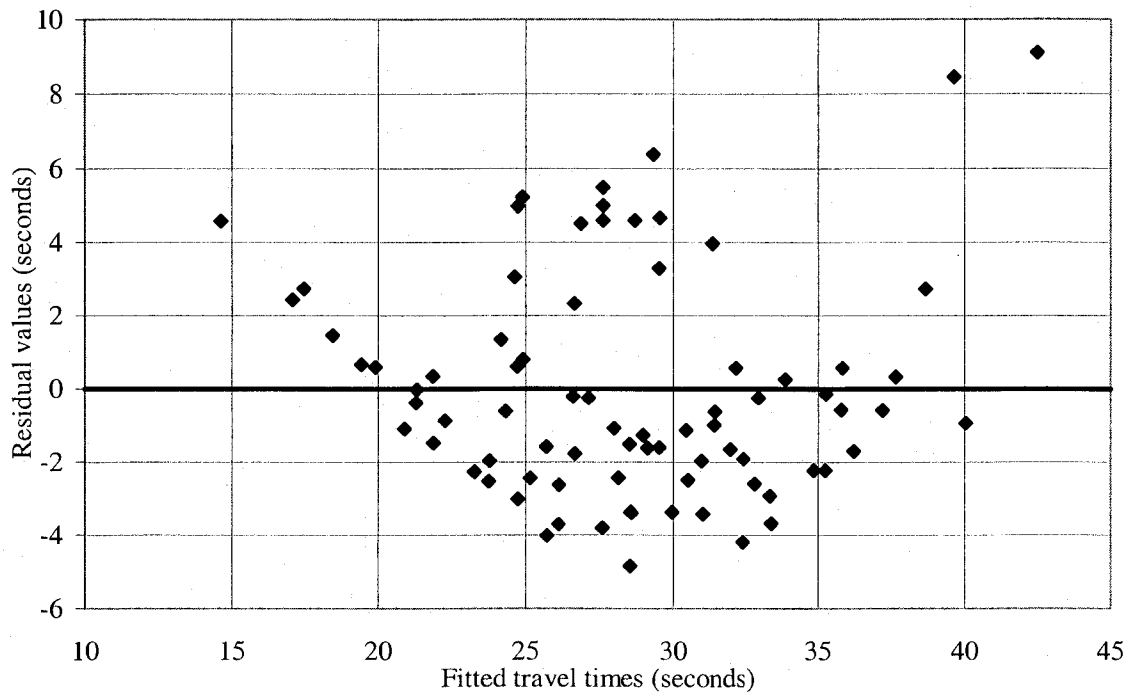


Figure 4-4: Residual travel times versus fitted from Harmon Avenue to the crosswalk

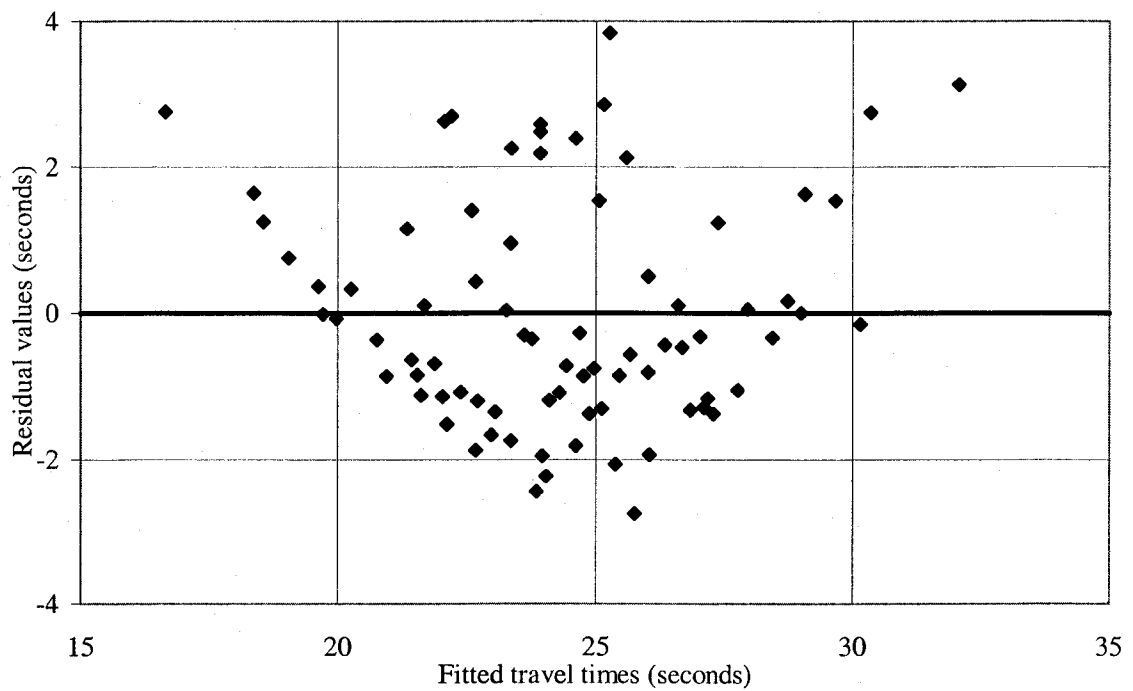


Figure 4-5: Residual travel times versus fitted from the crosswalk to University Road

The eight candidate parameter sets were then subjected to visual inspection. This process was to ensure proper model functioning. For instance, if a particular parameter set caused vehicles to not yield for pedestrians at all, or at inappropriate instances, this would be considered unfavorable operation. All of the eight sets were determined to be visually satisfactory.

Next, each parameter set was compared using the six test statistics as outlined by Hourdakis et al. (2003). Ideally, each test statistic should be in the favorable range, but this may not be possible for all test statistics and for all four travel times. The goal in this process was to select one or two sets that produce the most favorable test statistics, but with the fewest test statistic values which have unreasonable values. For every set being evaluated, the raw travel times for simulated vehicles were extracted and separated into the four respective travel times. Next, the occurrences for each second of travel time were compared between the simulated values and the actual values. In this way, both the simulated and actual travel time distributions for individual vehicles could be compared. The individual time categories were then expanded in even increments so that the actual travel times (which there were many fewer observations than the simulation provided) would have bins that were never zero, and the first and last bins had at least 5 observations. The mean travel times for each actual and simulated bin were compared with the mean and standard deviation for the actual and simulated vehicle travel times.

For example, six bins ($N = 6$) were created from the actual travel time raw data, and the mean values for actual and simulated travel times (in seconds) for set 1-3 were calculated for each bin:

- < 12 :   actual = 9.0   simulated = 10.5
- 12 – 15.9:   actual = 12.9   simulated = 13.5
- 16 – 19.9:   actual = 17.4   simulated = 17.7
- 20 – 23.9:   actual = 22.8   simulated = 22.0
- 24 – 27.9:   actual = 25.0   simulated = 25.8
- ≥ 28 :   actual = 34.9   simulated = 36.4

The mean actual travel time from University Road to the crosswalk calculated from the bins was 20.3 seconds, and the standard deviation was 9.3 seconds. For set 1-3, bin mean simulated travel time was 21.0 seconds, and the standard deviation was 9.4 seconds. The six test statistics were calculated to be:

- *RMSP*:   7.7%
- *r*:   0.996
- *U*:   0.02
- *U_m*:   0.42
- *U_s*:   0.00
- *U_c*:   0.69

In the above example, *U_m* and *U_c* would not be in the Hourdakis et al. (2003) favorable ranges. Detailed analysis as to how each test statistic performed will be addressed later in this section.

In addition to visual evaluation and test statistic evaluation, the occurrences of raw actual and simulated vehicle travel times were plotted as a proportion histogram to

ascertain whether the two distributions were similar. Figure 4-6 shows the proportion histogram for actual and simulated travel times from University Road to the crosswalk as generated from set 1-3. After all of these considerations, set 1-3 was chosen for further analysis. Table 4-17 shows the test statistic results for all four travel times using set 1-3.

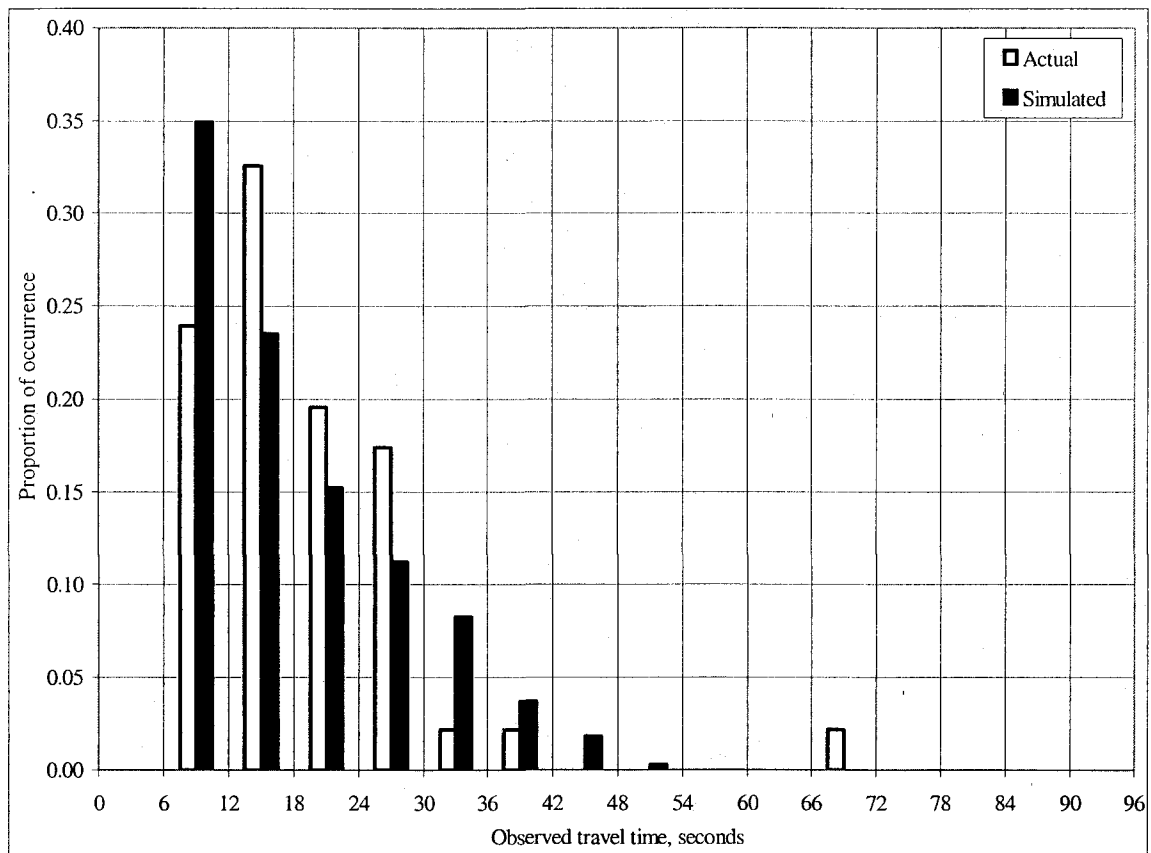


Figure 4-6: Proportion histogram for travel times from University road to the crosswalk generated from set 1-3

Table 4-17: Test statistic results for selected set 1-3

	<i>RMSP</i>	<i>r</i>	<i>U</i>	<i>U_m</i>	<i>U_s</i>	<i>U_c</i>
University to crosswalk	7.7%	0.996	0.02	0.42	0.00	0.69
Crosswalk to Harmon	4.3%	0.998	0.02	0.43	0.05	0.59
Harmon to crosswalk	11.1%	0.976	0.07	0.19	0.67	0.34
Crosswalk to University	11.7%	0.975	0.08	0.16	0.50	0.46

The minor calibration of set 1-3 was completed in two phases. First, the vehicle/pedestrian yielding parameters H_{yp} , G_{yp} , and G_{npx} were each adjusted individually, with the intention of obtaining simulation sample mean, standard deviation, and overall proportion distributions (Figure 4-2, for example) that were closer to those of the actual field measurements at each of the four travel time segments. As a result, these adjustments also made the test statistics slightly more favorable. Ideally, the intent of this calibration was to produce favorable values for the greatest number of test statistics.

