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Effects of Pedestrian Crossing on Roundabout Capacity

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Chapter 1: Introduction

1.1 Background

Modern roundabout is a relatively new form of intersection that has gained popularity in the United States due to its significant reduction of traffic accidents, reduction of vehicular delay and improvement in safety for pedestrians, cyclists and drivers(SIIH, 2000). There has been significant research to determine the entry capacity of roundabouts in countries such as Germany, U.K., Australia and others (Troutbeck, 1998). Although the number of pedestrians crossing a roundabouts approach has a clear effect on the approach entry capacity as with any other intersection, there has been no significant research in the U.S. on the effects of pedestrian crossing at roundabouts (HCM, 2010).

The recently released Highway Capacity Manual 2010 (HCM 2010) expresses the need to conduct research related to pedestrian effects on roundabout with the following statement "research on the operational performance of pedestrians at roundabouts is limited, both in terms of the effect of motor vehicles on pedestrians and of pedestrians on motor vehicles". It further adds that "additional research on pedestrian operations at roundabouts is needed to develop and refine procedures that adequately address these issues" (TRB, 2010).

The research presented in this thesis analyzes the effects of pedestrian volume on the entry capacity at roundabout entrances. In addition the impact of the pedestrian crosswalk position (with different pedestrian volume) on vehicles entry capacity is also examined.

1.2 Objective and Scope

The goal of this research is to develop a set of pedestrian reduction factors that reflect the capacity reductions associated with pedestrian crossing volume and crosswalk location using data collected from a simulated roundabout in the U.S. The entering capacity was analyze with (i) four pedestrian volumes; and (ii) four different pedestrian crosswalk locations. The factors found in this research are compared with the factors used in HCM2010. A two-lane circulating flow and a two-lane entry approach have been selected as the roundabout geometry to conduct this research. A traffic simulator will be used to replicate pedestrian and driver behavior at the roundabouts.

A four-leg roundabout was selected from the National Cooperative Highway Research Program (NCHRP) inventory. Gap acceptance parameters were calibrated and validated with data collected from two different peak hours of the selected roundabout. The calibrated and validated parameters were coded into a four-leg two-lane roundabout using a widely used microscopic traffic simulator. Simulation runs were made to study the effect of increasing pedestrian volume and crosswalk position on the approach capacity. Four crosswalk positions and five pedestrian volumes were used in the simulation scenarios.

1.3 Thesis Organization

Chapter 1 of the thesis introduces the need to study the pedestrian effects on roundabout capacity. The literature review in Chapter 2 examines the roundabout information available, software capabilities and research that has been done in the analysis of roundabout with the

presence of pedestrians using microscopic traffic simulation tools. Chapter 3 presents the research methodology for the subsequent steps of this research. Chapter 4 covers the site selection, data collection and simulation model coding process. The statistical analysis performed during the model calibration and validation is described in Chapter 5. Chapter 6 describes the different scenarios developed to integrate pedestrian crossing into the roundabout entry capacity model. Chapter 7 details the vehicle entry capacity captured during the simulation and the capacity adjustment factors estimated to account for the pedestrian presence. Finally Chapter 8 makes recommendations based on the outcomes of this research.

Chapter 2: Literature Review

2.1 Roundabout Research in U.S.

During the last two decades modern roundabouts have gain popularity in the U.S. as an efficient and safe form of traffic intersection. Research has supported the argument that modern roundabouts reduce the number of accidents and severity for both drivers and pedestrians (Persuad, et al., 2001). These modern roundabouts differ from the conventional roundabouts as they have incorporated many features from the U.K. design of the 1960s (ITE, 1992). Modern roundabout uses a yield control at entry lanes: vehicles yield instead of stopping as happens in a regular four-way stop controlled or signalized intersection. This provides a more efficient intersection control by maintaining a constant traffic flow with fewer stops, enabling a reduction in travel time and delay and an improvement in the level of service. The constant vehicle flow improves air quality because vehicles stop less frequently. A properly designed roundabout also provides a traffic claiming effect as it reduces vehicles speed (Troutbeck, 1998).

Roundabouts first appeared in the U.S. in the west coast with the Columbus traffic circle at New York City in 1904(Lounsbury, 2004). Documents of roundabout design, operation or functionality (concerning, for instance, the capacity reduction associated with the presence of pedestrians at a roundabout) were limited or non-existent for many years. It was not until 1995, for instance, that the states of Maryland and Florida started to document roundabout design and operation guidelines. For a long time transportation professionals had to rely on foreign research, mainly from Germany, Australia, U.K. and other countries to design or analyze roundabouts(FHWA, 2000).

In 2000, the Federal Highway Administration (FHWA) issued *The Roundabout: an Information Guide* (FHWA, 2000). This was the first attempt at the national level to document safety, operational and design practices related to roundabouts in the U.S. The pedestrian effects on entry capacity are mentioned, citing capacity reduction factors from the German research report "Safety and Capacity of Roundabout" (Sicherheit und Leistungsfähigkeit von Kreisverkehrsplätzen) developed by Brilon, Stuwe and Drews in 1993 (FHWA, 2000). In 2007, NCHRP published Report 572 *Roundabouts in the United States* as part of the NCHRP Project 3-65. This is most comprehensive report in the U.S. that documents many aspects of roundabout safety and operation. The pedestrian section emphasizes the interaction between vehicles and pedestrians, but does not attempt to determine capacity reduction related to pedestrians. No concrete effort was focused on capacity reduction caused by pedestrians.

A master inventory from hundreds of roundabout all over the U.S. was assembled for NCHRP Project 3-65. The project aimed to develop the methods to measure safety, operational impacts and to improve design criteria of roundabouts in the U.S. (NCHRP , 2003). The project gathered geometry design, safety and operational data from the more representative roundabouts in the U.S. The NCHRP Project 3-65 gathered 923 one minute data points from two lanes sites concerning the entry flow, conflicting flow and mean delay. Also gathered were travel time from traffic movements, turning movement's proportions, and several parameters related to the gap acceptance theory (NCHRP , 2003).

The information gathered was used to analyze the operation, safety, and other roundabout issues. In NCHRP Report 572, different analytical models were compared to analyze which capacity model replicated more accurately to the roundabouts in the U.S. (NCHRP, 2007).

2.2 Capacity Concept

Empirical models and gap acceptance models have been developed to predict roundabout capacity(Troutbeck, 1998). The U.K. empirical model was developed using data collected at 86 roundabouts in the U.K. in the 1970s, and updated with more roundabout data in the subsequent years. On the other hand, the gap acceptance model for roundabout capacity was initially developed by J.C. Tanner in 1962 whose work estimated the probability of a vehicle in a minor road entering into a gap into the traffic of a main road (TRL). There are many parameters that could affect the roundabout capacity (such as driving behavior, driver familiarity, geometry, speed, etc) and furthermore the capacity could differ from each country(Troutbeck, 1998). The current capacity model has been refined and developed using data with roundabout in the U.S. (NCHRP, 2007).

The method to estimate capacity of roundabout in the U.S. is detailed in HCM2010. The roundabout capacity is determined at each of its approaches. The HCM2010 states that the roundabout capacity is directly related to three volumes: entry volume (υ_e), conflicting volume (υ_c), and exit volume (υ_{ex}) at each approach, see Figure 2.1. The basic principle is that roundabout approach capacity decreases as circulating volume increases (HCM, 2010).

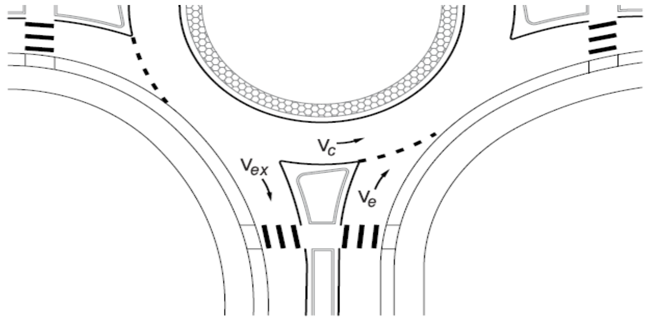

Figure 2.1: Roundabout traffic flow effect over capacity (from HCM2010).

The research presented in this document concentrates on the analysis of a two-lane entry approach that is in conflict with a two circulating lanes. The following paragraphs describe the entry capacity equations for single and multi lane roundabouts.

2.2.1 Single lane capacity

The NCHRP report 572 developed the entry capacity formula using regression analysis technique fitted to data collected in NCHRP Project 3-65. Figure 2.2 shows the data points and the fitted equation.

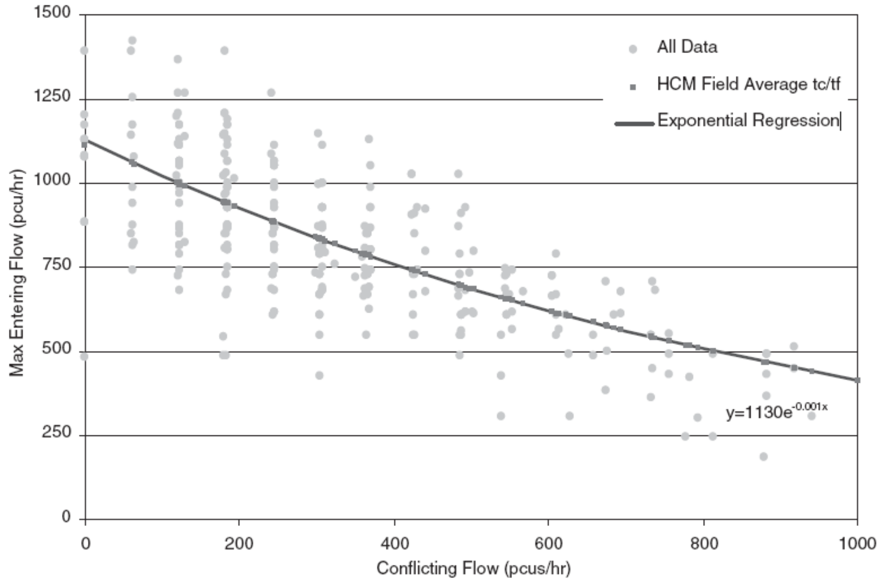

Figure 2.2: Single-lane roundabout capacity model and observations (from NCHRP Report 572).