Once it was determined that these values could not be adjusted any further without causing other measures to become more unfavorable, reduced speed areas were placed in the simulation model using field observations to more adequately reflect speed reductions caused by driveways and turning/weaving vehicles. Most of these were in the rightmost lanes. These reduced speed areas used a slower speed distribution than the original free flow speed distribution, as discussed in Data collection (Section 4.1, Table 4-8) for speed studies earlier in this chapter. The average speed for northbound Maryland Parkway upstream of the crosswalk was measured at 25.8 mph. The main intent of introducing these reduced speeds was to make the simulated travel time proportion distributions match more closely with the actual proportion distributions. Consequentially, the test statistics improved significantly for each travel time segment. However, not every test

statistic was able to be adjusted within the favorable range suggested by Hourdakis et al. (2003). The final result reflects the subjective minor calibration adjustments made to produce the best attainable means, standard deviations, proportion distributions, and test statistics. The final proportion distribution and test statistic results, as well as possible discrepancy causes, are addressed in greater detail at the end of this section. Table 4-18 shows the initial values, set 1-3 optimized values, and final adjustments made to the local vehicle/pedestrian yielding parameters. Parameters are only included in Table 4-18 if the initial values have changed in the calibration process. The initial values for all 64 parameters are included in Appendix III.

Table 4-18: Vehicle/pedestrian yielding parameters; initial values, set 1-3 optimized values, and final adjustments

Parameter	Direction (vehicular/ pedestrian)	Default (ft. or sec.)	Set 1-3 optimized (ft. or sec.)	Final adjustment (ft. or sec.)
H_{yp}	NB/EB	43.0	47.9	44.9
G_{yp}	NB/EB	13.0	9.1	8.0
G_{np1}	NB/EB	4.0	4.0	3.0
G_{np2}	NB/EB	4.0	4.0	3.0
G_{np3}	NB/EB	4.0	4.0	3.0
H_{yp}	NB/WB	43.0	47.9	44.9
G_{yp}	NB/WB	13.0	9.1	8.0
G_{np1}	NB/WB	4.0	44.0	3.0
G_{np2}	NB/WB	4.0	4.0	3.0
G_{np3}	NB/WB	4.0	4.0	3.0
H_{yp}	SB/EB	43.0	47.9	44.9
G_{yp}	SB/EB	13.0	9.1	7.0
G_{np1}	SB/EB	4.0	4.0	3.0
G_{np2}	SB/EB	4.0	4.0	3.0
G_{np3}	SB/EB	4.0	4.0	3.0
H_{yp}	SB/WB	43.0	47.9	44.9
G_{yp}	SB/WB	13.0	9.1	7.0
G_{np1}	SB/WB	4.0	4.0	3.0
G_{np2}	SB/WB	4.0	4.0	3.0
G_{np3}	SB/WB	4.0	4.0	3.0

Furthermore, the following segments and lanes were assigned reduced speed areas, where the free flow speed distribution (average of 33.7 mph) was replaced by a field running speed distribution (average of 25.8 mph):

- University to the crosswalk (northbound upstream):

Lane 1, from 300 feet north of University Road to the crosswalk yield bar

Lane 2, from 300 feet north of University Road to the crosswalk yield bar

- Crosswalk to Harmon (northbound downstream):
Lane 1, from 150 feet north of the crosswalk yield bar to Harmon Avenue
Lane 2, from 400 feet north of the crosswalk yield bar to Harmon Avenue
- Harmon to the crosswalk (southbound upstream):
Lane 1, from Harmon Avenue to the crosswalk yield bar
Lane 2, from 300 feet south of Harmon Avenue to the crosswalk yield bar
- Crosswalk to University (southbound downstream):
Lane 1, from 50 feet north of the crosswalk yield bar to University Road
Lane 2, from 50 feet north of the crosswalk yield bar to University Road

Although most of the travel time segments were adjusted to reflect means, standard deviations, or proportion distributions similar to those observed in the field, not every test statistic was capable of being in the acceptable range defined by Hourdakis et al. (2003). Possible causes are explained in greater detail later in this section. The original test statistics from set 1-3, intermediate adjustment steps, and the final adjusted test statistics at each segment are presented in Tables 4-19 through 4-22. Figures 4-7 through 4-10 show the proportion histograms for actual, uncalibrated simulation, and calibrated simulation travel times.

Table 4-19: Test statistics for University Road to the crosswalk segment for uncalibrated, intermediate, and final adjusted models

University to crosswalk						
	$RMSP$	r	U	U_m	U_s	U_c
Uncalibrated (default)	13.5%	0.973	0.09	0.26	0.51	0.38
Set 1-3 (optimized)	7.7%	0.996	0.02	0.42	0.00	0.69
Step 1	12.0%	0.994	0.03	0.37	0.09	0.66
Step 2	7.8%	0.995	0.02	0.41	0.01	0.69
Step 2a	7.8%	0.995	0.02	0.39	0.02	0.71
Step 2b	11.4%	0.994	0.03	0.36	0.10	0.67
Step 2c (final model)	8.9%	0.994	0.03	0.41	0.01	0.70

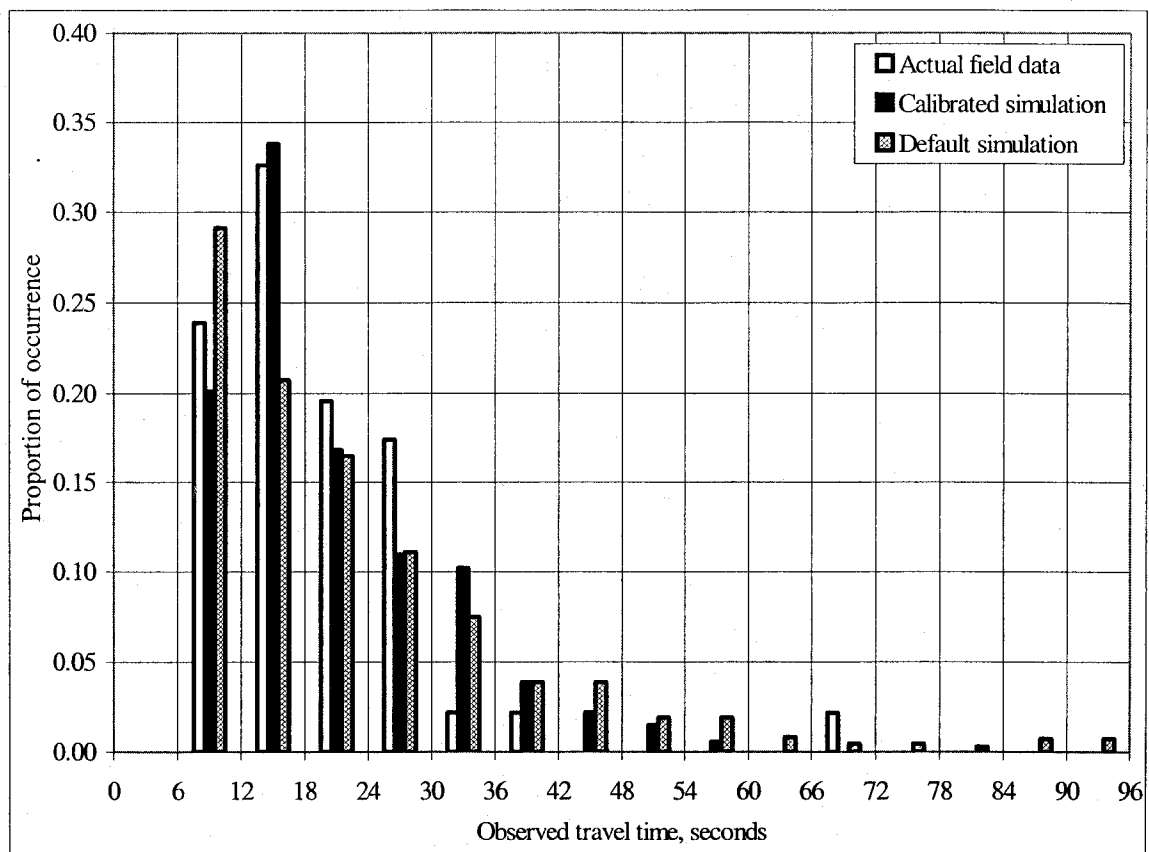


Figure 4-7: Proportion histogram of travel times for actual, uncalibrated, and calibrated model from University Road to the crosswalk

Table 4-20: Test statistics for the crosswalk to Harmon Avenue segment for uncalibrated, intermediate, and final adjusted models

Crosswalk to Harmon						
	<i>RMSP</i>	<i>r</i>	<i>U</i>	<i>U_m</i>	<i>U_s</i>	<i>U_c</i>
Uncalibrated (default)	4.4%	0.998	0.02	0.52	0.07	0.47
Set 1-3 (optimized)	4.3%	0.998	0.02	0.43	0.05	0.59
Step 1	4.9%	0.998	0.02	0.47	0.02	0.58
Step 2	5.1%	0.998	0.02	0.49	0.02	0.56
Step 2a	5.0%	0.998	0.02	0.46	0.00	0.60
Step 2b	4.7%	0.998	0.02	0.47	0.04	0.56
Step 2c (final model)	4.4%	0.997	0.02	0.32	0.10	0.67

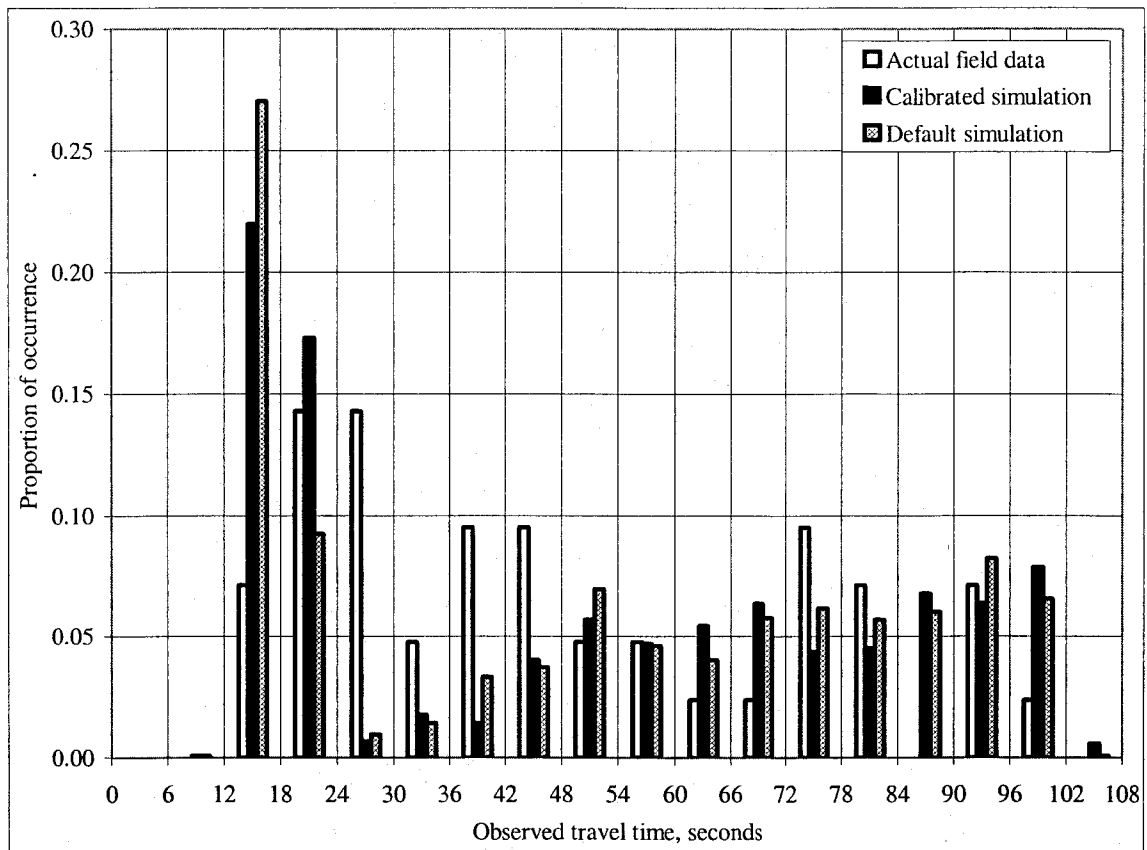


Figure 4-8: Proportion histogram of travel times for actual, uncalibrated, and calibrated model from the crosswalk to Harmon Avenue

Table 4-21: Test statistics for Harmon Avenue to the crosswalk segment for uncalibrated, intermediate, and final adjusted models

Harmon to crosswalk						
	$RMSP$	r	U	U_m	U_s	U_c
Uncalibrated (default)	20.6%	0.950	0.13	0.21	0.73	0.26
Set 1-3 (optimized)	11.1%	0.976	0.07	0.19	0.67	0.34
Step 1	12.1%	0.977	0.08	0.21	0.70	0.29
Step 2	14.2%	0.968	0.09	0.20	0.70	0.30
Step 2a	13.5%	0.968	0.09	0.18	0.70	0.33
Step 2b	11.5%	0.973	0.07	0.22	0.60	0.38
Step 2c (final model)	4.2%	0.995	0.03	0.18	0.45	0.58

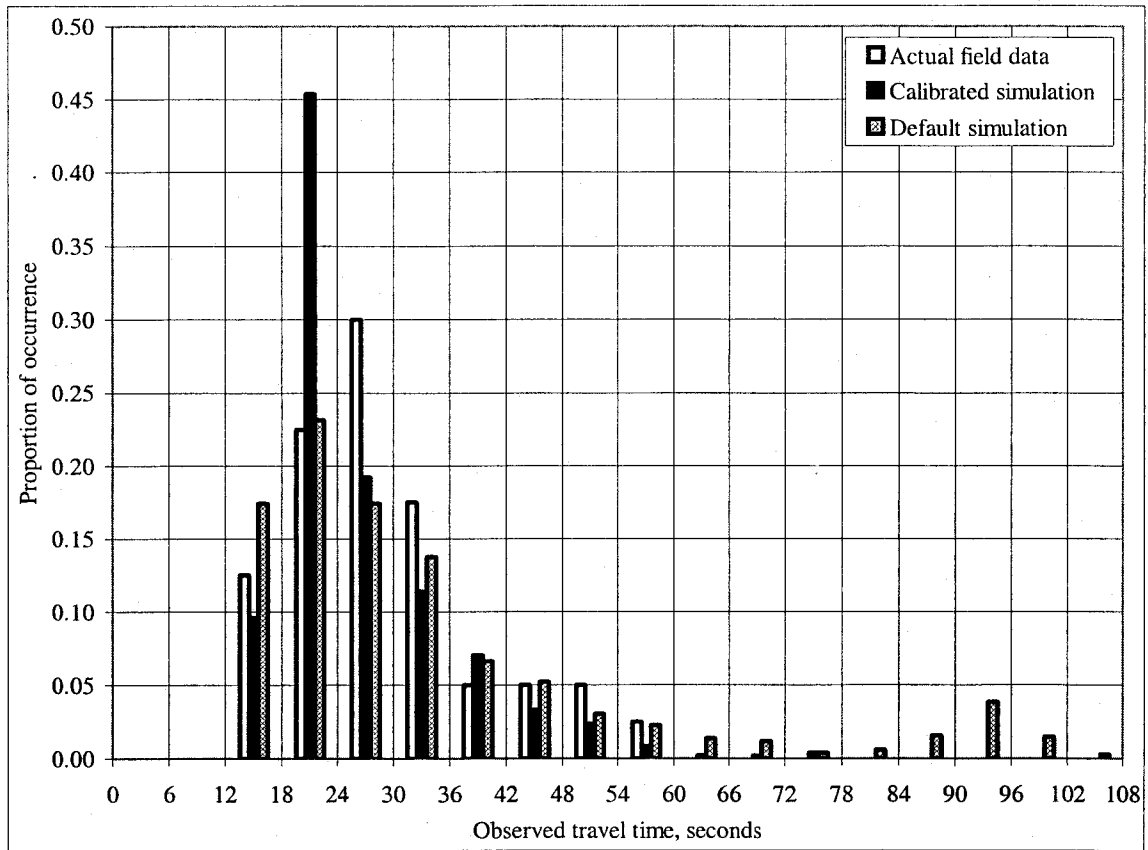


Figure 4-9: Proportion histogram of travel times for actual, uncalibrated, and calibrated model from Harmon Avenue to the crosswalk

Table 4-22: Test statistics for the crosswalk to University Road segment for uncalibrated, intermediate, and final adjusted models

Crosswalk to University						
	$RMSP$	r	U	U_m	U_s	U_c
Uncalibrated (default)	12.1%	0.973	0.08	0.18	0.47	0.46
Set 1-3 (optimized)	11.7%	0.975	0.08	0.16	0.50	0.46
Step 1	11.1%	0.975	0.08	0.14	0.48	0.50
Step 2	10.6%	0.978	0.07	0.15	0.49	0.49
Step 2a	11.5%	0.976	0.08	0.16	0.47	0.48
Step 2b	11.8%	0.975	0.08	0.17	0.49	0.47
Step 2c (final model)	12.9%	0.972	0.09	0.16	0.50	0.47

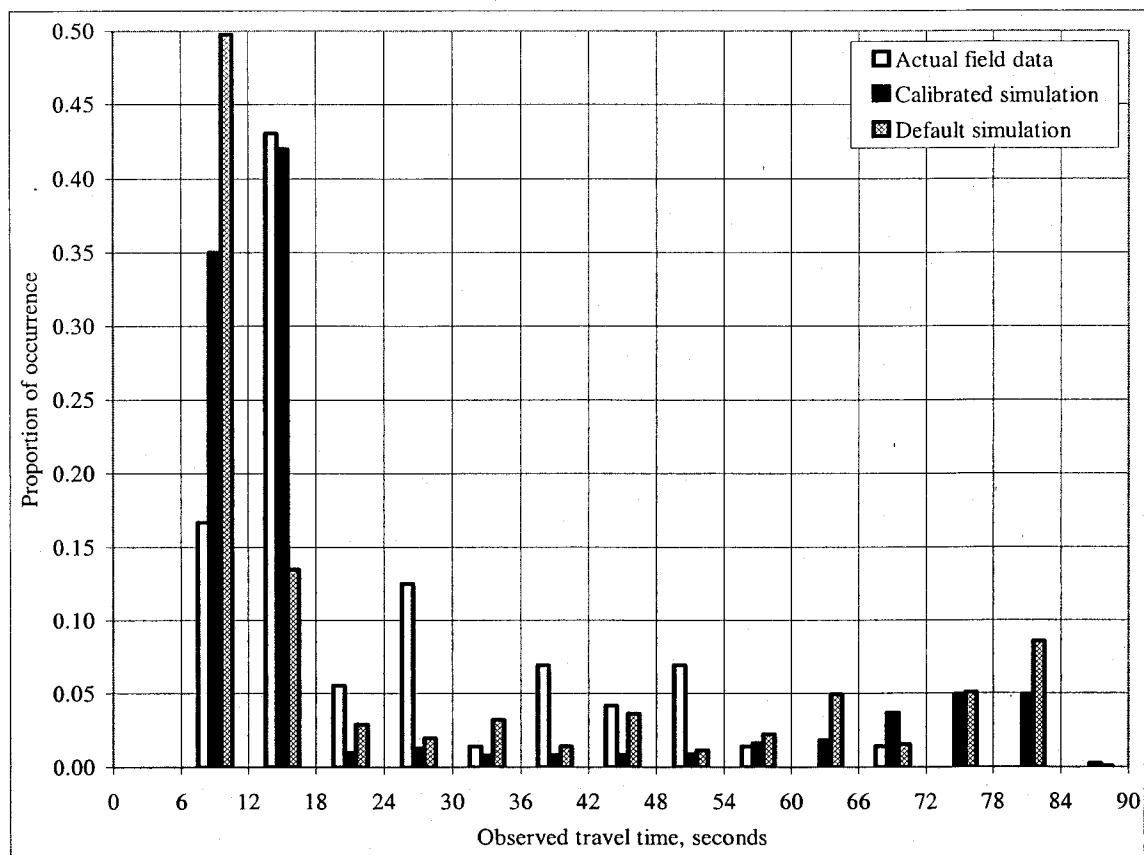


Figure 4-10: Proportion histogram of travel times for actual, uncalibrated, and calibrated model from the crosswalk to University Road

Attempts were made to adjust all local parameters so that each test statistic was in the acceptable range as discussed in Hourdakis et al. (2003). The *RMSP*, *r* correlation coefficient, and *U* statistic for each segment were in the acceptable range. However, the U_m values for the University to crosswalk and Crosswalk to Harmon segments (northbound traffic) were higher than the values recommended. For both of these segments, it was desired to reduce the U_m value as best possible without producing unfavorable U_s values. This coincidentally also made the U_c more favorable. These adjustments also produced better *RMSP* and *r* statistics, and moved the proportion distribution to be better aligned visually with the actual travel times. Adjustments made to local vehicle/pedestrian yielding behavior parameters for northbound traffic had a significant impact on the test statistics, averages, standard deviations, and proportion distributions of the two southbound segments.

Because of this interaction, certain measures that were acceptable on the southbound side were altered to become more unfavorable. Again, the *RMSP*, *r* correlation coefficient, and *U* statistic for the southbound segments were in the acceptable range. Adjustments were made to reduce the U_m statistic for both segments, which resulted in more favorable U_m values and a lower U_s for the Harmon to crosswalk segment, but a slightly higher U_s for the Crosswalk to University segment. These adjustments also yielded a more favorable U_c statistic. The two proportion distributions were much better aligned visually with the actual travel times than those produced from the set 1-3 parameter values.

Set 1-3 was chosen because it produced the greatest number of test statistics within the Hourdakis et al. (2003) recommendations. The parameter was adjusted for

southbound traffic in Step 1 which slightly improved test statistics for the southbound segments, but caused a significant increase in the northbound *RMSP* values. In Step 2, the northbound values for G_{yp} were altered, which significantly reduced the *RMSP* value on the northbound upstream segment while causing slight degradations in various other test statistics on the other segments. The *RMSP* for the southbound downstream segment increased significantly as a result, so adjustments for H_{yp} on all segments were introduced in Step 2a. This produced slightly more favorable test statistics for most segments. Step 2b involved implementing adjustments for all G_{npx} variables in each direction and lane. This produced better test statistics for most segments except for the northbound upstream. At this point, most U_m statistics were indicating mean values that were slightly different than the actual mean values, but U_s values were indicating that the simulation was producing significantly shorter travel times. Reduced speed areas were implemented in Step 2c to adjust these travel times to better reflect those shown on the proportion histograms. The result of this was a significant improvement for most *RMSP*, U_s , and U_c . The tradeoff was that a couple test statistics were made slightly more unfavorable by this action. However, the significant improvements in other test statistics outweighed these slight degradations. At this point, any other adjustments that were made by alterations in local parameters produced significantly more unfavorable test statistics on all segments. Therefore, Step 2c was determined to be the end of minor calibration.

One reason that the two U_m statistics for the northbound segments were not less than 0.1 was because of how the actual and calibrated model differed in the proportion of observations between 24 and 36 seconds for the University to crosswalk, and between 30 to 48 seconds for the crosswalk to Harmon segment. The number of actual observations

was much smaller than those produced by the simulation. As a result, bins were created to represent travel time ranges. Since the actual number of observations was smaller, the variability on the bin means for the actual travel times had a higher variance than those of the simulated travel time bins. For the two U_s statistics on the southbound segments, the discrepancies could exist because of the same reason. Since the actual bin standard deviations would be larger, the actual bin means could vary more than those of the simulated travel times. The U_c statistics for all travel time segments could not be adjusted to be equal to or greater than 0.9, as suggested by Hourdakis et al. (2003). Again, one likely reason for this was the discrepancy in the number of samples.

4.3 Model Validation

Model validation was accomplished through the methods as outlined in the Methodology (Chapter 3). The measure of effectiveness by which the final model was validated was the distribution of vehicle queues at the crosswalk. A vehicle queue survey was conducted at the crosswalk on November 17, 2004 (Wednesday) on the middle through lanes of northbound and southbound Maryland Parkway between the hours of 11:00 AM and 1:00 PM.

The actual and simulated number of queued vehicles in the middle through lane for northbound and southbound Maryland Parkway were compared using a Chi-squared goodness-of-fit test. For the northbound vehicle queues at the crosswalk, only the second hour of actual observations were used, but both hours for the southbound observations were averaged. After further inspection it was discovered that queue counts during the first hour of observations for southbound traffic did not include a full hour of

observations, and so this first hour was discarded from analysis. For northbound and southbound vehicle queues, the actual counts could not be rejected as being from the hypothesized distribution (simulated queues) with $\alpha = 0.05$. Table 4-23 shows the actual and simulated northbound vehicle queue frequency, with the Chi-squared values calculated and evaluated. Table 4-24 shows the same for southbound vehicle queues. Figures 4-11 and 4-12 show the actual and simulated vehicle queue distributions graphically.