The equation to estimate the capacity of a single lane entry conflicted by one circulating lane is shown in [2.1] where C_{pce} is the lane capacity adjusted for heavy vehicles, in passenger cars per hour (pc/h); and $\upsilon_{c,pce}$ *i*s the conflict volume, in pc/h.

$$C_{\mathrm{pce}}=1{,}130\ e^{\left(-1.0\mathrm{x}10^{-3}\right)\upsilon_{\mathrm{c,\ pce}}} \qquad [2.1]$$

2.2.2 Multilane Capacity

Multilane roundabout can have different configuration depending on the number of entry lanes and the number of circulating lanes. The HCM2010 addresses these conditions by providing three different equations:

1. The lane capacity of **two entry lanes conflicted by one circulating lane**:

$$C_{pce}=1{,}130\ e^{\left(-1.0x10^{-3}\right)\upsilon_{c,\ pce}} \qquad [2.2]$$

2. The capacity of a **one lane entry approach conflicted by two circulating lanes**:

$$C_{pce}=1{,}130\ e^{\left(-0.7x10^{-3}\right)\upsilon_{c,\ pce}} \qquad [2.3]$$

3. For an approach with **two entry lanes facing two circulating lanes,** the capacity of an entry lane is given by [2.4], and the left entry lane by [2.5], where $C_{e,R,pce}$ is the right entry lane capacity, and $C_{e,R,pce}$ is the left entry lane capacity. The $\upsilon_{c,pce}$ is the conflict volume of both circulating lanes, in pc/h.

$$C_{e,R,pce}=1{,}130\ e^{\left(-0.7x10^{-3}\right)\upsilon_{c,\ pce}} \qquad [2.4]$$

$$C_{e,L,pce}=1{,}130\ e^{\left(-0.75x10^{-3}\right)\upsilon_{c,\ pce}} \qquad [2.5]$$

Figure 2.3 exhibits an approach with two lane entries conflicted by the two circulating lanes.

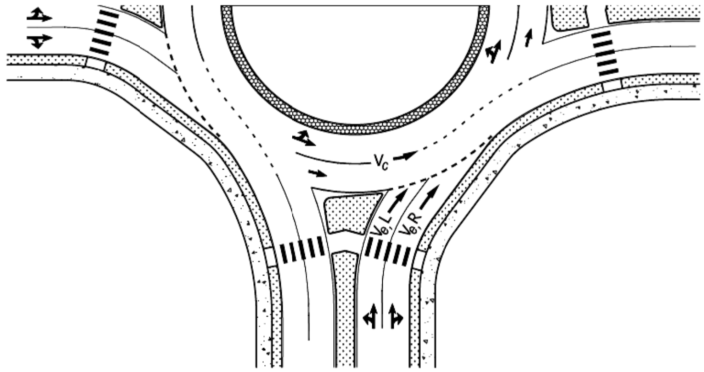

Figure 2.3: Two-lane entry and two-lane circulating roadway.

These models were developed using observation taken from roundabouts as part of NCHRP Project 3-65. Figure 2.4 exhibits the critical right and left lane observations and the regression models for multilane roundabouts in NCHRP Report 572.

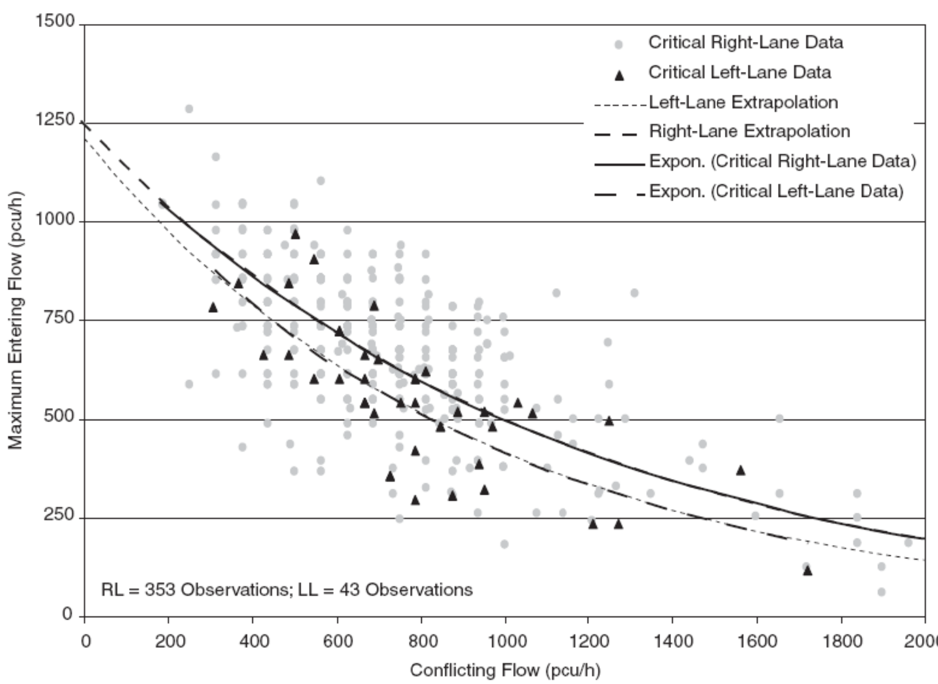

Figure 2.4: Multi-lane roundabout capacity model and observations (from NCHRP Report 572).

2.3 Pedestrian Impedance to Vehicles

Pedestrians functions as an additional conflicting "vehicle" to the entry vehicles. Therefore at high pedestrian volume the pedestrian traffic can reduce the vehicular capacity at an entry lane. In order to adjust the vehicular entry capacity of a roundabout (according to the number of pedestrian crossing per approach) the HCM2010 uses the adjustment factor as shown in Figure 2.5. and Figure 2.6.

Figure 2.5 shows the entry capacity adjustment factor for pedestrians crossing a one lane roundabout approach. The pedestrian reduction factor curves merge at high conflicting volume. Under this condition pedestrians are assumed to pass between vehicles waiting in the queue and therefore have a negligible impact in vehicular entry capacity (HCM, 2010).

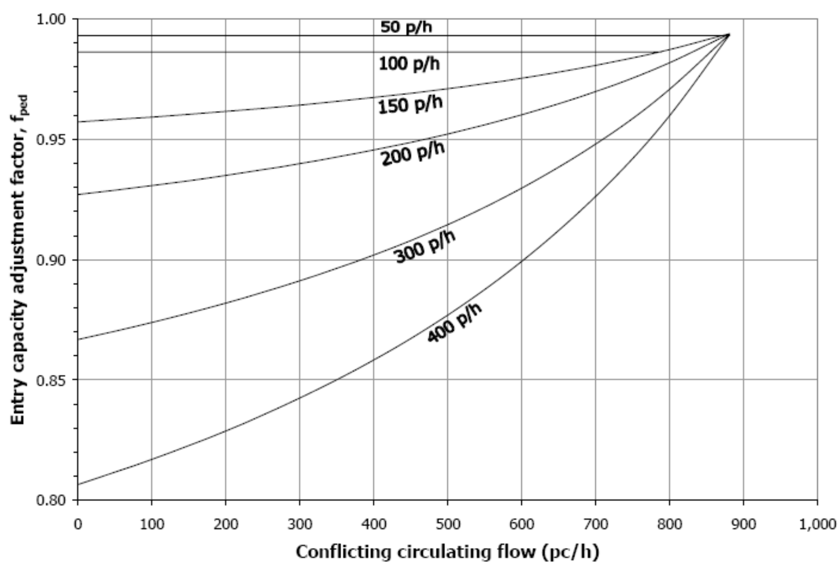

Figure 2.5: Capacity adjustment factor for pedestrians crossing one lane entry (from HCM2010).

The Figure 2.6 illustrates the entry capacity adjustment factor for pedestrians crossing a two-lane entry approach. The models developed to predict these capacity adjustment factors assumed that pedestrians have absolute priority (HCM, 2010).

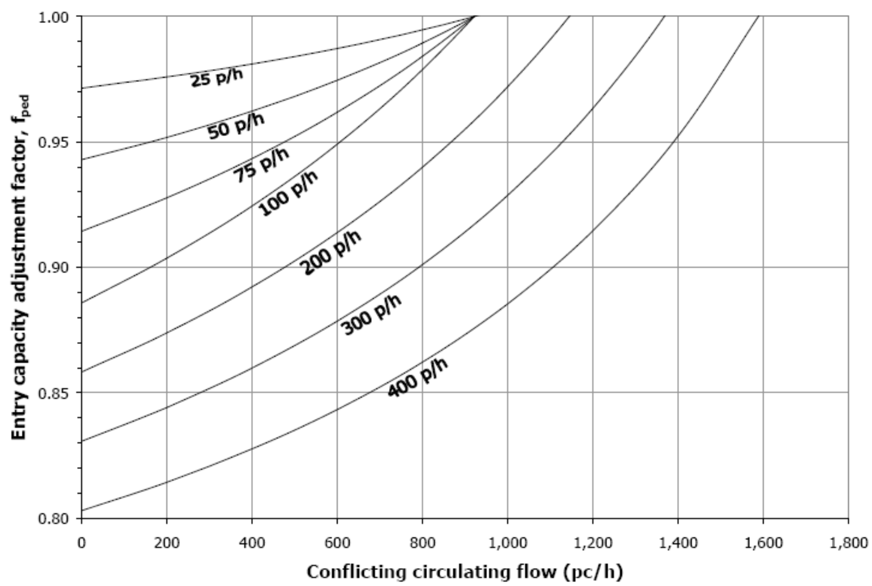

Figure 2.6: Capacity adjustment factor for pedestrians crossing two lane entry (from HCM2010).

2.4 Simulation Software

Traffic simulation tools have been gaining popularity in the past decades among researcher and practitioners. Newer and more advanced capabilities of these simulation tools allow engineers to model a range of scenarios, from single intersections to more complex and larger networks. Traffic simulation creates a scenario in which results can be gathered relatively quickly. Therefore it is valuable to know which simulation tool is most applicable, based on the software capabilities and limitations of each. RODEL, SIDRA and VISSIM are among the top

software choices to simulate roundabouts. Three programs were compared for the applicability in simulating roundabouts with pedestrian crosswalks.

2.2.3 RODEL

RODEL is a software that uses empirical equations to predict traffic performance on roundabouts. The program was originally developed in a Disk Operating System (DOS) environment by Barry Crown in the 1980s. It uses empirical lane capacity equations developed by the British Transport and Road Research Laboratory (TRRL). Later the software evolved into RODEL V2, a Windows application with new updates to correct estimates to the queue and delay of the empirical equations (Stanek, et al., 2005).

The software allows for a variety of geometric layouts, such as single, double, triple and quadruple lane roundabouts, as well as peak hour flow and traffic flow. The software can incorporate bypass lanes, and geometric delays. RODEL allows users to facilitate the optimization of roundabout design. The scenarios are developed by changing the input parameters to test the design features that optimize roundabout capacity (Stanek, et al., 2005).

2.2.4 SIDRA Intersection

SIDRA stands for Signalized and Unsignalized Intersection Design and Research Aid It is a evaluation-analysis tool that uses vehicle drive cycle and lane by lane models (Akcelik, 2003). SIDRA Intersection 3.0 is the latest version for the Windows environment. SIDRA uses the gap acceptance theory, and was developed with field surveys of roundabouts in Australia. Its

unique feature is that lane capacity automatically changes with the user defined roundabout geometry and demand flow levels. SIDRA also takes into consideration the utilization of circulating lanes, approach queuing prior to entering the roundabout, as well as demand flow patterns and total circulating volumes (Akcelik, 2003).

2.2.5 VISSIM

VISSIM is a microscopic, time stepping and behavior-based simulation model developed to model urban traffic and public transportation operations. VISSIM uses a discrete, stochastic, time-step based microscopic model. The traffic flow is control by car following models and a rule-based algorithm for lateral movements (PTV, 2009).

VISSIM is extremely flexible and it is able to simulate different roadways facilities with various transportation modes such as light duty vehicle, heavy duty vehicles, buses, light rail, heavy rail, pedestrians and bicyclists. The software was developed in Germany at the University of Karlsruhe during 1970s and it is distribute by PTV AG (Trueblood, et al., 2003).