Table 4-23: Actual and simulated northbound vehicle queue Chi-squared evaluation

Vehicles queued	Observed frequency	Simulated frequency	Chi-squared value
1	97	95	0.04
2	19	13	2.77
3	3	4	0.25
4	1	3	4.17
5	0	2	
6	0	1	
	Total: 120	Total: 118	Total: 7.23
$\alpha = 0.05$			$\chi^2 = 7.81 > 7.23$
Degrees of freedom = 4 - 1 = 3			Acceptable

Table 4-24: Actual and simulated southbound vehicle queue Chi-squared evaluation

Vehicles queued	Observed frequency	Simulated frequency	Chi-squared value
1	91	92	0.01
2	11	12	0.08
3	8	7	0.14
4	7	3	3.57
5	2	2	
6	2	1	
7	1	1	
	Total: 122	Total: 118	Total: 3.81
$\alpha = 0.05$			$\chi^2 = 7.81 > 3.81$
Degrees of freedom = 4 - 1 = 3			Acceptable

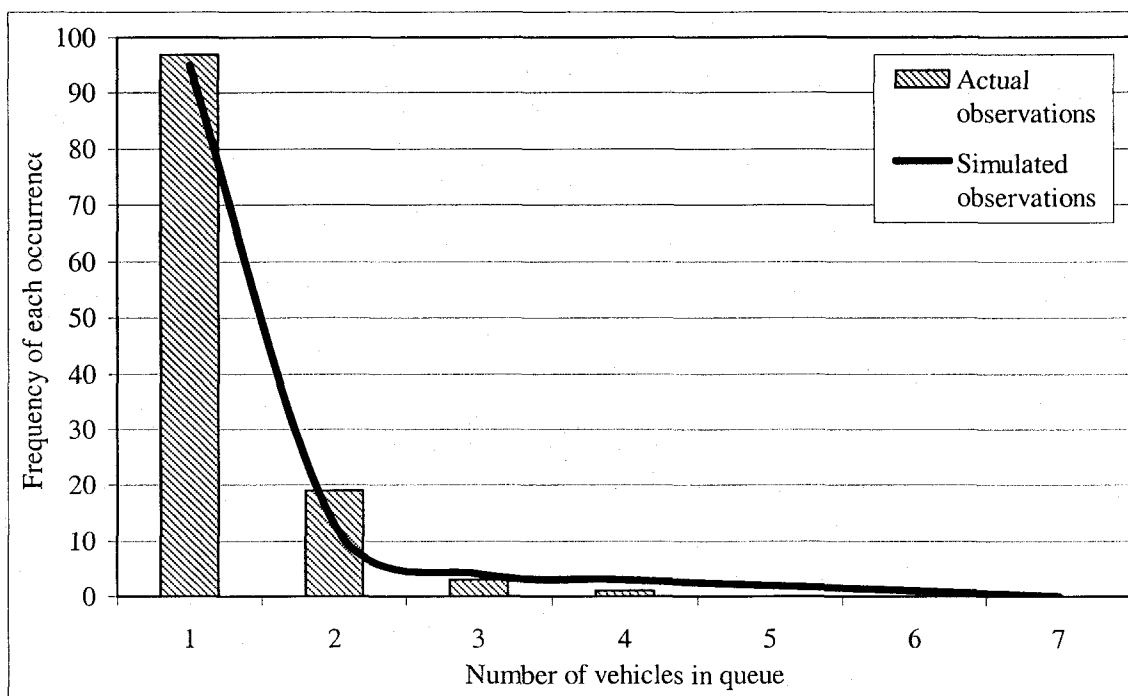


Figure 4-11: Actual and simulated northbound vehicle queue distributions

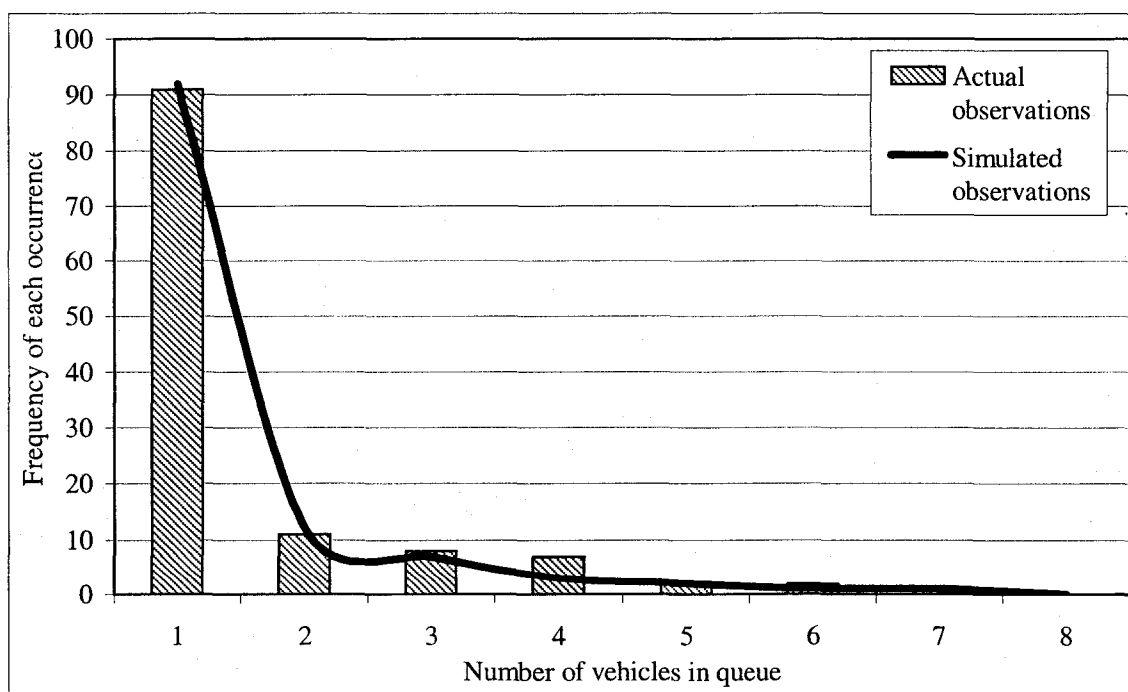


Figure 4-12: Actual and simulated southbound vehicle queue distributions

Queue frequency distributions were compared against those generated by the simulation model. For the northbound queues, the observer did not record queues for the first four minutes of the study period. Therefore, the first hour of the northbound queue study was discarded from validation analysis. The simulation was run 5 times to generate average simulated queue lengths. These queue lengths were then converted as explained previously, to generate an average queue frequency distribution for simulated number of vehicles in queue. After comparing the second hour frequency distribution of northbound and average frequency distribution of southbound, the results were verified as being acceptable.

Upon verification of the queue distributions, the model was executed to observe its operation. The final model appeared to operate well at the crosswalk location. On occasion, pedestrians would not accept a gap while a yielding vehicle was decelerating to stop. At the Harmon Avenue intersection, right turning vehicles would sometimes choose an inappropriate gap and turn into traffic, causing the following vehicle after turning to decelerate. At the southbound u-turn/left turn bay from Maryland Parkway to Del Mar Street, vehicles would occasionally accept a gap that was insufficient to clear. The likely cause of this behavior was that northbound vehicles queued at the crosswalk were let out of the queue just as these southbound vehicles were turning. Overall, the final model visualization appeared to be acceptable.

In the measure of effectiveness validation phase, the simulated northbound queued vehicles frequency distribution did not match the actual observed queues. After going through additional calibration of global and/or local parameters, it was determined that these parameters could not be adjusted to produce the field queues. The validation raw

data was then re-reviewed, and the simulated queue frequency distribution was compared to the field queue distribution for the second hour, as recorded from 12:00 to 1:00 PM. The Chi-squared results showed that the queue distributions were acceptable (Tables 4-23 and 4-24). At this point, the queue data was analyzed to determine possible causes. The first hour queue count was not completed over an entire hour, and likely caused an incorrect field distribution.

4.4 Delays and Level of Service Evaluations

After the methodology was applied, delays in travel time at the crosswalk were computed and averaged per vehicle by VISSIM. VISSIM defined the delay calculated as being the total delay between two points at which VISSIM recorded the travel times. The free flow travel times between the two points were then subtracted from these observed travel times. Reduced travel time areas that have been created along the evaluation segment were included in the free flow travel times. Therefore, if the crosswalk did not exist, the total delay calculated by VISSIM would be close to zero. So, the total delay calculated by VISSIM was a reasonable approximation of the control delay at the crosswalk. VISSIM also calculated average stopped delay per vehicle, and total delay per person.

For the simulation model, the average total delays reported for northbound and southbound traffic were considered to be the average control delay at the crosswalk. For pedestrians, two types of pedestrians were used in the simulation: individual pedestrians and pedestrian groups. The total delay per person would then be the appropriate value to use for pedestrian delays. Twenty simulation runs were performed, and the values

averaged. Table 4-25 is a summary of the delays as calculated by VISSIM for the final calibrated simulation model, and delays calculated from the validation vehicle queue study.

Table 4-25: Simulated vehicular control delays and pedestrian delays at the crosswalk

Delays, in seconds	Vehicular		Pedestrian	
	Northbound	Southbound	Eastbound	Westbound
Total (control) delay	8.1	7.4	3.2	3.5
Stopped delay	2.1	1.8	1.9	2.1
Total (control) delay per person	8.5	7.3	3.3	3.4
Delay, from field queues	6.5	7.2	---	---

Although the Highway Capacity Manual (ITE 2000) does not specifically include level of service guidelines for unsignalized pedestrian crossings, it does address levels of service at unsignalized intersections. Milazzo et al. (1999) alluded to this methodology as being appropriate for unsignalized pedestrian crossings. However, for most four-way unsignalized intersections, the through volumes are often much less than the through volumes on Maryland Parkway at the crosswalk. Therefore, this methodology may not be applicable for determining vehicular level of service at this location. However, using the predicted control delays in Table 4-25, the northbound level of service would be “B” (5 to 10 seconds). For southbound, the level of service would also be “B”.

The methodology outlined in Milazzo et al. (1999) could be applicable to determining the pedestrian level of service at the crosswalk based on pedestrian total delays. The total delay per pedestrian would be the most appropriate value to use in this case. Using the predicted delays in Table 4-25, the level of service for eastbound pedestrian traffic would be “A” (less than 5 seconds). For westbound pedestrians, the level of service would also

be “A”. The implications of these levels of service would be that pedestrians are not likely to select an inappropriately small gap, or to become impatient and chance an unsafe crossing behavior. One drawback to the Milazzo et al. (1999) methodology is that it does not consider perceived safety or quality of the mid-block crossing.

4.5 Sensitivity of Parameters

Several key parameters were varied to evaluate the effects that these parameters had on measures of effectiveness. This type of sensitivity analysis was intended to judge whether the model performed as expected using varying degrees of input parameters. Each sensitivity analysis was run five times on the final calibrated model, and the outcomes were averaged.

For comparison with the outcomes as reported in Kaseko and Karkee (2005), several key parameters were compared with varying proportions of yielding drivers in the simulation model. It was expected that the average vehicular delays for vehicles would decrease with increasing proportion of yielding drivers, while average pedestrian delays would increase at a lower rate. Figure 4-13 shows that this expectation holds true for the model. The number of stopped vehicles was also expected to increase with an increase in the proportion of yielding drivers, as demonstrated in Figure 4-14. For queue lengths, it was expected that longer queue lengths would be experienced as the proportion of yielding drivers increased, which is shown in Figure 4-15. These results appear to exhibit similar trends as reported in Kaseko and Karkee (2005).

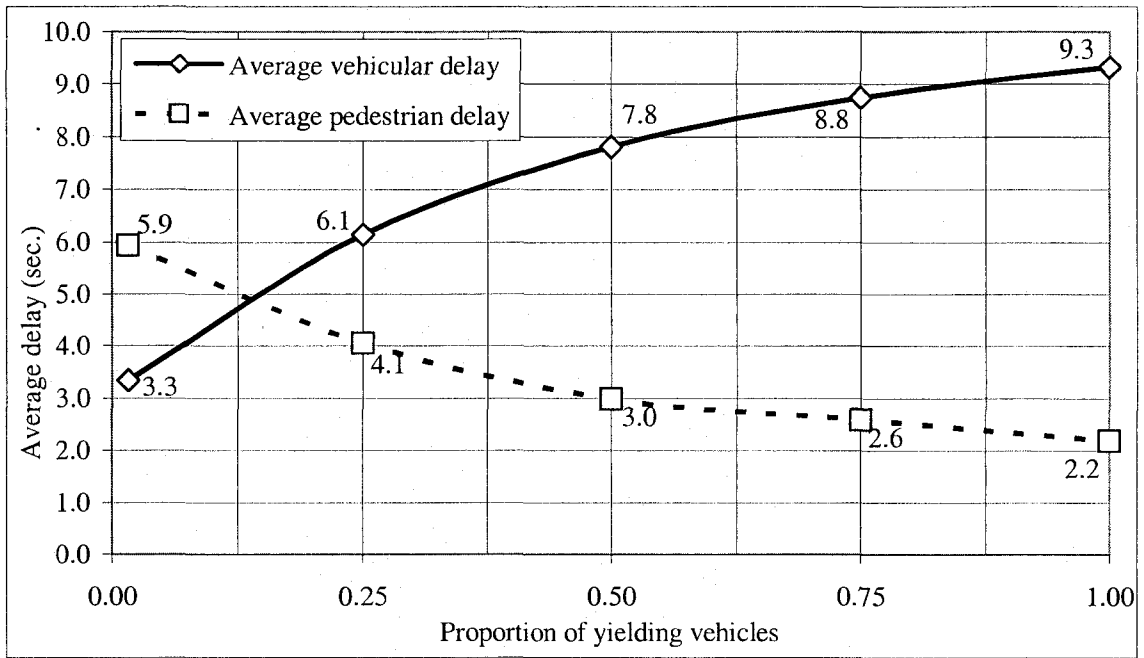


Figure 4-13: Average delays versus yielding driver proportion for the calibrated model