VISSIM network are coded using links and connectors. Links are used to simulate main roadway segments. Two links and connectors give users the flexibility to the users to simulate any kind of traffic behavior (Trueblood, et al., 2003). VISSIM is recommended among other software to simulate roundabouts when over saturated conditions and/or unique street geometry are present (Stanek, et al., 2005).

Chapter 3: Research Methodology

The analysis of a roundabout's entry lane capacity with the presence of pedestrians consisted of five major steps. The methodology described here follows the microscopic traffic simulation modeling guidelines established by the Federal Highway Administration (FHWA 2004). Figure 3.1 illustrates the five main steps used for the microscopic traffic simulation modeling: (1) project scoping, (2) initial traffic simulation model, (3) calibration process, (4) validation process, and (5) pedestrian model.

In the first step, the project scope was determined. The objective of the research is to analyze the effects of pedestrian on roundabout entry capacity. The effects of pedestrians are analyzed with respect to two factors: pedestrian volume and crosswalk location. Microscopic traffic simulation software was selected based on its ability to replicate geometry, traffic patterns and pedestrian behavior. Site selection was based on the best available data for model calibration and validation.

The second step, initial traffic simulation modeling, involved the data collection of the roundabout selected in step one. The data collected included geometry, traffic volumes, traffic movements, traffic composition, travel times, speed, lane selection and lane usage.

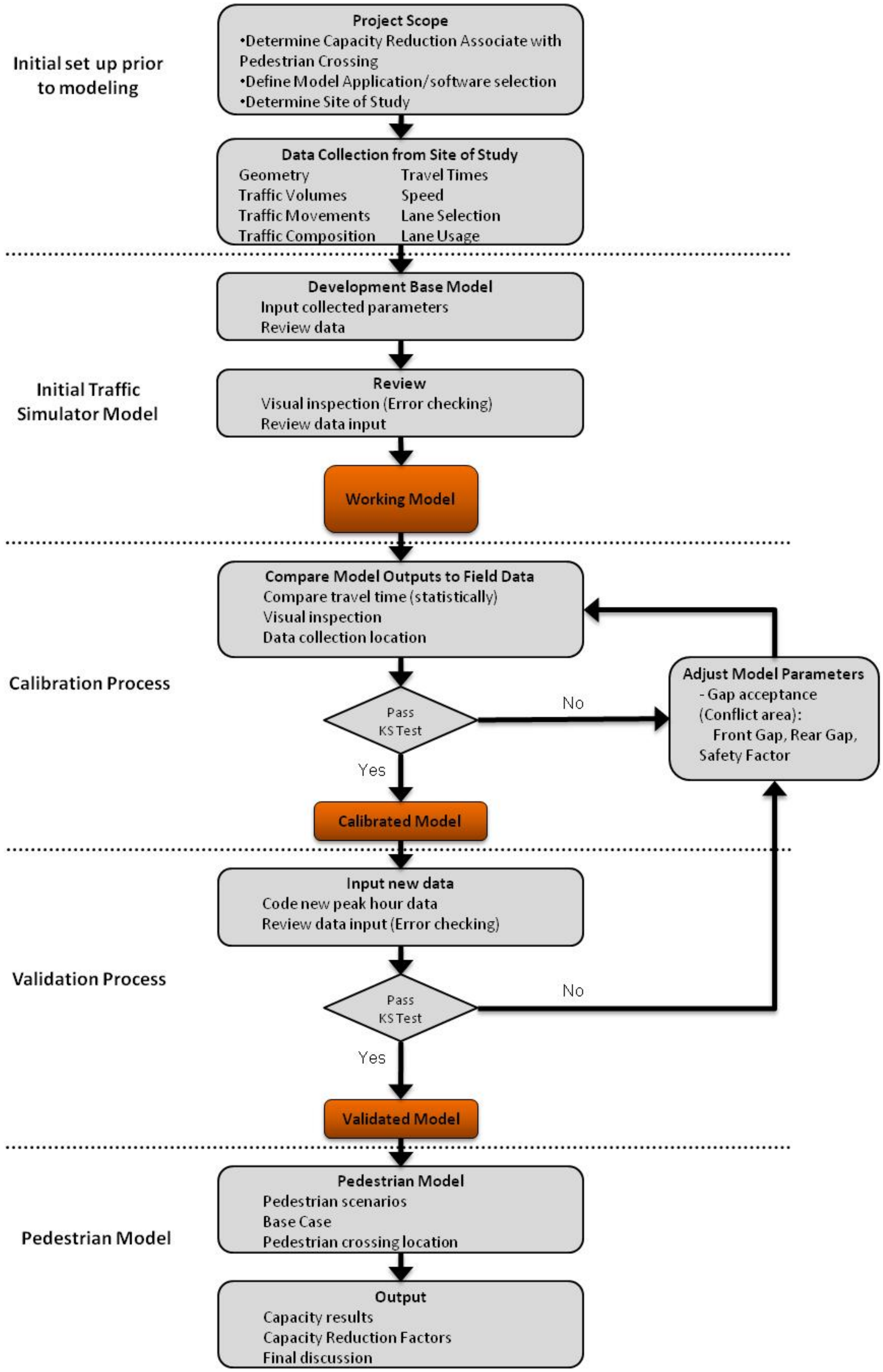

Figure 3.1: Microscopic traffic simulation model development.

The third and fourth step, model calibration and validation process, involved the development of an initial simulation model. A model was developed that incorporated all the site features and data. The initial model was thoroughly reviewed to detect coding errors, until a working model was found to replicate the drivers' behavior reasonably. During the calibration and validation processes, the Measure of Effectiveness (MOE) — in this case the travel time — was selected for the comparison between the travel time data collected at the actual site with the VISSIM model's travel time output. The Kolmogorov-Smirnov (K-S) test was used to determine if the difference between the VISSIM output and the real data is statistically significant. Conflict area parameters in VISSIM were adjusted until the simulation model's output passed the K-S test. Once calibrated, the model was tested with a new set of data for validation. During the validation process, the conflict area parameters were adjusted again, if necessary. If any adjustment to the parameters was made, the new parameters values were re-coded in the calibration model until the same set of parameters values enable the model to pass the calibration and validation tests.

With the calibration and validation process completed, a new and typical roundabout model was created using the calibrated parameters. Pedestrian presence scenarios were developed, simulation runs performed and results analyzed. As a control experiment, the VISSIM model was run without a pedestrian crosswalk.

Finally the results were analyzed in terms of the effect of (i) pedestrian volume; and (ii) the crosswalk position on capacity reduction at a roundabout entrance. The final results show a set of capacity adjustment factors developed for each scenario.

Chapter 4: Model Development

In order to develop an accurate microscopic traffic simulation model, a representative roundabout was selected from the available sites featured in NCHRP Project 3-65. Once the site was selected, data were extracted and coded into a VISSIM model to replicate the operational and driving behavior in the roundabout. This chapter describes the before and during the model development.

4.1 Site Selection

A site was selected from the inventory 310 roundabouts in NCHRP Report 572. The site inventory lists the roundabouts previously used in NCHRP Project 3-65. The inventory attributes include information on the geometry (diameters; entry and exit widths, angles, radii, and flares; circulating roadway widths; intersection sight distances; treatments for bicycles and pedestrians, markings, and signs), safety records, traffic characteristics as well as site characteristics (rural, urban, and suburban) (NCHRP, 2007).

Specifically, the inventory details used in the site selection for model calibration and validation included the following attributes: site ID, state, county or borough, intersection, setting, number of legs (approaches) and number of lanes. The three main attributes established for the first round of selection were: four-leg roundabout with two circulating lanes in an urban setting. The number of sites was reduced from 310 roundabouts to 104 with an urban setting, 76 with an urban setting and four approaches, and 22 with an urban setting, four approaches and two lanes. These 22 sites listed in Table 4-1 were analyzed individually to refine the selection.

Table 4-1: Preliminary site selection.

Site ID	State	City	County/ Borough	Intersection	Setting	No. of Legs	No. of Lanes
CO02	CO	Golden	Jefferson	South Golden Road/Johnson Rd/16th Street	Urban	4	2
CO03	CO	Golden	Jefferson	South Golden Road/Utah St.	Urban	4	2
CO07	CO	Avon	Eagle	Avon Rd./Benchmark Road	Urban	4	2
CO08	CO	Avon	Eagle	Avon Rd./I-70 Eastbound Ramp	Urban	4	2
CO09	CO	Avon	Eagle	Avon Rd./I-70 Westbound Ramp	Urban	4	2
CO10	CO	Avon	Eagle	Avon Rd./U.S. Hwy 6	Urban	4	2
CO37	CO	Golden	Jefferson	South Golden Road/ Lunnanhaus	Urban	4	2
CO42	CO	Loveland	Larimer	Rocky Mountain Ave/Fox Trail	Urban	4	2
CO43	CO	Loveland	Larimer	Rocky Mountain Ave/McWinney	Urban	4	2
FL16	FL	Lake Worth	Palm Beach	Lakeworth Ave (SR 802)/South A Street	Urban	4	2
IN05	IN	Indianapoli s	Marion	Monument Cir. (Market/Meridian)	Urban	4	2
KS01	KS	Olathe	Johnson	Sheridan St./Rogers Rd	Urban	4	2
KS17	KS	Topeka	Shawnee	I-70 EB Ramps/Rice Rd/Cyprus Dr	Urban	4	2
KS18	KS	Topeka	Shawnee	I-70 WB Ramps/Rice Rd/Sycamore Dr	Urban	4	2
MD04	MD	(unincorpo rated)	Baltimore	MD 139 (Charles St.)/Bellona Ave	Urban	4	2
MD08	MD	Annapolis	Anne Arundel	MD 450/Spa Rd./Taylor Ave (Annapolis Gateway)	Urban	4	2
MI03	MI	East Lansing	Ingham	Bogue Street/Shaw Lane	Urban	4	2
NV09	NV	Las Vegas	Clark	Carey Ave/Hamilton St	Urban	4	2
NV10	NV	Las Vegas	Clark	Carey Ave/Revere St	Urban	4	2
TX01	TX	Addison	Dallas	Mildred St./Quorum Dr	Urban	4	2
UT01	UT	Orem	Utah	1200 South/400 West	Urban	4	2
UT02	UT	Orem	Utah	2000 South/Sandhill Rd	Urban	4	2

Aerial pictures gathered from Google Earth were used to visually inspect the candidate roundabout locations and surroundings. Roundabouts located at a relatively close proximity to other roundabouts, and those with signalized or stop controlled intersections and major

driveways were discarded due to their unknown effect imposed by local traffic patterns. Also, roundabouts with angles of less than 30 percent between two approaches were not preferable.

One of the major concerns during site selection was to select locations in which clear aerial photographs were available. The VISSIM model relies on a correct replication of the geometric design in order to measure travel time and other MOEs. The above critieria further reduced the selection to two sites: the roundabout located in Golden, Colorado (Site ID CO-02) and the one in Olathe, Kansas (Site ID KS-01). The KS-01 roundabout was finaly selected due to the better quality of video recording available from NCHRP Project 3-65.