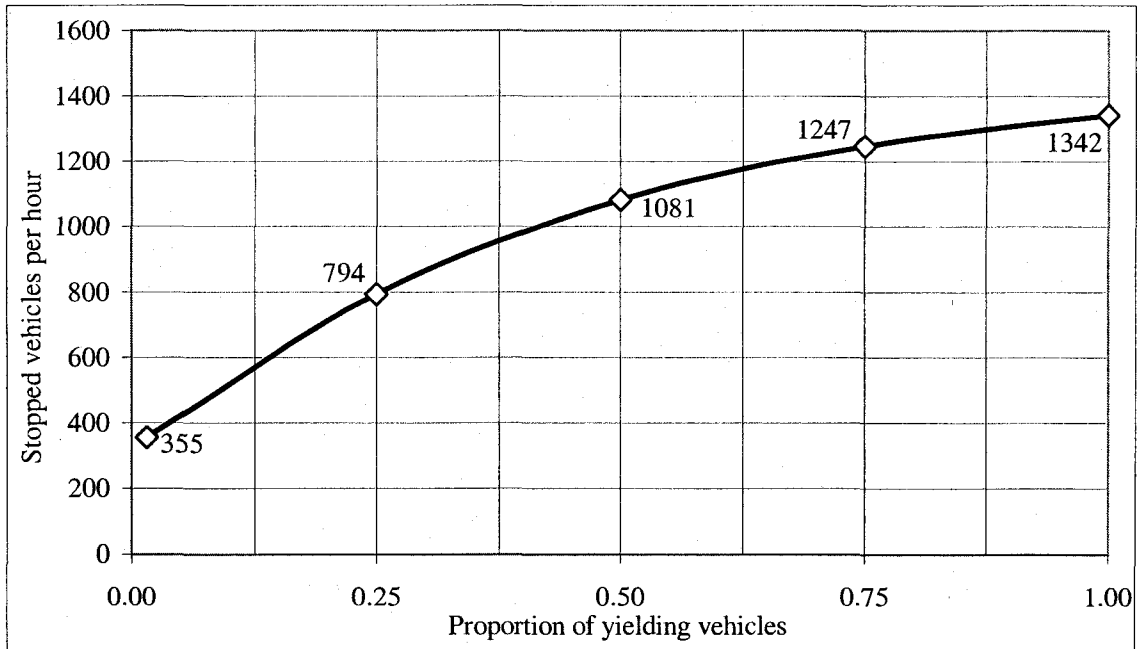


Figure 4-14: Total stopped vehicles per hour versus yielding driver proportion for the calibrated model

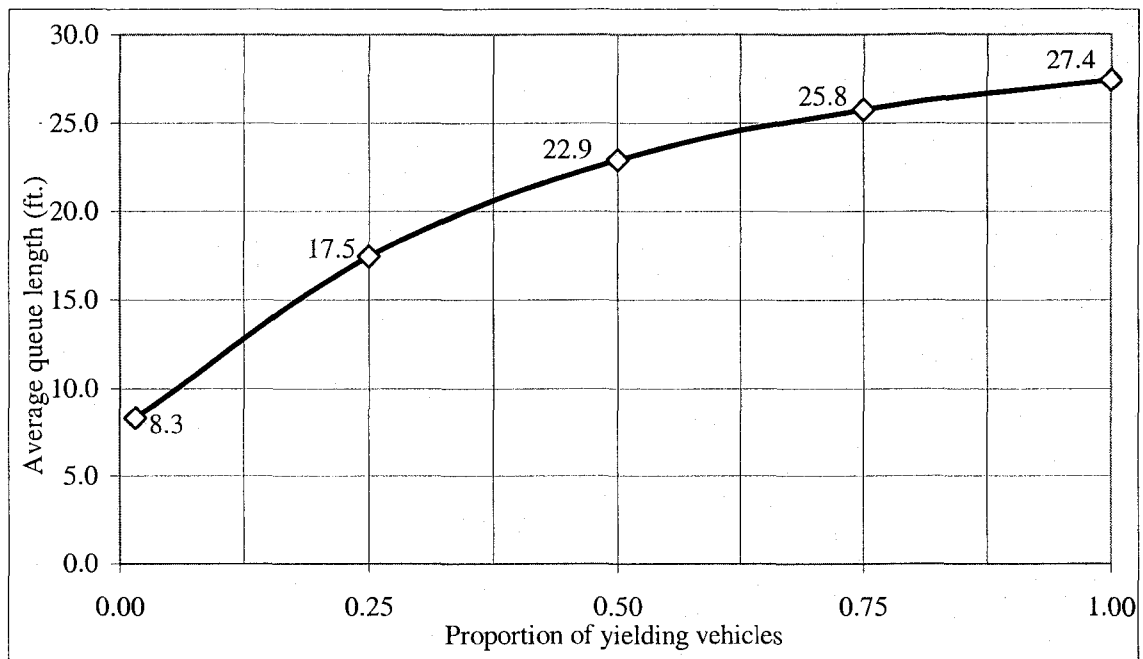


Figure 4-15: Average queue lengths versus yielding driver proportion for the calibrated model

To compare the effect of varying vehicle volumes on average delays, all locations where vehicle volumes were generated in VISSIM were reduced or increased by a percentage of the original value as input into the model. It was expected that as vehicle volumes increased, the average vehicular delays would increase, as well as average pedestrian delays, but at a lower rate. Figures 4-16 and 4-17 demonstrate that the model performed as expected when vehicular volumes were adjusted. For pedestrian volume adjustments, vehicular delays increased as expected. Pedestrian delays reported were more erratic as the pedestrian volume was adjusted, and did not exhibit a particular trend. This could be the result of additional pedestrian congestion offsetting any potential benefit from more pedestrians crossing at the same time.

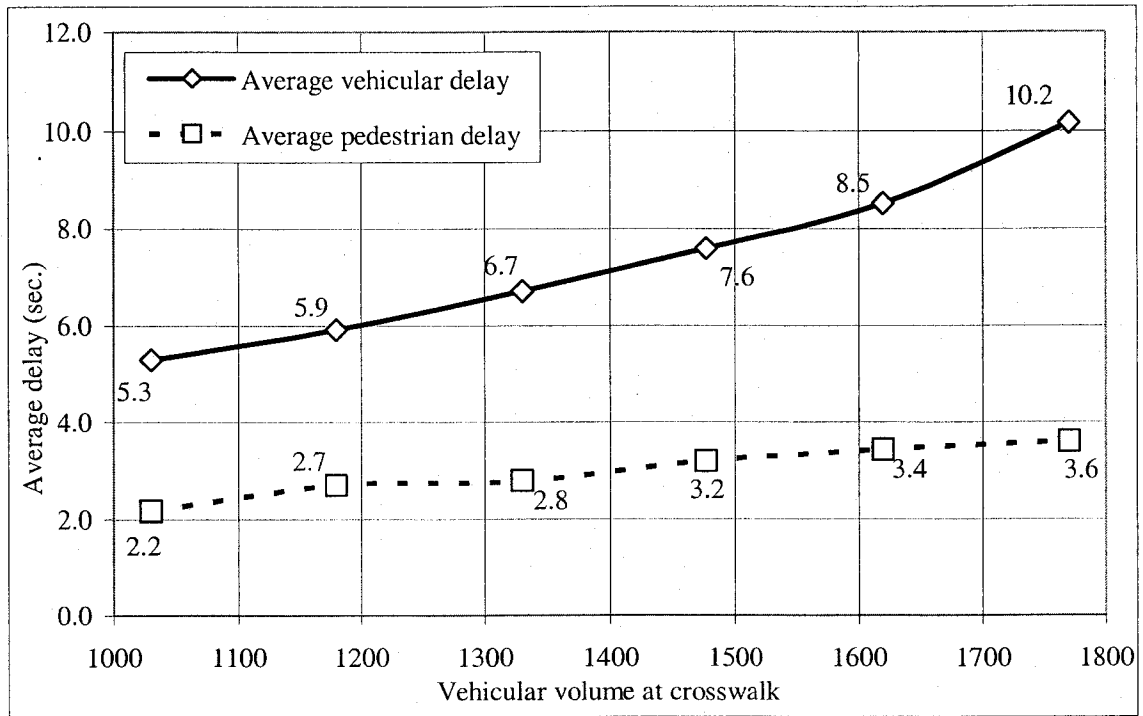


Figure 4-16: Average vehicle delays versus vehicular volumes for the calibrated model

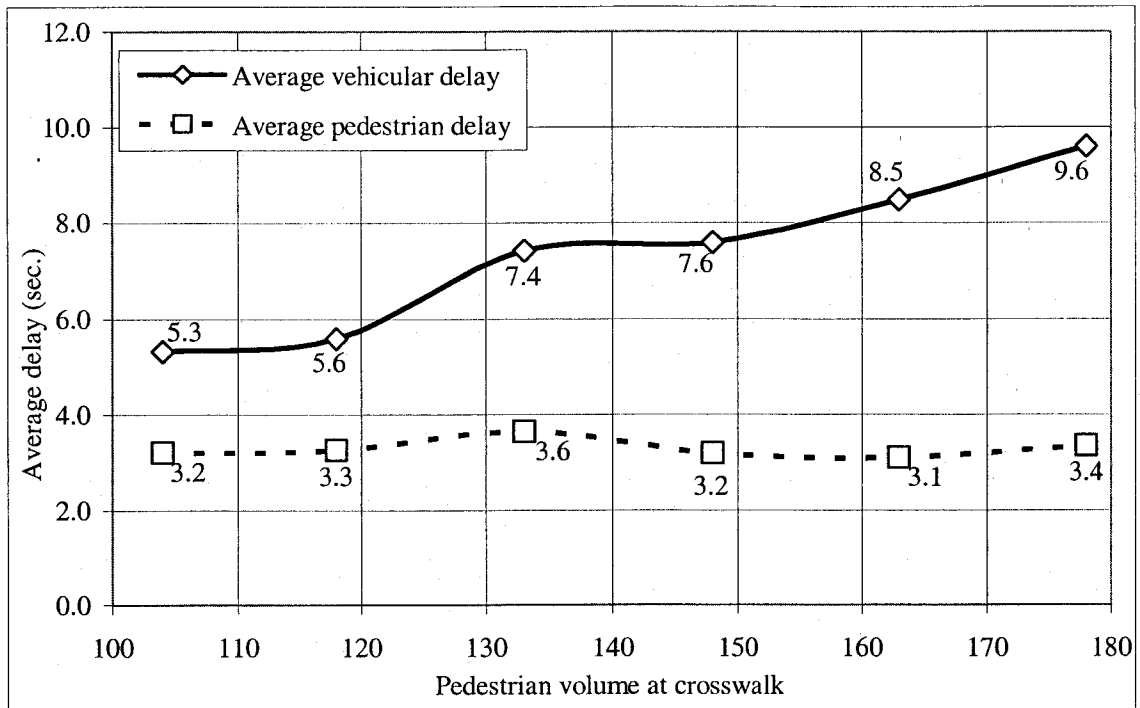


Figure 4-17: Average vehicle delays versus pedestrian volumes for the calibrated model

Three calibration parameters were also selected for sensitivity analysis. These three were selected because the amount of delay varied greatly depending on the values chosen. First, the minimum accepted gap by a yielding driver for pedestrians, G_{yp} , was varied against average vehicular and pedestrian delays. G_{yp} was assigned two different values in the final model: 8.0 seconds for northbound traffic, and 7.0 seconds for southbound traffic. Sensitivity was performed by subtracting or adding the same number of seconds for each direction, and comparing these differences with the average delays. It was expected that average vehicular delays would increase, and pedestrian delays would decrease, as the seconds of accepted gap times increased. Figure 4-18 confirms this expectation.

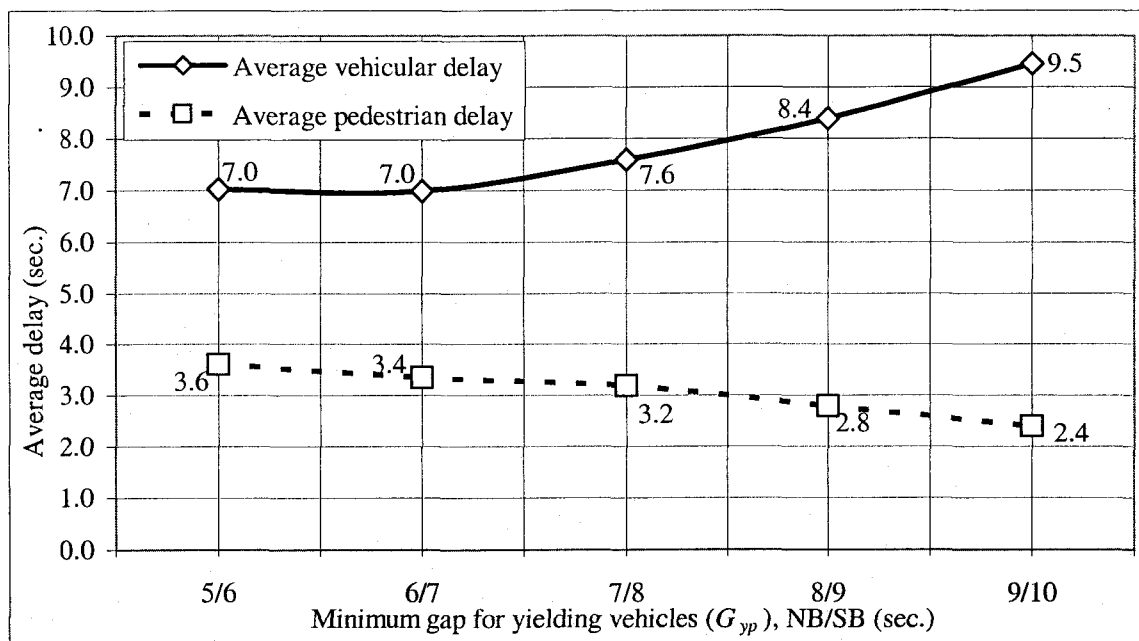


Figure 4-18: Average delays versus G_{yp} (minimum gap times for yielding vehicles) for calibrated model

Additionally, the “Additive part of Weidemann 74” and “Multiplicative part of Weidemann 74” global car-following parameters were varied against average vehicular and pedestrian delays. Again, it was expected as both of these parameters increased, the average vehicular delays would increase, and average pedestrian delays would decrease. Figures 4-19 and 4-20 demonstrate that the expectations for increased vehicular delays were exhibited by the model. However, pedestrian delays were not significantly impacted. One reason for this is because the additional gaps created by both the “Additive” and “Multiplicative” global parameters were not significantly large enough to allow additional pedestrians to cross.

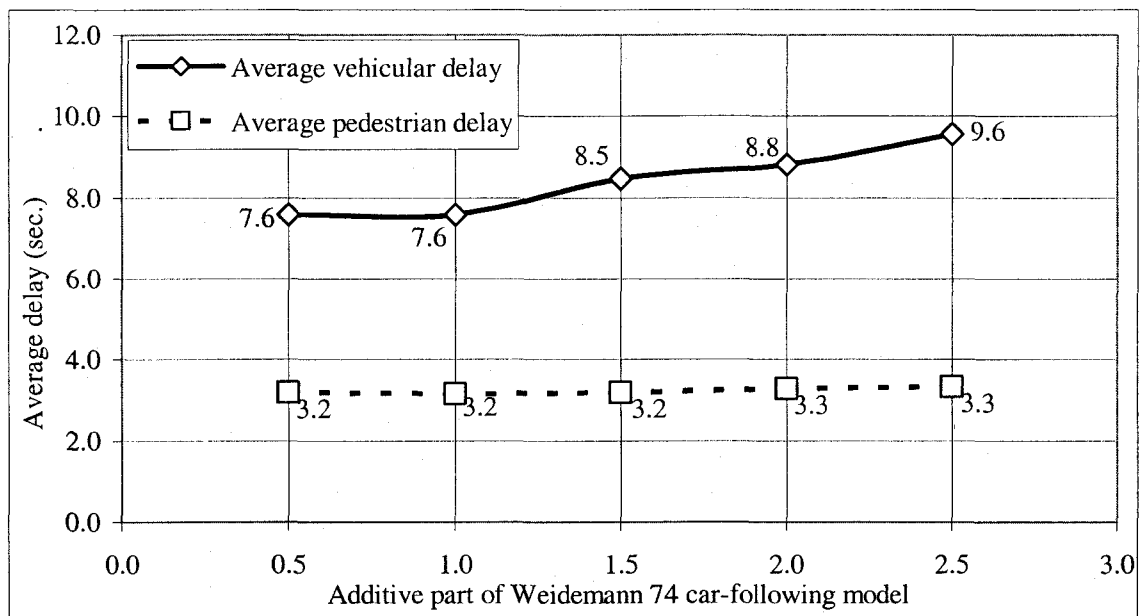


Figure 4-19: Average delays versus “Additive part of Weidemann 74” global parameter for the calibrated model

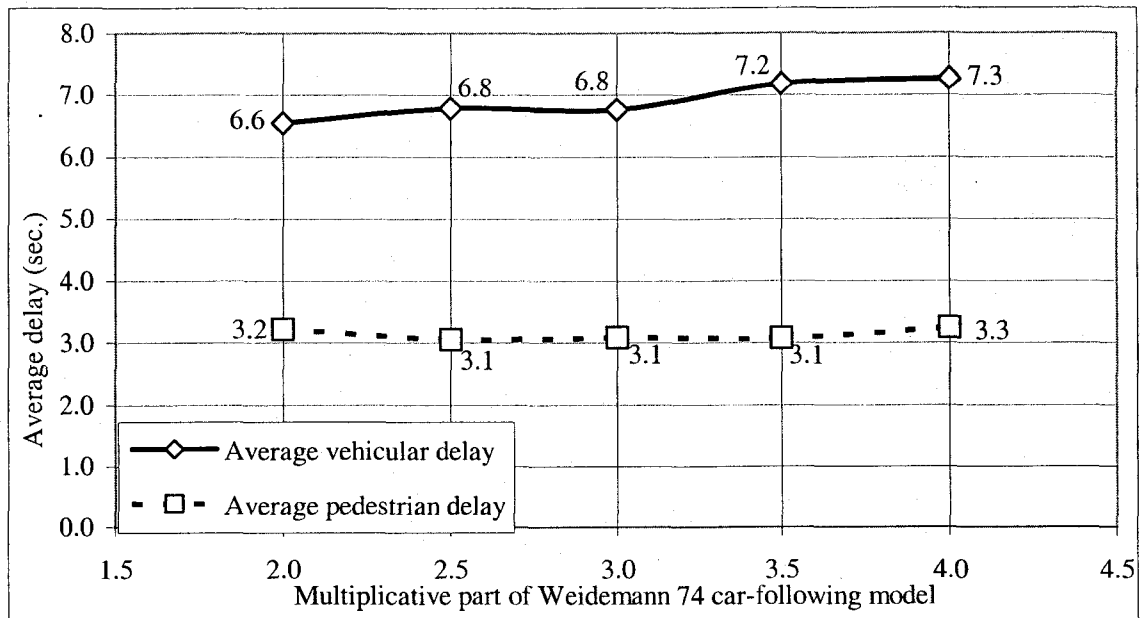


Figure 4-20: Average delays versus “Multiplicative part of Weidemann 74” global parameter for the calibrated model

4.6 Uses for Model and Methodology

To illustrate the usefulness of developing a microsimulation model such as the one developed for this thesis, design scenarios can be created easily using the final calibrated model. Regarding urban arterial level of service performance, the study corridor segment level of service can be evaluated with and without a pedestrian mid-block crossing. The level of service criteria used were based on those for urban streets as published in the Highway Capacity Manual (HCM) (Transportation Research Board [TRB] 2000).

The effect on running speeds through the case study corridor with and without pedestrians was evaluated. First, the model was executed 20 times and the average delay times were calculated for the entire corridor from approximately 500 feet north of Harmon Avenue and approximately 500 feet south of University Road. The HCM (TRB

2000) stated that levels of service for urban streets were based on the average running speeds calculated for the segment, which could be derived from total delays incurred through the segment. The free flow speed for these two directions of traffic already determined earlier in this study was 33.7 miles per hour. The free flow speed can be used to determine the free flow travel times. VISSIM calculates total delay as the difference between the free flow travel times and the individual vehicle travel times. Therefore, the average total delay calculated by VISSIM can be used to calculate the average running speeds through the segment. These speeds can then be used to determine level of service for each direction of traffic as defined in the HCM (TRB 2000).

Travel times, distances, and levels of service were determined for the model for both the base case, as well as without pedestrians. The results of this analysis are shown in Table 4-26 and 4-27. The running speed only increased by about 1.5 miles per hour when pedestrian traffic was removed from the crosswalk, which did not improve the level of service calculated for the segment.

Table 4-26: Urban street level of service calculation for calibrated model

Roadway Element	Evaluation distance, feet	Free flow running time, seconds	Total delay, seconds	Actual running time, seconds	Running speed, mph	LOS
Northbound Maryland Parkway	2,118	42.8	59.9	102.7	14.0	C
Southbound Maryland Parkway	2,044	41.3	45.4	86.7	16.0	C

Table 4-27: Urban street level of service calculation for calibrated model, excluding pedestrians

Roadway Element	Evaluation distance, feet	Free flow running time, seconds	Total delay, seconds	Actual running time, seconds	Running speed, mph	LOS
Northbound Maryland Parkway	2,118	42.8	44.1	86.8	16.6	C
Southbound Maryland Parkway	2,044	41.3	34.1	75.4	18.5	C

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Methodology Performance and Limitations

Overall, the methodology that was developed for this study appeared to have functioned well. However, there are a few areas which could improve the developed methodology. The steps outlined should be considered as guidelines, but not a fixed process. It is essential that selecting the measure(s) of effectiveness commence before data is collected. The process used for performing “major calibration” must be performed in the exact sequence as outlined in the Methodology (Chapter 3).

The test statistics used in the methodology were adequate for determining the goodness of fit for the actual location versus the simulated model. The *RMSP* and correlation coefficient r were not nearly as sensitive to changes made to global and local parameters as were the Theil (1961) U statistics. Furthermore, the values obtained for the U components did not always reach the values as recommended by Hourdakis et al. (2003), who did not justify the selection of these values in their publication. A more practical recommendation for ensuring that the actual field observations match those by the simulation may be to first check the simulation model using the *RMSP* and r statistics for general fit. After conducting this general fit, the model should be calibrated using the four U statistics so that the values calculated for each measure of effectiveness are approximately equal.

Since the “minor calibration” and “measure of effectiveness validation” stages were related closely, it may be more effective to combine these into one distinct step. The “minor calibration” step was generally used to satisfy both the calibration and validation measures of effectiveness simultaneously.

The simulation model did not take into consideration erratic actions of drivers or pedestrians. Drivers could exhibit various yielding behaviors such as much slower deceleration rates, or stopping before or after the yield bar. Because of this, the model cannot be used to analyze the safety of the crossing. It could, however, be used to quantify the benefit of proposed safety improvements. The potential for crashes between vehicles and pedestrians were not considered in this model, since microsimulations are not intended to simulate these events. Since the purpose of this study was to develop the methodology for designing, calibrating, validating, and analyzing the operations of mid-block pedestrian crossings using a microsimulation model, safety concerns were not directly considered to be part of the scope. However, much research has been done regarding the quantification of pedestrian safety and crash rates at pedestrian crossings; Cui and Nambisan (2003), and Pulugurtha and Nambisan (2003), are examples of publications which address this subject.

5.2 Proportion of Yielding Drivers and Public Policy

Determination of the proportion of yielding drivers is important because it 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. Operational analyses would benefit by being able to determine how the proportion of law-abiding drivers affects the delay and

queue experienced by both drivers and pedestrians. This data could also be useful in supplementing methods to analyze pedestrian safety at mid-block crossings. 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.

As discussed earlier in the Methodology (Chapter 3), there are several reasons why a driver may not yield to a pedestrian. Among these are ignorance of respective laws, not recognizing the condition to yield, and willful non-compliance with the law.

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 this procedure will vary based on each practical application. For instance, 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 a sample proportion of 0.435, where the confidence interval of the proportion was $0.378 \leq p \leq 0.509$ at 95% confidence. 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.

5.3 Recommendations for Further Research

The research conducted in this thesis presented numerous additional research topics. Studies confirming the methodology for constructing, calibrating, validating, and evaluating other mid-block pedestrian crossing locations would be one obvious direction for future research. Another subject for exploration would be how signal offset and green splits affect the delays incurred by vehicles and pedestrians through the corridor. 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 use of this methodology could be to conduct before and after studies (such as different pavement markings or signage at the crossing) at particular crossing locations to determine whether there has been an increase in the proportion of yielding drivers. Many public policy decisions would also need to be formulated regarding the outcomes of studies of yielding driver proportion, to ensure that the conclusions drawn from these studies are valid. The proportion of yielding drivers could also be studied for use as an indirect measure of effectiveness being based on safety (as opposed to delay),

for the computation of level of service at the mid-block crossing. Data pertaining to the yielding parameters may also be ideal for further study, such as minimum or average gaps accepted by pedestrians waiting to cross at the crosswalk.