Figure 4.1 shows an image of the video recorded using an omni-directional camera on top of the roundabout located at Olathe, Kansas. The extra space video recordings were provided by the NCHRP Project 3-65 research team in DVD format. The recordings were made from 9:31 a.m. to 12:41 p.m., and from 2:20 p.m. to 4:09 p.m. on July 25, 2003. From preliminary traffic counts peak hour were found at 10:30 a.m. to 11:30 a.m. (morning peak) and at 3:30 p.m. to 4:30 p.m. (afternoon peak).

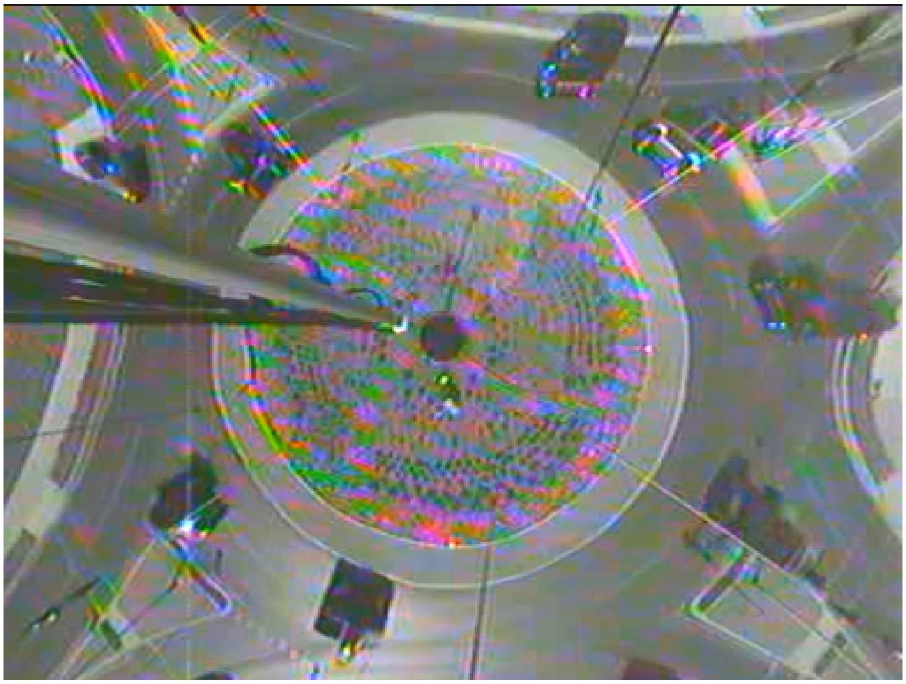

Figure 4.1: Omni-directional camera located at top of roundabout in Olathe, Kansas.

4.2 Data Collection

Using the video recordings, the following data were extracted to setup the simulation model: volume counts, turning movements, traffic composition, travel times, lane use and lane selection. All the data were collected for the morning and afternoon peak hours.

Heavy vehicles accounted for 1.7% and 2.5% of the vehicle composition during the morning and afternoon peak hours, respectively. Heavy vehicles were classified as any vehicles larger than a motorcycle, car or pick-up truck. There was only a single observation of a motorcycle, which was counted as a regular vehicle.

Turning counts were obtained for each approach of the roundabout. Table 4-2: Roundabout traffic volume and composition, morning peak.

Approach	Passangers Vehicles (veh/h)				Heavy Vehicles (veh/h)	
	Right	Through	Left	Total	Total	Percent
Southbound	141	137	13	291	0	0
Eastbound	228	321	69	618	23	0.65
Westbound	10	202	43	255	4	9.02
Northbound	72	113	171	356	11	3.09
Total	451	773	296	1520	38	2.50

Table 4-3 and Table 4-2 show the traffic volume for all the movements per approach as well as heavy vehicle percentages.

Table 4-2: Roundabout traffic volume and composition, morning peak.

Approach	Passangers Vehicles (veh/h)				Heavy Vehicles (veh/h)	
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Southbound	141	137	13	291	0	0
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Westbound	10	202	43	255	4	9.02
Northbound	72	113	171	356	11	3.09
Total	451	773	296	1520	38	2.50

Table 4-3: Roundabout traffic volume and composition, afternoon peak.

Approach	Passangers Vehicles				Heavy Vehicles	
	Right	Through	Left	Total	Total	Percent
Southbound	90	97	7	194	0	0
Eastbound	165	196	43	404	13	0.50
Westbound	13	213	64	290	2	4.48
Northbound	48	130	149	327	6	4.83
Total	316	636	263	1215	21	1.73

Point to point travel time data was gathered in both peak hours. The travel time was recorded from the instant the rear bumper of a vehicle entered the screen until the same bumper left the screen of the recording. Table 4-4 and Table 4-5 show the number of observation and the statistics of travel time collected for the four through movements.

Table 4-4: Statistics of travel time, morning peak.

Through Movement	No. observed	Travel time (sec)		
		Min	Max	Average
Northbound	61	5.6	15.5	8.8
Southbound	43	6.6	15.9	10.2
Eastbound	47	5.7	14.3	8.5
Westbound	49	6.6	28.8	12.7

Table 4-5: Statistics of travel time, afternoon peak.

Through Movement	No. observed	Travel time (sec)		
		Min	Max	Average
Northbound	54	5.41	29.13	11.64
Southbound	69	7.54	34.75	14.01
Eastbound	135	5.91	23.62	9.46
Westbound	108	8.28	33.19	14.08

Lane use refers to the lane a vehicle used when entering and exiting the roundabout, and in the circulatory roadway. Lane use was determined by observing vehicles' lane preferences at the entry and exit of every approach. This data collection effort required the coordination of two observers and low speed video playback. Table 4-6 and Table 4-7 show the usage of each lane at the entry and exit approaches.

Table 4-6: Lane changing, morning peak.

Exit Lanes	Left	Right	Right	Left
Entry Lanes	**Left**	**Left**	**Right**	**Right**
Southbound	50%	0%	50%	0%
Eastbound	64%	29%	7%	0%
Westbound	74%	21%	5%	0%
Northbound	65%	17%	19%	0%
Average	**63%**	**17%**	**20%**	**0%**

Table 4-7: Lane changing afternoon peak.

Exit Lanes Entry Lanes	Left Left	Right Left	Right Right	Left Right
Southbound	0%	0%	100%	0%
Eastbound	14%	5%	81%	0%
Westbound	57%	8%	33%	2%
Northbound	4%	4%	88%	4%
Average	**19%**	**4%**	**75%**	**1%**

Lane selection affects vehicle operation and safety in roundabouts (FHWA, 2000). For example, double left-turn is not allowed in a roundabout. The 2009 edition of Manual on Uniform Traffic Control Devices (MUTCD) addresses this issue by requiring that drivers choose a lane before entering a roundabout. Lange changing activity can also increase or decrease the travel time, or interfere with other vehicles. At the selected site, right and left turn movements were observed to determine if drivers maintained proper lane choice. Drivers making a right-turn tended to stay in the right lane of an approach, while those making a left turn tended to stay in the left lane of an approach. This behavior was observed at the site in nearly all the turning movements.

The through traffic has four lane use options: enter from the left lane (of an approach) and exit to the left lane (of an approach); enter from the left lane and exit to the right lane; enter from the right lane and exit to the right; and enter from the right and exit to the left. Most of the vehicles entered and exited in the same lane. During the morning peak, 63% of drivers used the left lane and 24% used the right lane. Only 17% switched lanes from the left lane to the right lane, and 0% switched from the right lane to the left lane. The afternoon peak hour also reflects

the preference to stay in the same lane, with 19% of traffic used the left lane and 75% used the right. Only 4% and 1% of the vehicles changed lanes from left to right, and right to left respectively.

Table 4-8: Lane use in morning and afternoon peak hours.

Lane behavior	Entry Lane Left Lane	Right Lane
Stay in same lane	41%	48%
Merge into other lane	10%	1%
Total	51%	49%

As shown in Table 4-8 the lane usage obtained from the entry lane shows a 51 % left lane preference and 49 % right lane preference.

4.3 Model Coding

The initial model was developed following the standard procedures as described in the VISSIM 5.20 User Manual(PTV, 2009). The following list briefly describes the model set up:

1. The simulation parameters were set up to run for 3600 seconds (one-hour simulation). The simulation resolution was set to 10 time steps per simulated second. The resolution was never changed to ensure consistency in all models.
2. The speed was set up accordingly to the speed data measured (from the video recordings) at the inner circulating lanes and through movements.

3. The vehicle characteristics such as acceleration, deceleration, torque and other vehicle attributes were set as their default values provided by VISSIM.
4. The traffic composition was setup to replicate the vehicle composition observed in the videos.
5. The roundabout's basic geometry was coded from the aerial images obtain from Google Earth. Two major images were used in this process. A lower quality image taken at a higher altitude was used to generate an overview of the area. Then, a high quality image taken at a lower altitude was obtained with Google Earth.
6. The scale of both images was based on the known distance between objects located at the bottom of both images. The scale of the image is extremely important to accurately represent the real distance of the whole network. A basic skeleton of the network was drawn and redrawn several times to recreate the geometric layout. Figure 4.2 shows the link and connectors which replicate the entry lanes and circulating lanes of the roundabout.

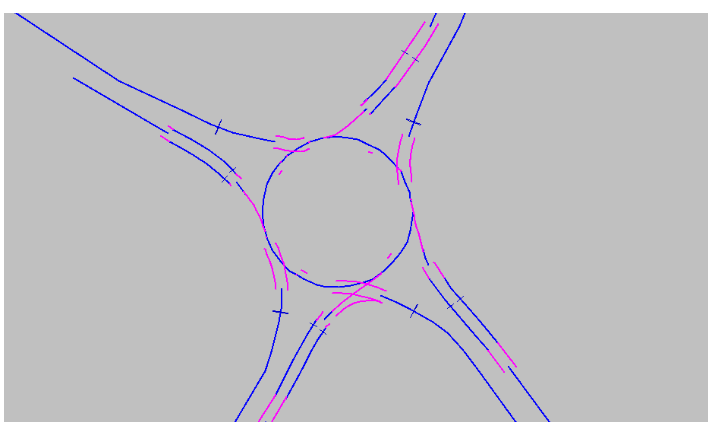

Figure 4.2: VISSIM model links and connectors.

7. All vehicle volumes were coded to appear at the network entry points.
8. Four routing decisions points were used to code the driving direction of vehicles in the network. Right and left turn vehicles were coded to use only the right and left approach lanes respectively. Lane use and lane selection were edited in the network to follow the observe behavior as described in the previous subsection.
9. Three speed limits were coded in the network: general speed limit at the approach links, speed limit at the circulating lanes, desired speed distribution for the vehicle composition. In addition, reduced speed areas were placed at roundabout entrances.
10. Conflict areas were used to model the gap acceptance behavior at roundabout entrances. The gap acceptance parameters available in the conflict areas are visibility, front gap, rear gap, safety distance factor, additional stop distance, observed adjacent lanes, anticipated routes and avoid blocking.

    Visibility needs to be considered when buildings or any other object limit the driver's view. The visibility was left at the default value (328 ft or 100 m) since the aerial images suggest no visible obstruction from driver's view.