APPENDIX I

SAMPLE DATA COLLECTION SHEETS

INTERSECTION STOPPED - DELAY FIELD SHEET

Intersection SWARD/DEL MAR Time 11:00 To 1:00
 Date 11/17 Day Sunny Wind 0
 Weather Sunny Observer(s) Silvan Lane middle
 Street Maryland Hwy

VEHICLES, SOUTH BOUND (MARQUE TRAIL LANE)

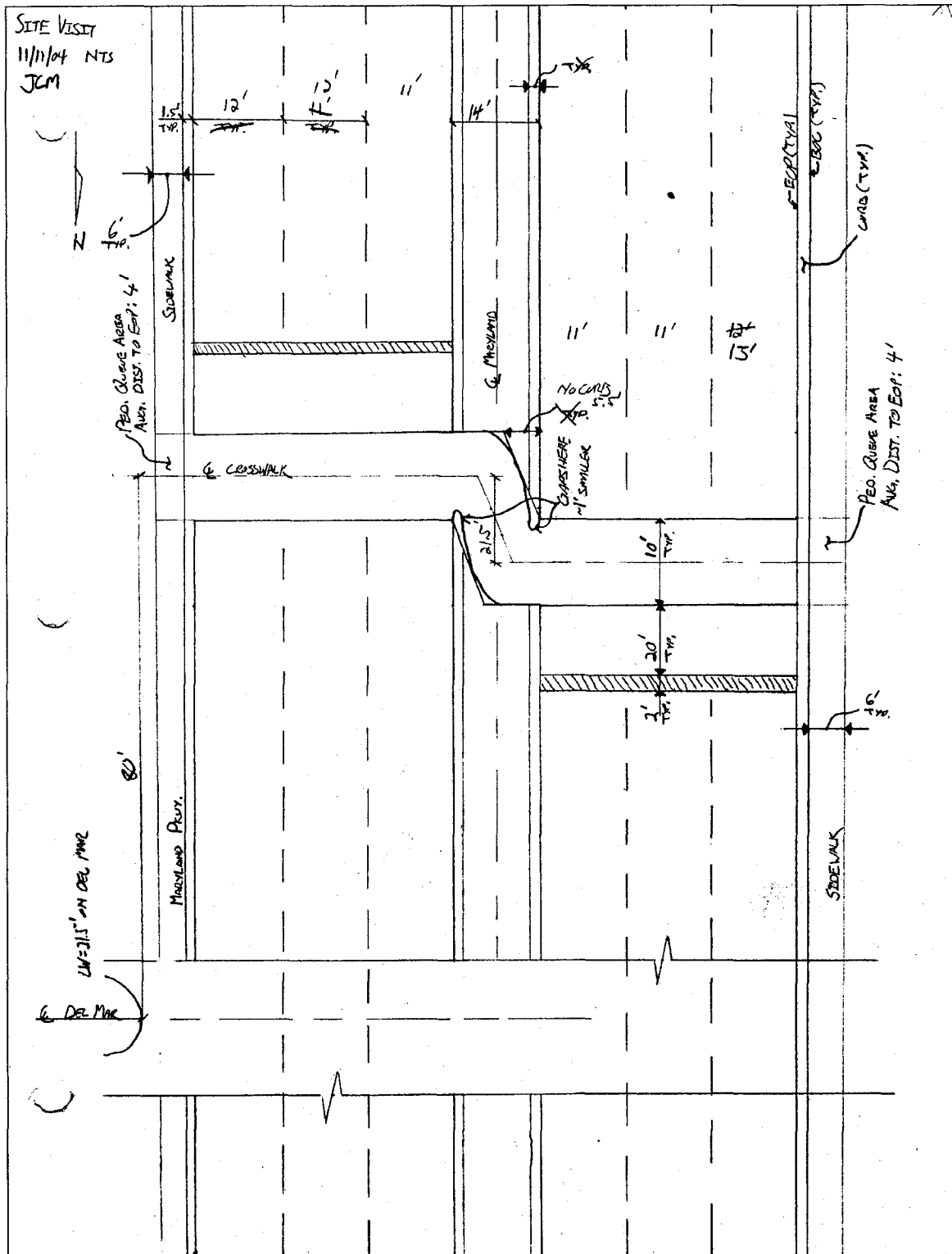
Min	Sec	Number of Stopped V's/P's							
		+30	+60	+90	+120	+150	+180	+210	+240
12.00p		0	0	0	1	0	0	0	0
12.04		0	0	2	0	0	3	0	1
12.08		2	0	0	0	0	0	2	0
12.12		1	0	0	4		1	0	1
12.16		2	1	2	1	1	0	0	0
12.20		4	0	1	1	0	0	0	0
12.24		0	0	1	0	0	1	0	3
12.28		2	1	0	0	0	0	0	1
12.32		1	0	4	3	0	5	0	0
12.36		3	0	1	1	0	0	0	0
12.40		1	0	0	0	2	0	1	2
12.44		3	0	0	1	4	6	4	0
12.48		3	0	2	0	0	0	6	0
12.52		6	5	7	4	0	0	0	0
12.56		4	0	0	0	5	3	0	0

PEDESTRIANS, EAST BOUND

Min	Sec	Number of Stopped V's/P's							
		+30	+60	+90	+120	+150	+180	+210	+240
12.00p		0	2	0	3	0	0	1	0
12.04		0	0	3	0	0	1	0	1
12.08		2	0	0	0	0	0	1	0
12.12		2	3	0	3	0	1	0	2
12.16		5	2	2	2	2	0	0	0
12.20		1	0	2	0	0	1	0	0
12.24		0	0	1	0	0	1	0	3
12.28		1	3	0	0	1	0	0	1
12.32		6	1	1	2	0	3	0	0
12.36		1	0	3	1	0	0	1	0
12.40		1	0	0	0	1	0	1	1
12.44		1	0	2	2	3	2	1	0
12.48		1	0	2	0	0	0	0	0
12.52		2	1	2	1	0	0	0	0
12.56		2	0	0	0	2	1	0	0

Vehicular and pedestrian queue count sheet, as adapted from the Manual of Traffic

Engineering Studies (ITE 1994)



Site investigation sketch of the mid-block crossing at Del Mar Street

NORTH / EAST APPROACHES NB/EB APPROACHES

VEHICLE TURNING MOVEMENT COUNT

FOUR-APPROACH FIELD SHEET

Time 11:00 to 11:07

N/S Street MARYLAND PKWY. Date 11/30/54 Day Tues.

E/W Street HARMON AVE. Weather Sunny

P = passenger cars, station wagons, motorcycles, pick-up trucks. Observer JCM

T = other trucks. (Record any school bus as SB; other buses as B).

U-TURN

U-TURN

Harmon Avenue southbound/westbound volume count sheet, as adapted from the Manual of Transportation Engineering Studies (ITE 1994)

North   *Movements*

VEHICLE TURNING MOVEMENT COUNT

FOUR-APPROACH FIELD SHEET *

Time 11.14 to 11.24

N/S Street MARYLAND PKWY   Date _____ Day _____

E/W Street UNIVERSITY AV.   Weather _____

P = passenger cars, station wagons, motorcycles, pick-up trucks.   Observer _____

T = other trucks. (Record any school bus as SB; other buses as B).

Intersection Diagram Data:

- Northbound (University Av. from South):**
  - Left Turn: 15 (P)
  - Through/Right: 111 (P)
- Southbound (University Av. to North):**
  - Left Turn: 16 (P)
  - Through/Right: 115 (P)
- Westbound (Maryland Pkwy. to East):**
  - Left Turn: 22 (P)
  - Through/Right: 32 (P)
- Eastbound (Maryland Pkwy. to West):**
  - Left Turn: 4 (P)
  - Through/Right: 20 (P)

Additional Notes:

- 1 Bus (B)
- 1 Truck (T)
- U-Turn: 10 (P)

University Road northbound movements volume count sheet, as adapted from the Manual of Transportation Engineering Studies (ITE 1994)

VEHICLE TURNING MOVEMENT COUNT FOUR-APPROACH FIELD SHEET

Time 11:45 a to 11:50 a

N/S Street MARYLAND PKWY.

Date WED 12/11/74 Day _____

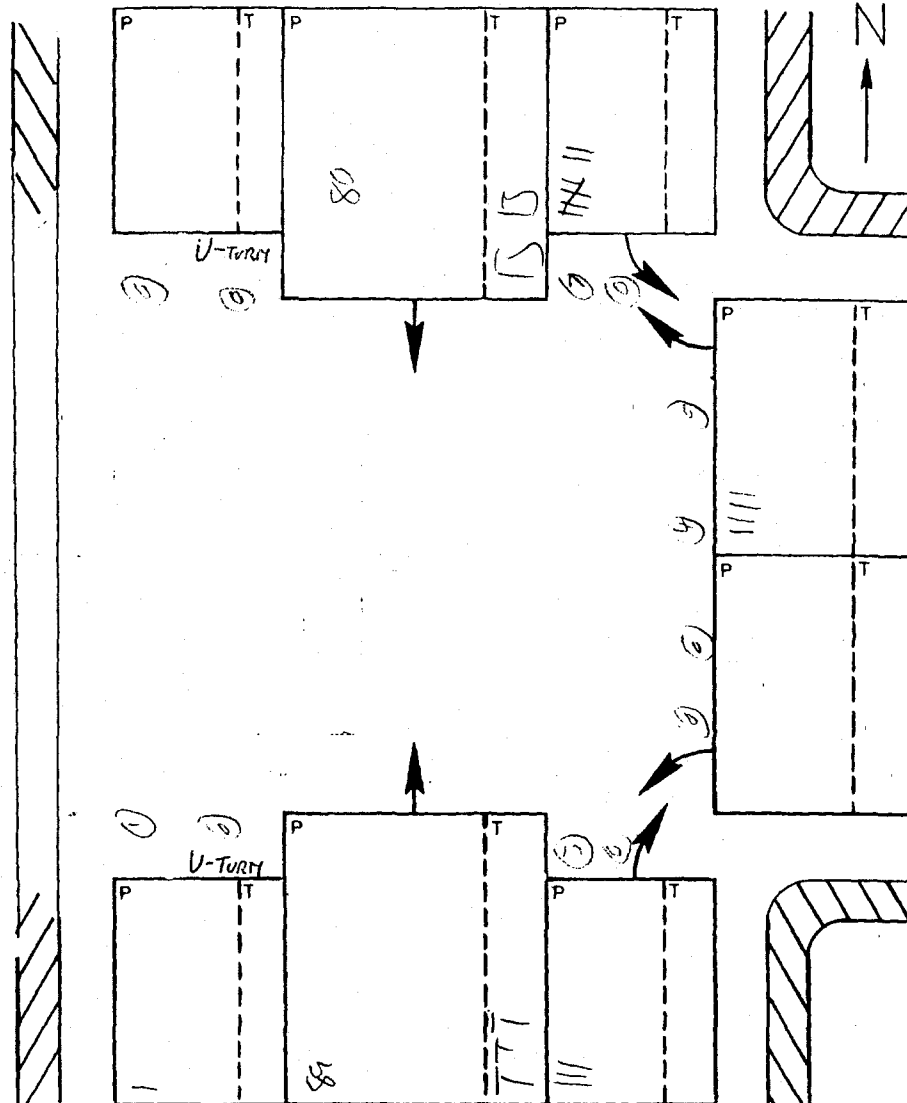
E/W Street DEL MAR ST.

Weather SPRINKLY

P = passenger cars, station wagons, motorcycles, pick-up trucks.

Observer JCM

T = other trucks. (Record any school bus as SB; other buses as B).



Del Mar Street turning movements and volume count sheet, as adapted from the Manual of Transportation Engineering Studies (ITE 1994)

UNLV

TIME 5 TO 11:15

DATE _____

OBSERVER _____

CROSSWALK FIELD SHEET
PEDESTRIAN COUNT
 DEL MAR X-WALK

DEL MAR ST.

EINSTEIN DRIVEL

CHILDREN	ADULTS UNDER 65	ADULTS OVER 65	DISABLED	
	⑪			← MAYLAND AVE.
	⑪			
	⑪			→
				→

N
↑

← CROSSWALK →

① CIRCLE IF GROUP. EXAMPLE: ⑪ INL ⑪ INL

② IF MORE THAN ONE CLASSIFICATION IN GROUP, USE SLOWEST CLASSIFICATION.

③ CHILDREN ARE 14 YEARS OR YOUNGER.