    The front gap is defined as the "minimum gap in seconds between the rear end of a vehicle on the main road and the front end of a vehicle on the minor road" (PTV, 2009).

    Rear gap is defined as the "minimum gap in seconds between the rear end of a vehicle on the minor road and the front end of a vehicle on the main road" (PTV, 2009)

    The safety distance factor is compared with the normal desired safety distance between vehicles on the main road to determine the minimum headway that a vehicle from the minor road must provide at the moment when it is completely inside the merging conflict area(PTV, 2009).

For the calibration and validation, the three parameters (front gap, rear gap, safety distance factor) were deemed to affect the gap acceptance behavior and hence the travel time at the roundabout.

The parameters observe adjacent lanes, anticipate routes and avoid blocking were activated. The additional stop distance option was not activated since the model did not require any additional distance other than that already provided by the default behavior model.

11. Finally, the travel time sections were designated to collect travel time and delay data from the required movements.
12. Pedestrian crosswalks were added after model calibration and validation. The pedestrian crosswalk was set up at 20 ft (one car length), 40 ft (two car length), 60 ft (three car length), and 80 ft (four car length) upstream from the yield line in an approach. This information will be further discussed in Chapter 6.

The model was thoroughly reviewed to detect any coding error. Once the working model was found to replicate the drivers' behavior reasonably, model calibration was carried out. The model calibration and validation are discussed in the next chapter.

Chapter 5: Model Calibration and Validation

The following section describes the MOE selected to compare the data collected at the Olathe, Kansas roundabout with the outputs generated by the VISSIM model, the statistical method used to measure the significant difference between the observed field data and the simulation output, and the calibration process.

Travel time, delay and queue are among the preferable performance measures to calibrate and validate a model (FHWA, 2003). During data collection (as described in Chapter 4), there was no obvious indication of large queue. Travel time seemed to offer the best option to calibrate and validate the set of data described in Chapter 3.

5.1 Kolmogorov-Smirnov Test

The Kolmogorov-Smirnov (K-S) test was used to determine if the travel times distributions between the VISSIM outputs and the observed values from the video recording were significantly different. The K-S test quantifies the maximum discrepancies D_n, of the two cumulative distribution functions (Ang, et al., 2007). Figure 5.1 shows the sample distribution (in this research VISSIM outputs) and theoretical distribution (site data as observed from the video recordings) used in the K-S test.

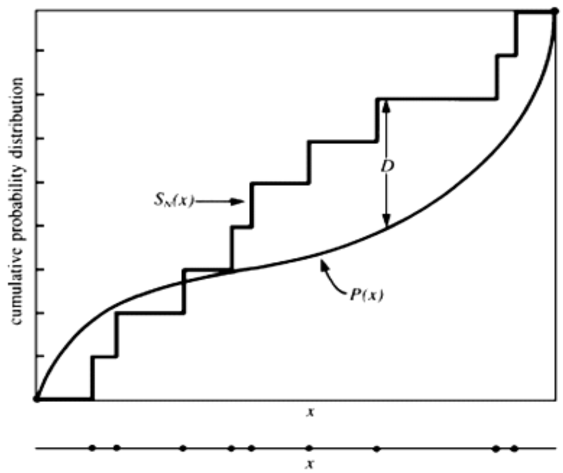

Figure 5.1: Cumulative distributions for K-S test (from Ang, et al., 2007).

Equation [5.1] shows the maximum discrepancies between the cumulative probabilities. The D_n was compared to a critical value which is a function of the sample size.

$$D_n = max_x|F_x(x) - S_n(x)| \qquad [5.1]$$

The rear gap, front gap and safety distance factor were adjusted to replicate the behavior of vehicles entering the circulating lanes. The default values were 0.5 sec for rear gap, 0.5 for sec front gap, and a 1.5 for safety distance factor. These values were adjusted through a sensitivity analysis. Two values were fixed while the third one was increased or decreased at intervals of 0.1. The sample distribution plots displayed the impact of each factor on travel time, and the factors were adjusted until the approach's travel time distribution passed the K-S test. Since four conflict areas were coded, one at each entry approach, this process was conducted simultaneously at the four approaches. The final combination of values was determined as 0.6 sec rear gap, 1.5 sec front gap, and 0.9 safety distance factor.

5.2 Calibration

The model was tested using the afternoon peak data. Table 5-1 shows the critical values and the number of observation used during the model calibration. Appendix A shows the complete set of figures plotted for the K-S test (one figure for each approach).

Table 5-1: K-S test critical values, afternoon peak.

Through Movements	Number of Observations	Critical Value
Southbound	69	0.16
Eastbound	135	0.12
Westbound	108	0.13
Northbound	54	0.19

Figure 5.2 illustrates the distribution obtained from VISSIM outputs after calibration and the sample distribution obtained from the video recordings during the afternoon peak hour at the westbound approach.

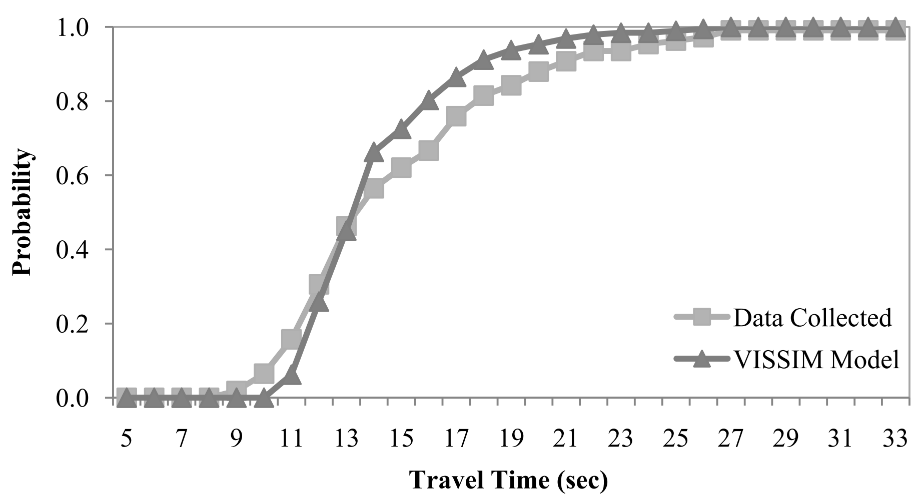

Figure 5.2: K-S test, westbound after calibration, afternoon peak.

5.3 Validation

The model was tested using a new set of data (morning peak), following the same K-S test procedure as applied in the calibration data.

Table 5-2 shows the critical values and the number of observations used during the validation of the model. Appendix A show the complete set of figures for the K-S tests for the purpose of model validation.

Table 5-2: K-S test critical values, morning peak.

Through Movements	Num. of Observations	Critical Value
Southbound	43	0.21
Eastbound	47	0.19
Westbound	49	0.19
Northbound	61	0.17

Figure 5.3 illustrates the distribution obtained from VISSIM outputs and the distributions obtained from the video recordings during the morning peak hour.

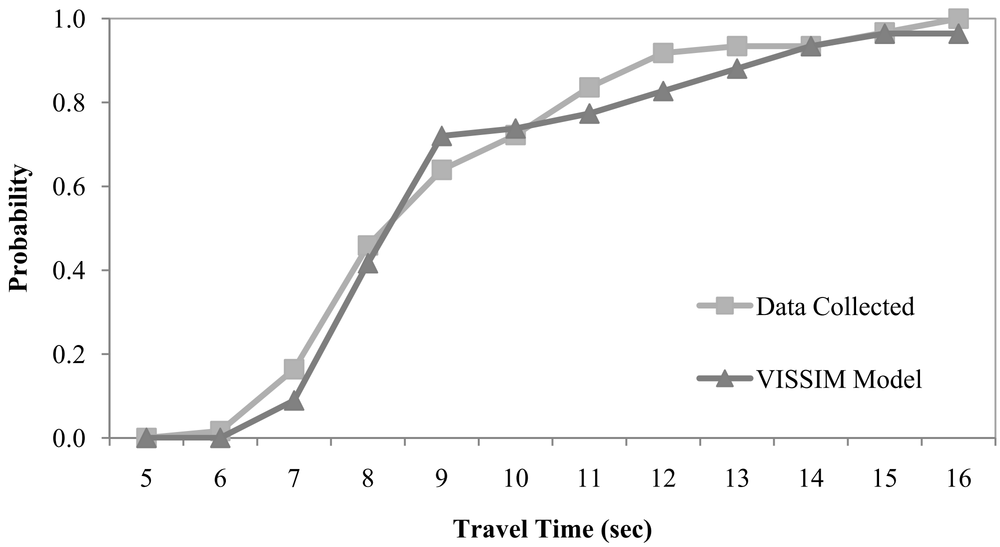

Figure 5.3: K-S test, northbound after validation, morning peak hour.

Chapter 6: Pedestrian Modeling

The initial roundabout model was developed using a four-leg two-lane roundabout located in an urban setting (as described in Chapter 4). The parameters associated with the conflict area were calibrated with two sets of peak hours, data in Chapter 5. These calibrated parameters were used in a new roundabout model. The new model was created based on a standard roundabout design recommended in the 2009 edition of MUTCD.

Figure 6.1 illustrates an example of markings for the approach and circulatory roadways at a roundabout, this figure is taken from the 2009 edition of MUTCD and has the best illustration for the crosswalk, although the approach has only one entry lane. Lane marking for pedestrian crosswalk and pedestrian volumes were added to the new model in VISSIM to conduct simulation experiments. The modeling scenarios are presented in this chapter.

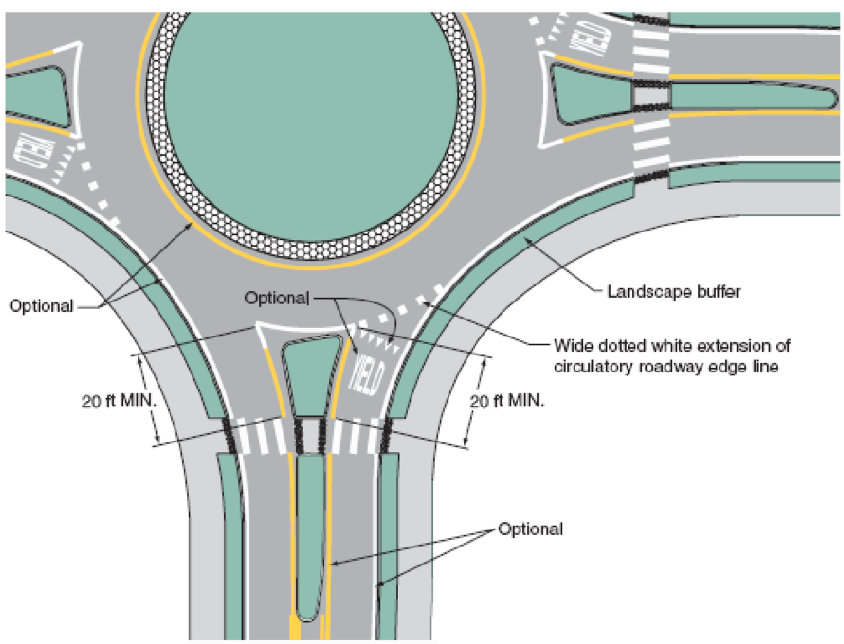

Figure 6.1: Example of roundabout markings recommended by MUTCD.

6.1 Crosswalk Positions

Four crosswalk positions were tested in this research. The crosswalk positions were located at 20 ft, 40 ft, 60 ft and 80 ft upstream of the yield line at an entry approach, as illustrate Figure 6.2 and Figure 6.3. The distances correspond to 1, 2, 3 and 4 car lengths respectively. Figure 6.2 shows the layout of the pedestrian crosswalk at 20 ft upstream from the yield line in the VISSIM model.

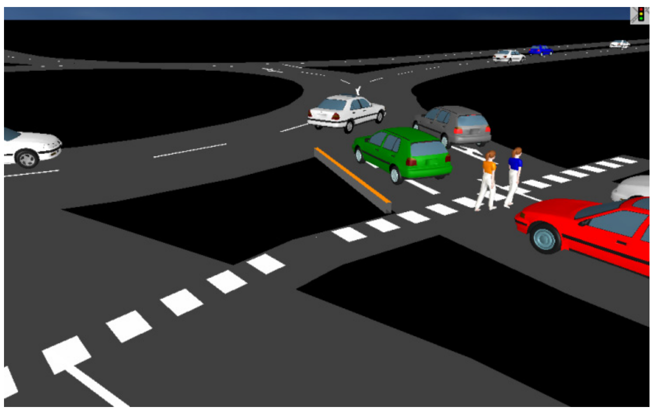

Figure 6.2: Pedestrian crosswalk at a 20 ft upstream from the yield line.

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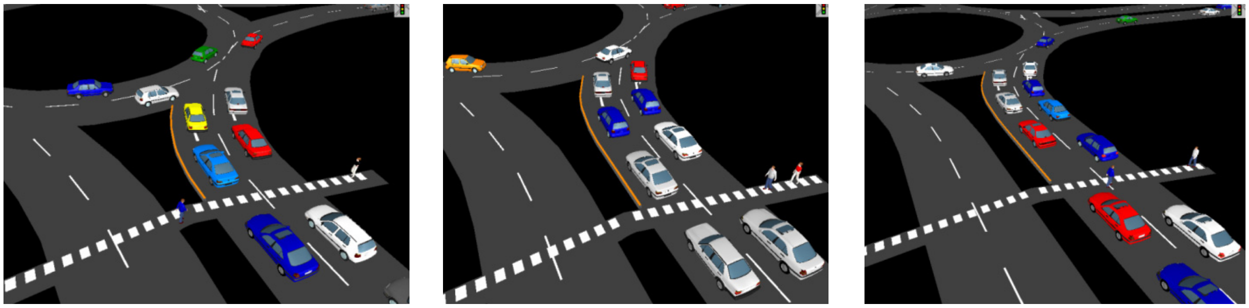
The pedestrian crosswalk was set as a two-way link with a width of 6 ft. The link covered two lanes of an entry approach. The pedestrians were generated at both ends of the two-way link, and they traveled in both directions.



(a) at 40 ft (b) at 60 ft (c) at 80 ft

Figure 6.3: Pedestrian crosswalks at 40 ft, 60 ft and 80 ft upstream from the yield line.

6.2 Pedestrian Walking Speed And Volume

The pedestrian volume described in the rest of the thesis is the combined volume in both directions. The volume is divided equally between the two directions. The pedestrians are represented as one type of vehicle in VISSIM. The 2009 edition of MUTCD recommends a walking speed of 4-ft/s in facility design. In order to represent a variation in pedestrian walking speed, the walking speed was edited in VISSIM to follow a probability distribution. Two distributions were used, one for young pedestrians (aged 60 and younger) and one for old pedestrians (older than 60 years) (TCRP, 2006). Figure 6.4 shows the speed distribution of senior and young pedestrians. The walking speed distributions were taken from NCHRP Report 562 and edited into VISSIM as vehicle speed.

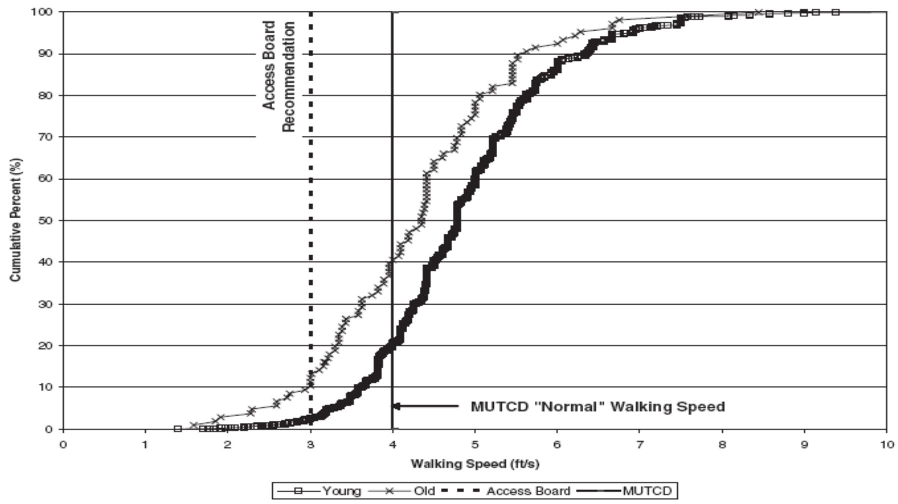


Figure 6.4: Pedestrians walking speed distribution (from NCHRP Report 562).

The proportion of senior to young pedestrians was estimated from the national age statistic. The total population younger than 60 years old add up to 83.73% and the population older than 60 years old is 16.27% (Census 2000). The pedestrian composition was set up to 84% and 16% to represent the younger and senior pedestrians, respectively. Figure 6.5 show the edited walking speed distributions for senior and young population.

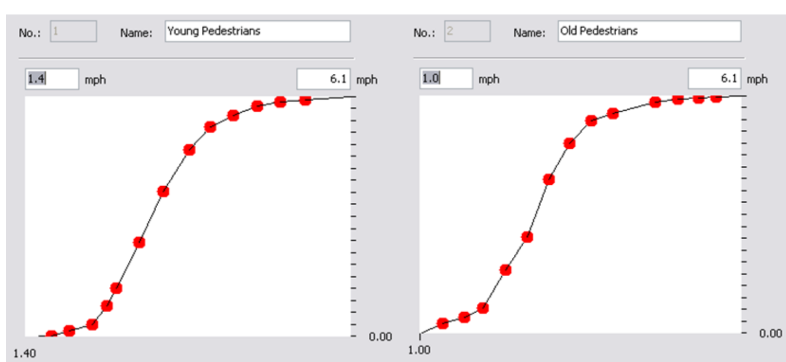


Figure 6.5: Pedestrians walking speed distribution in VISSIM

6.3 Pedestrian Volumes

The simulation of pedestrians was set from 100 pedestrians per hour (p/h) to 400 p/h at intervals of 100 p/h. Each of the scenarios described in Chapter 5 was run with 0, 100, 200, 300, and 400 p/h. The volumes were selected to covers the range used in HCM2010 capacity adjustment factors (see Figure 2.5 and Figure 2.6). The interaction of pedestrians and vehicles in the VISSIM model are illustrated in Figure 6.6. Conflict areas were used to simulate the interaction of pedestrian crossing a street in the model, following parameters (rear gap of 0.5, front gap of 0.5, and a safety distance factor of 1.5) used in previous research (Bonisch, et al., 2009).

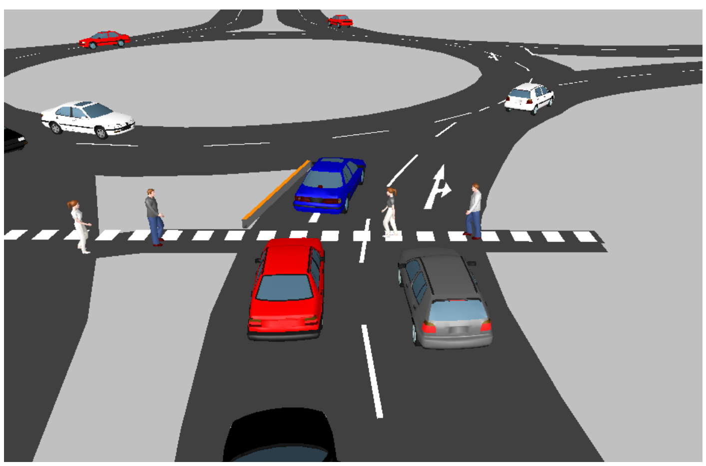


Figure 6.6: Roundabout simulation with pedestrians.

Each of the five pedestrian volumes was run in combination with the four pedestrian crosswalk locations using 10 different random number seeds. The random number seeds provided a stochastic variation of the vehicle arrival times in the model(PTV, 2009). The conflicting volumes was initially setup at 200 pc/h and run up to 1600 pc/h at increment of 100

pc/h, with a total of 15 different conflicting volumes. A total of 3,000 simulations were run during the pedestrian simulations.

6.4 Capacity Estimation

In order to estimate the roundabout's entry capacity from the VISSIM model, four sets of data collection points were located at the entry lanes and circulating lanes, as observed in Figure 6.7. The data collection points 7 and 8 (as example in Figure 6.7) measured the circulating volumes while the data collection points 9 and 10 measured the entering volumes.

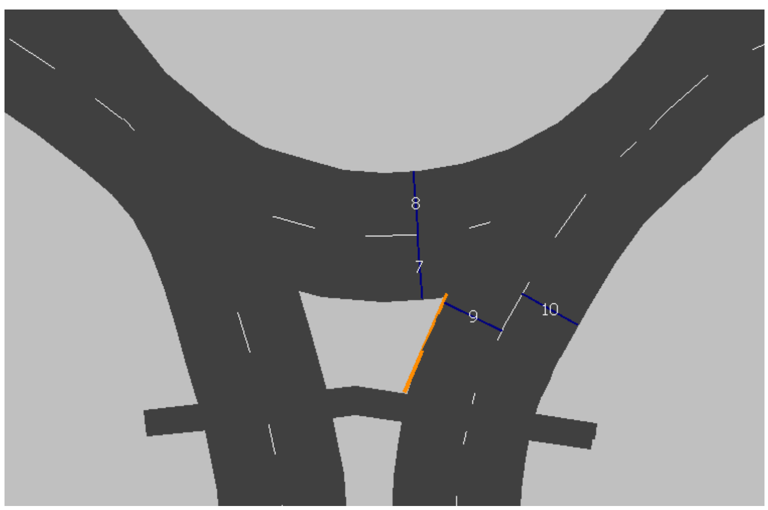


Figure 6.7: Data collection points.

The input volumes at the entry lanes were set up at the maximum capacity of 1130 veh/h per lane. The lane selection was coded to use 51% of the traffic to use the right lane and 49% to use the left lane accordingly to the average values reported in Appendix C in NCHRP Report 572(NCHRP , 2003).