Pedestrian count sheet for the Del Mar mid-block crossing, as adapted from the Manual of Transportation Engineering Studies (ITE 1994)

[illegible]

113

Date/Time 11/1/79 11:00 AM **Spot Speed Study** Weather Sunny

Observer EP Location 11th St. & 1st St. NE

100 50 $\alpha = 45^\circ$

Speed	Lane	Class	Speed	Lane	Class	Speed	Lane	Class
31	3	P						
28	1	P						
31	2	P						
35	1	P						
35	1	P						
35	1	P						
32	2	P						
36	3	P						
37	1	P						
35	2	P						
34	2	P						
32	2	P						
28	2	P						
27	1	P						
31	1	P						
25	2	P						
25	2	P						
32	1	P						
35	2	P						
27	1	P						
35	2	P						
30	2	P						
33	1	P						
29	1	P						
31	1	P						
31	1	P						
33	1	P						
37	2	P						
31	1	P						
32	2	P						
31	2	P						
38	3	P						
38	3	P						
31	2	P						
31	1	P						
30	1	P						
34	2	P						
28	1	P						
34	1	P						
32	1	P						
33	1	P						
35	2	P						
37	1	P						
42	3	P						
34	3	P						
23	1	P						
28	1	P						
32	1	P						
14	2	P						
27	2	P						

Speed study data collection sheet, with radar angle of incidence

Vehicle Yielding Study														
Location _____		Start time <u>11:00</u>												
Observer <u>JLM</u>		End time <u>11:00</u>												
Date <u>11/17/04</u>														
# Vehicles observed (X)	1	2	3	4	5	6	7	8	9	10	11	12	13	< 13
# Passing before 1 stop	0	1	2	3	4	5	6	7	8	9	10	11	12	< 12
Occurrence 1	X													
Occurrence 2							X	X						
Occurrence 3	X													
Occurrence 4	X													
Occurrence 5	X													
Occurrence 6			X											
Occurrence 7	X													
Occurrence 8		X												
Occurrence 9		X												
Occurrence 10	X													
Occurrence 11			X											
Occurrence 12	X													
Occurrence 13												X		
Occurrence 14		X												
Occurrence 15			X											
Occurrence 16	X													
Occurrence 17	X													
Occurrence 18		X												
Occurrence 19			X											
Occurrence 20	X													
Occurrence 21		X												
Occurrence 22		X												
Occurrence 23														
Occurrence 24														
Occurrence 25														
Occurrence 26														
Occurrence 27														
Occurrence 28														
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Occurrence 39														
Occurrence 40														
Occurrence 41														
Occurrence 42														
Occurrence 43														
Occurrence 44														
Occurrence 45														

Proportion of yielding drivers data collection sheet

APPENDIX II

SUMMARIES FOR TRAFFIC DATA COLLECTED

Northbound Maryland Parkway	Univ. to crosswalk	Crosswalk to Harmon	University to Harmon
Average travel time (seconds)	19.4	48.3	74.0
Standard deviation	10.4	25.8	30.1
Minimum	8.0	15.0	26.0
Maximum	69.0	101.0	134.0
Number of samples	46	42	45
Distance traveled (feet)	482	700	1182
Average running speed (mph)	16.9	9.9	10.9

Southbound Maryland Parkway	Harmon to crosswalk	Crosswalk to University	Harmon to University
Average travel time (seconds)	28.7	23.4	52.3
Standard deviation	9.9	14.1	22.4
Minimum	14.0	9.0	27.0
Maximum	55.0	69.0	115.0
Number of samples	40	72	49
Distance traveled (feet)	762	453	1215
Average running speed (mph)	18.0	13.1	15.8

Summary of license plate travel time study results

Maryland/University	EBL	EBR	NBU	NBL	NBT	SBT	SBR
Passenger	97	158	22	175	1367	1250	121
Truck/RV	0	0	0	1	19	10	0

Maryland/Del Mar	WBL	WBR	NBU	NBT	NBR	SBU	SBL	SBT
Passenger	31	74	47	1501	36	17	104	1299
Truck/RV	0	2	0	22	0	0	1	25

Maryland/Harmon	EBL	EBT	EBR	WBL	WBT	WBR	NBU
Passenger	77	39	37	216	39	131	45
Truck/RV	0	0	0	4	0	0	0
Maryland/Harmon	NBL	NBT	NBR	SBU	SBL	SBT	SBR
Passenger	70	1285	87	14	144	1223	92
Truck/RV	0	14	3	0	2	19	0

Summary of turning movement counts at the three intersections

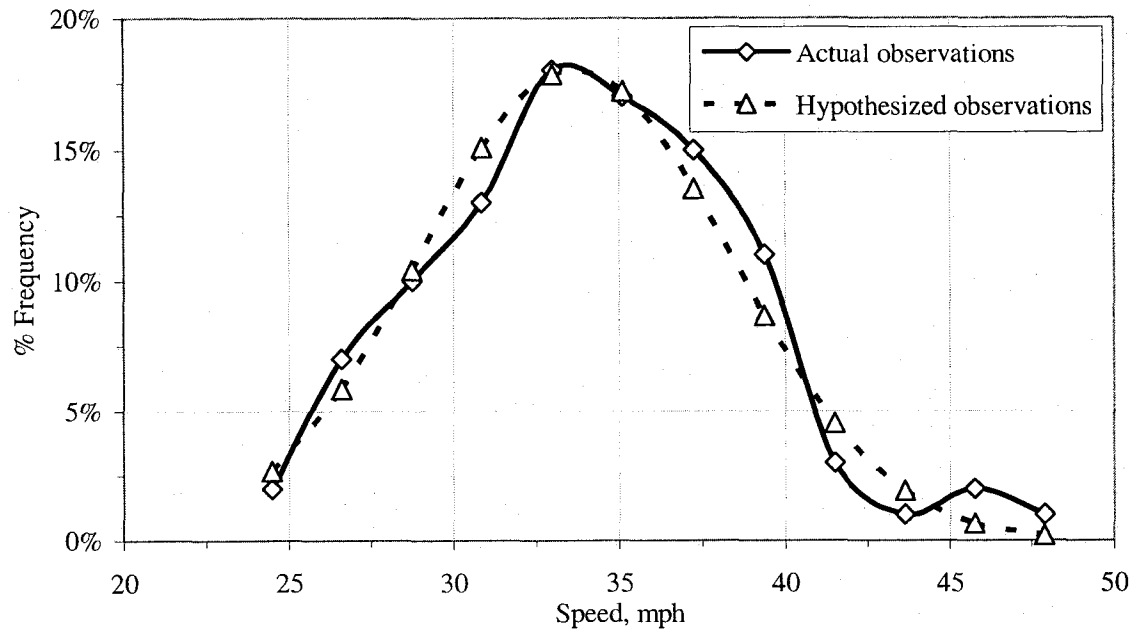
Eastbound Del Mar crosswalk							
Number ped. in group	1	2	3	4	5	6	Total
Average group/hr.	106	18.5	5	0.5	0	0	130
Average ped./hr.	106	37	15	2	0	0	160

Westbound Del Mar crosswalk							
Number ped. in group	1	2	3	4	5	6	Total
Average group/hr.	77.5	19.5	4.5	0.5	0	0.5	102.5
Average ped./hr.	77.5	39	13.5	2	0	3	135

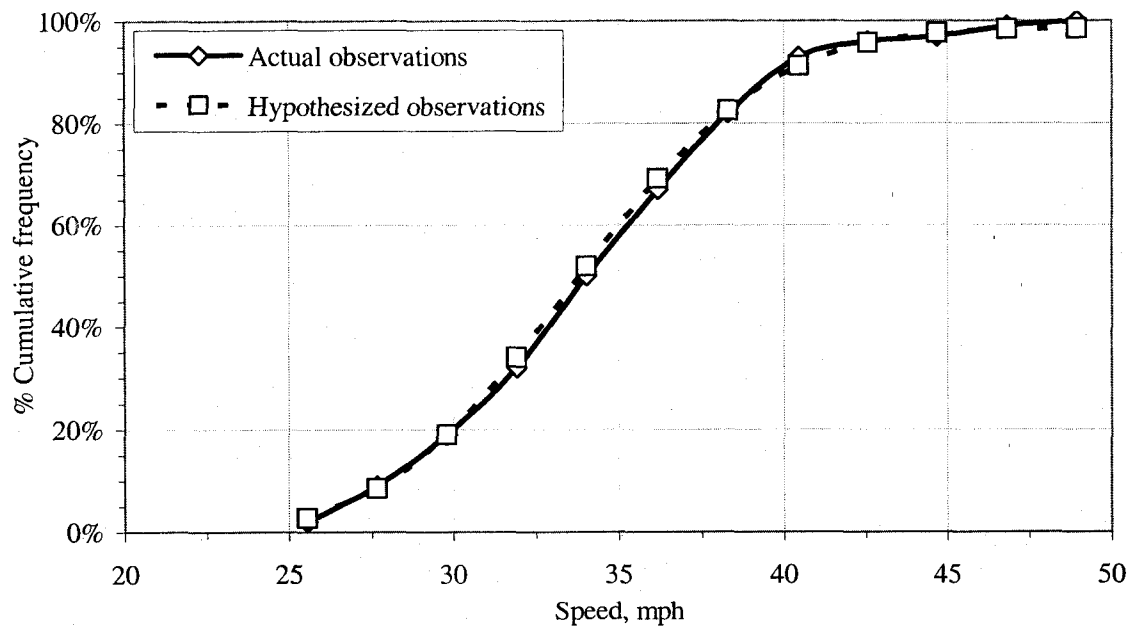
Summary of pedestrian counts at crosswalk

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 $\alpha = 20^\circ$)	(adjusted for $\alpha = 20^\circ$)	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



Frequency distributions for measured and hypothesized free flow speeds



Cumulative frequency distributions for measured and hypothesized free flow speeds

APPENDIX III

SIMULATION MODEL YIELDING PARAMETERS

Parameter	Direction (vehicular/ pedestrian)	Unit	Default (calculated) value
H_{yp}	NB/EB	feet	43.0
H_{np1}	NB/EB	feet	39.0
H_{np2}	NB/EB	feet	27.0
H_{np3}	NB/EB	feet	15.0
H_{py}	NB/EB	feet	30.0
H_{pn1}	NB/EB	feet	30.0
H_{pn2}	NB/EB	feet	30.0
H_{pn3}	NB/EB	feet	30.0
G_{yp}	NB/EB	seconds	13.0
G_{np1}	NB/EB	seconds	4.0
G_{np2}	NB/EB	seconds	4.0
G_{np3}	NB/EB	seconds	4.0
G_{py}	NB/EB	seconds	1.0
G_{pn1}	NB/EB	seconds	3.0
G_{pn2}	NB/EB	seconds	3.0
G_{pn3}	NB/EB	seconds	3.0
H_{yp}	NB/WB	feet	43.0
H_{np1}	NB/WB	feet	16.0
H_{np2}	NB/WB	feet	28.0
H_{np3}	NB/WB	feet	39.0
H_{py}	NB/WB	feet	30.0
H_{pn1}	NB/WB	feet	30.0
H_{pn2}	NB/WB	feet	30.0
H_{pn3}	NB/WB	feet	30.0
G_{yp}	NB/WB	seconds	13.0
G_{np1}	NB/WB	seconds	4.0
G_{np2}	NB/WB	seconds	4.0
G_{np3}	NB/WB	seconds	4.0
G_{py}	NB/WB	seconds	1.0
G_{pn1}	NB/WB	seconds	3.0
G_{pn2}	NB/WB	seconds	3.0
G_{pn3}	NB/WB	seconds	3.0

Vehicle/pedestrian yielding parameter default values, as calculated

Parameter	Direction (vehicular/ pedestrian)	Unit	Default (calculated) value
H_{yp}	SB/EB	feet	43.0
H_{np1}	SB/EB	feet	17.0
H_{np2}	SB/EB	feet	28.0
H_{np3}	SB/EB	feet	39.0
H_{py}	SB/EB	feet	30.0
H_{pn1}	SB/EB	feet	30.0
H_{pn2}	SB/EB	feet	30.0
H_{pn3}	SB/EB	feet	30.0
G_{yp}	SB/EB	seconds	13.0
G_{np1}	SB/EB	seconds	4.0
G_{np2}	SB/EB	seconds	4.0
G_{np3}	SB/EB	seconds	4.0
G_{py}	SB/EB	seconds	1.0
G_{pn1}	SB/EB	seconds	3.0
G_{pn2}	SB/EB	seconds	3.0
G_{pn3}	SB/EB	seconds	3.0
H_{yp}	SB/WB	feet	43.0
H_{np1}	SB/WB	feet	39.0
H_{np2}	SB/WB	feet	26.0
H_{np3}	SB/WB	feet	15.0
H_{py}	SB/WB	feet	30.0
H_{pn1}	SB/WB	feet	30.0
H_{pn2}	SB/WB	feet	30.0
H_{pn3}	SB/WB	feet	30.0
G_{yp}	SB/WB	seconds	13.0
G_{np1}	SB/WB	seconds	4.0
G_{np2}	SB/WB	seconds	4.0
G_{np3}	SB/WB	seconds	4.0
G_{py}	SB/WB	seconds	1.0
G_{pn1}	SB/WB	seconds	3.0
G_{pn2}	SB/WB	seconds	3.0
G_{pn3}	SB/WB	seconds	3.0

Vehicle/pedestrian yielding parameter default values, as calculated (continued)

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