Chapter 7: Simulation Results

This chapter presents the results obtained from the simulation experiments. The results are analyzed in terms of the effects of (i) pedestrian volume; and (ii) the crosswalk position on the capacity of roundabout entrance. The simulation outcomes were analyzed using the combined entry capacity from the right and left lanes of an approach. Figure 7.1 shows a plot of the capacity of the right and left lanes, calculated according to [1.4] and [1.5] as described in Chapter 1, and the combined capacity of both lanes (the approach capacity).

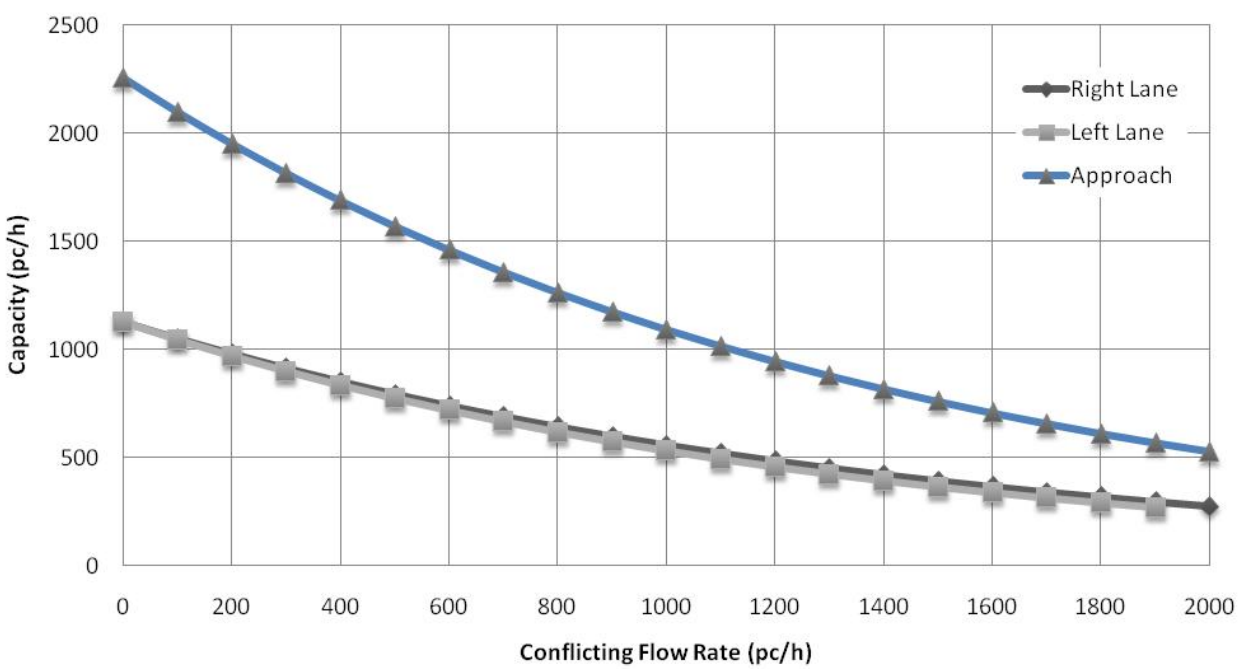


Figure 7.1: Multilane capacity of right lane, left lane and approach.

Figure 7.2 displays the approach capacity based on the HCM2010 procedure and VISSIM simulation. The VISSIM simulations were repeated for 10 random seeds. Both approaches produced very similar results.

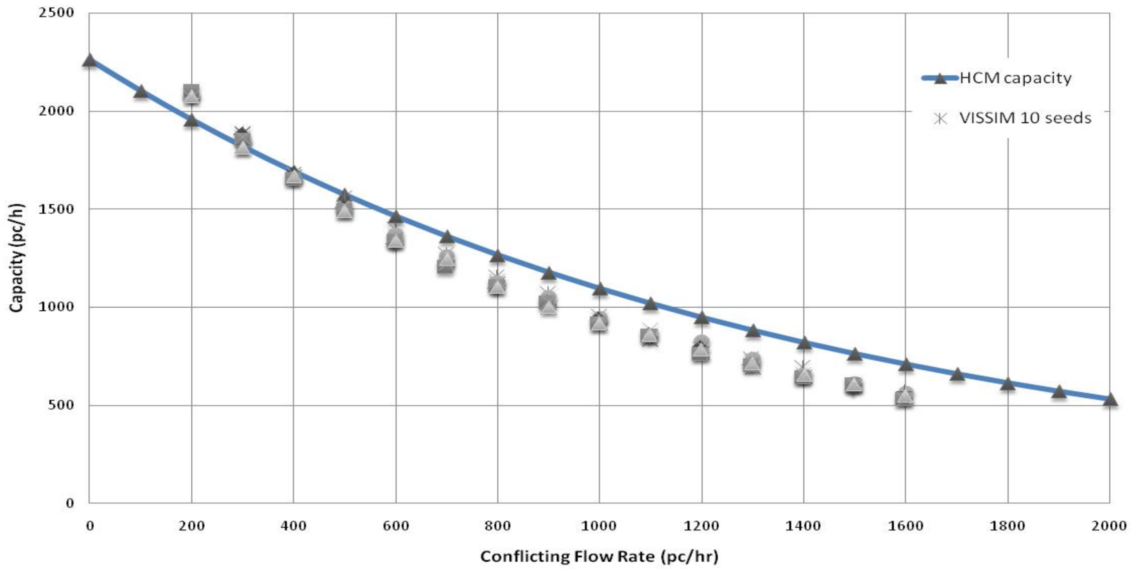


Figure 7.2: HCM capacity and VISSIM capacity models.

7.1 Capacity Reduction Per Crosswalk Location

The first crosswalk position was located at 20 ft upstream from the yield line. Figure 7.3 shows the curves of approach capacity versus conflicting volume. Each curve corresponds to a fixed pedestrian volume.

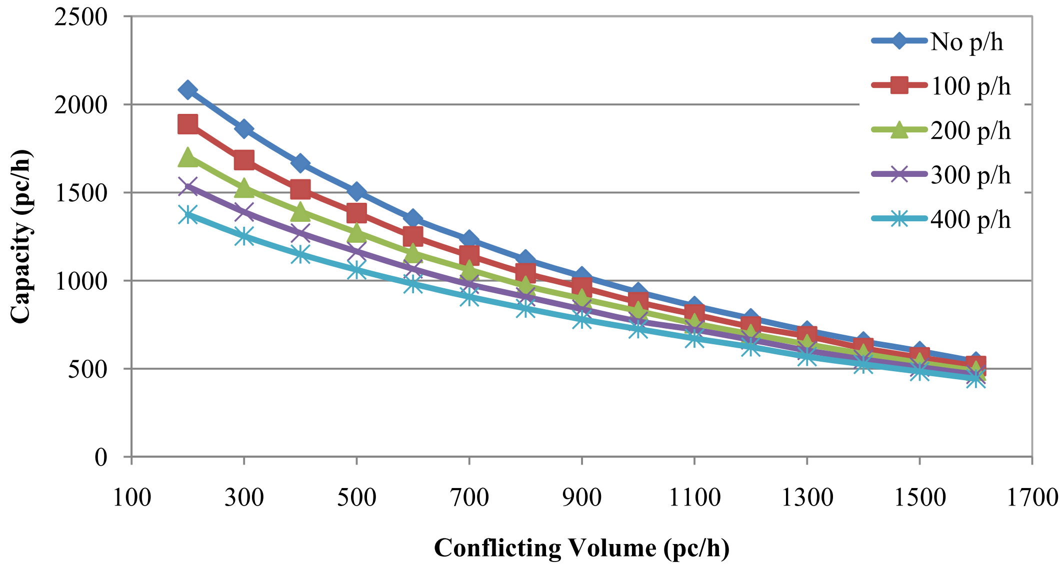


Figure 7.3: Approach capacity for crosswalk located at 20 ft upstream from the yield line.

Figure 7.4 illustrates the approach capacity when the crosswalk is located at 40 ft upstream from the yield line

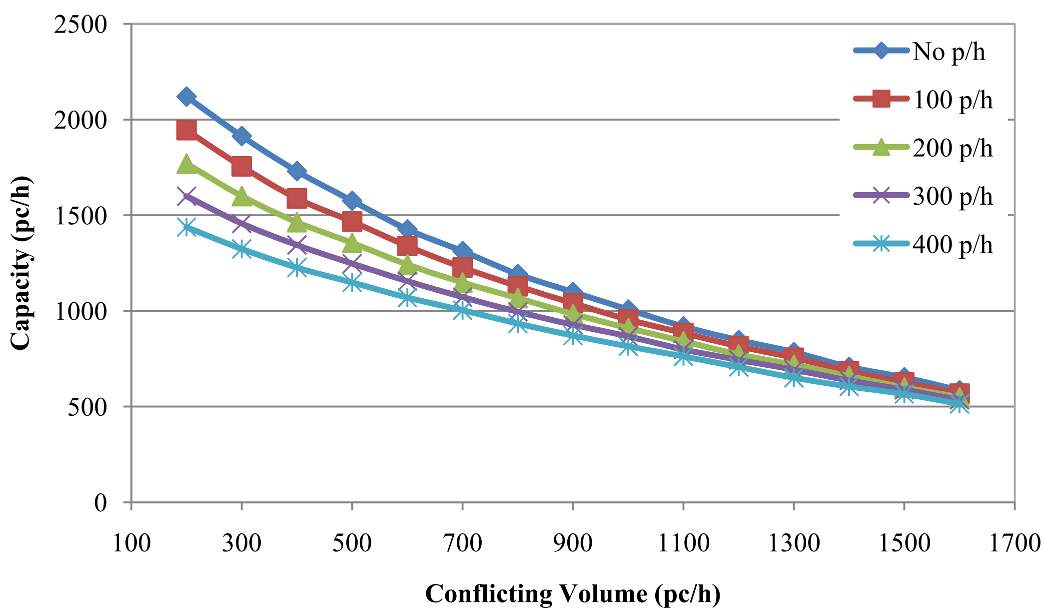


Figure 7.4: Approach capacity for crosswalk located at 40 ft upstream from the yield line.

Figure 7.5 shows the approach capacity when the crosswalk is located at 60 ft upstream from the yield line.

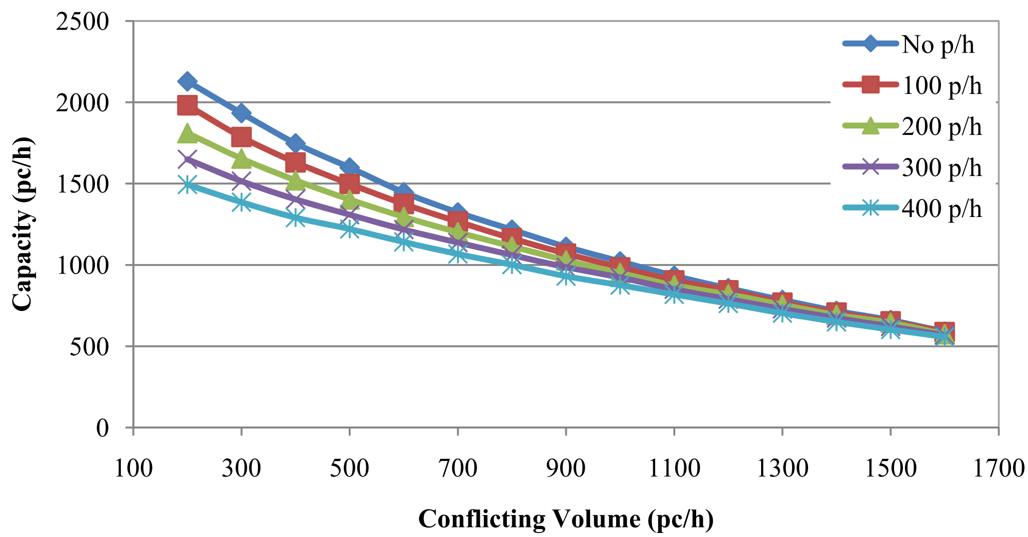


Figure 7.5: Approach capacity for crosswalk located at 60 ft upstream from the yield line.

Figure 7.6 shows the approach capacity when the pedestrian crosswalk is located 80 ft upstream from the yield line.

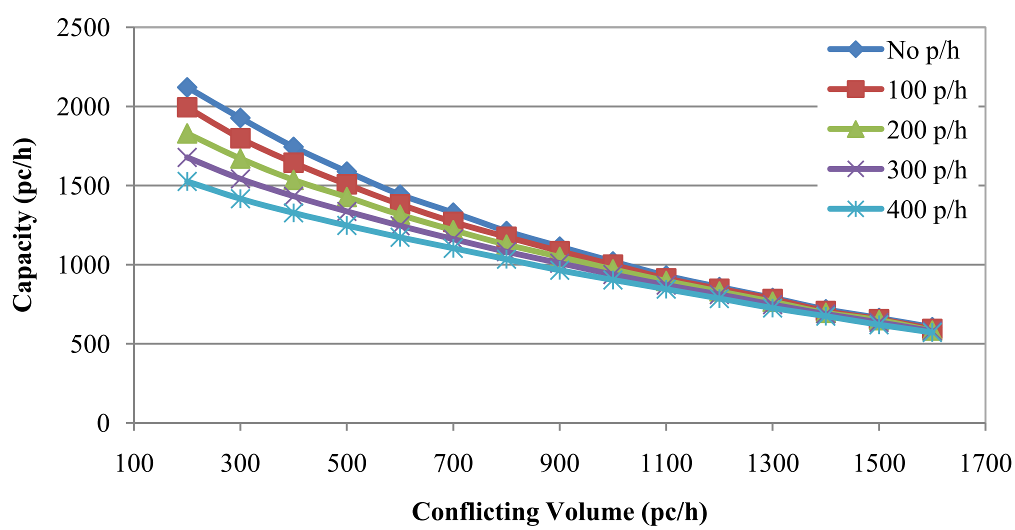


Figure 7.6: Approach capacity for crosswalk located at 80 ft upstream from the yield line.

Comparing the above four figures, it is observed that when the crosswalk position is further away from the yield line, the capacity curves for different pedestrian volumes converge faster with increasing conflicting volume, i.e., the curves are closer to each other.

While the results show a consistent reduction trends for all four crosswalk locations, the differences between the crosswalk locations are observed in the range of capacity reduction. For example, when the pedestrian crosswalk is located at 20 ft upstream from the yield line, the approach volume has an average reduction of 180 pc/h for every 100 pc/h at a conflicting volume of 200 pc/h. While an average reduction of 169 pc/h, 158 pc/h, and 148 pc/h were observed for a pedestrian crosswalk located at 40 ft, 60 ft, and 80 ft respectively.

7.2 Pedestrian Capacity Adjustment Per Pedestrian Volume

The data obtained was also organized to show the effect of crosswalk position on approach capacity. The following figures plots the approach capacity versus conflicting volume, with each curve joining the data points for each crosswalk position. Each figure is produced with the data points obtained with the same pedestrian volume.

Figure 7.7 represent the scenario when there is no pedestrian. On would first expect that without any pedestrian, the entry capacity curves are the same irrespective of the crosswalk position. However, in Figure 6.5, the curve for crosswalk position at 20 ft upstream from the yield line is lower than those obtained with three positions.

After reviewing the simulations it was found that the presence of pedestrian crosswalk had an effect on the vehicle queue at the approach. In VISSIM and day-to-day driving, vehicles are not allowed to stop on top of a pedestrian crosswalk while queuing to provide the right of way to the pedestrians. This restriction has an noticeable impact on crosswalk 20 ft upstream from the yield line. As a control experiment, the VISSIM model was run without a pedestrian crosswalk. The entry capacities obtained with this set of control runs were used as the benchmark to develop pedestrian capacity adjustment factors (described later in this chapter).

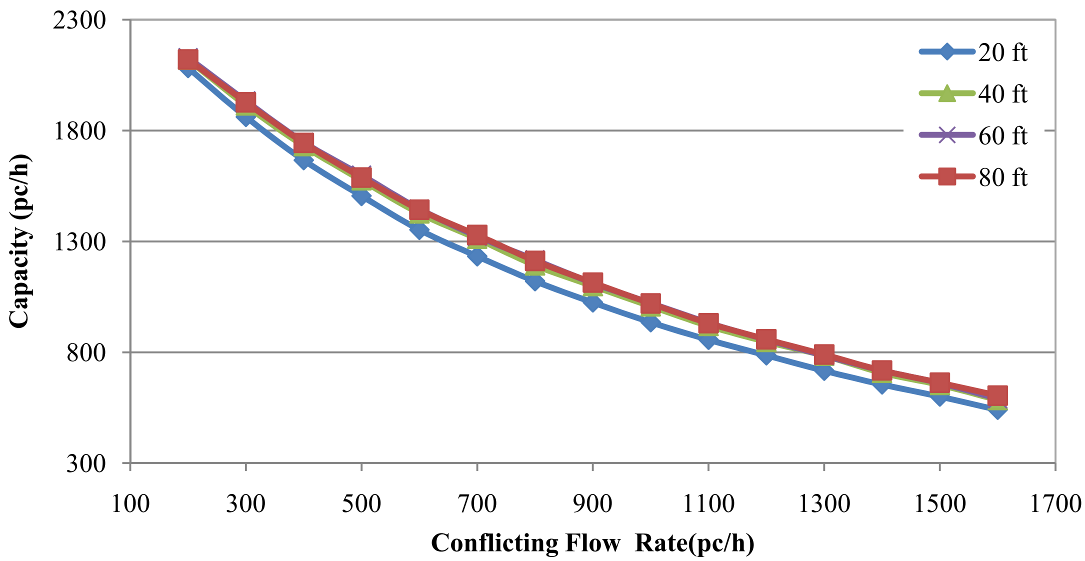


Figure 7.7: Approach capacity without pedestrians.

Figure 7.8 illustrates the approach capacity with 100 p/h crossing the entry approach at each of the crosswalk locations.

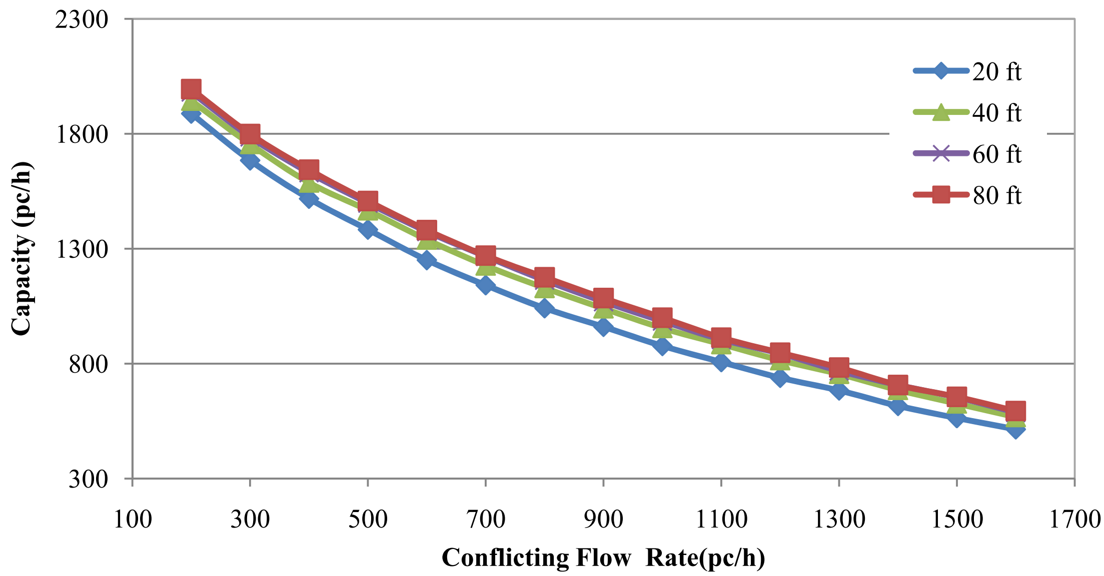


Figure 7.8 Approach capacity with 100 p/h

Figure 7.9 illustrates the approach capacity with 200 p/h crossing the entry approach at each of the crosswalk locations.

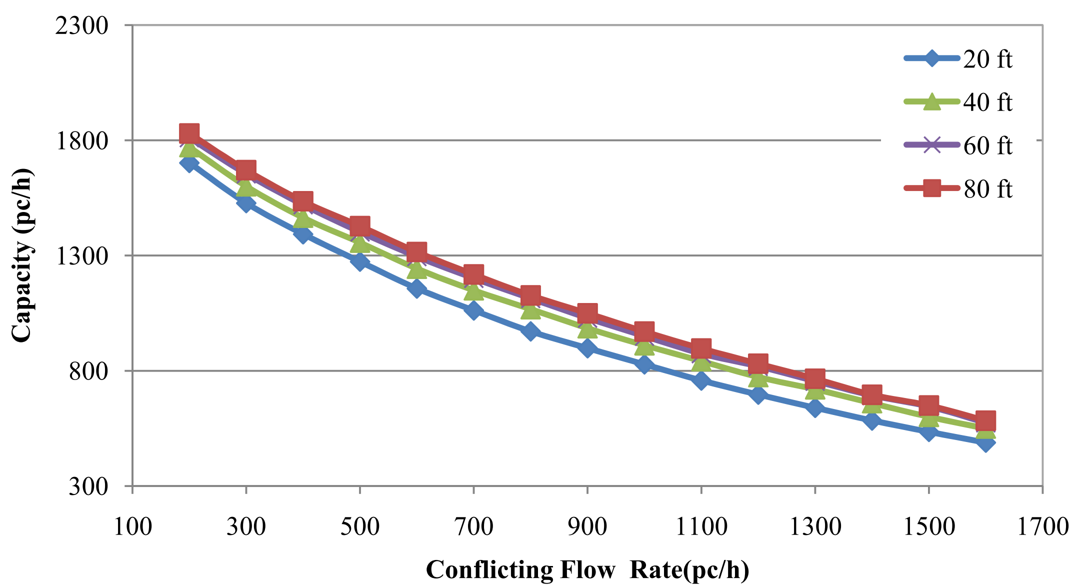


Figure 7.9: Approach capacity with 200 p/h.

Figure 7.10 illustrates the approach capacity with 300 p/h crossing the entry approach.

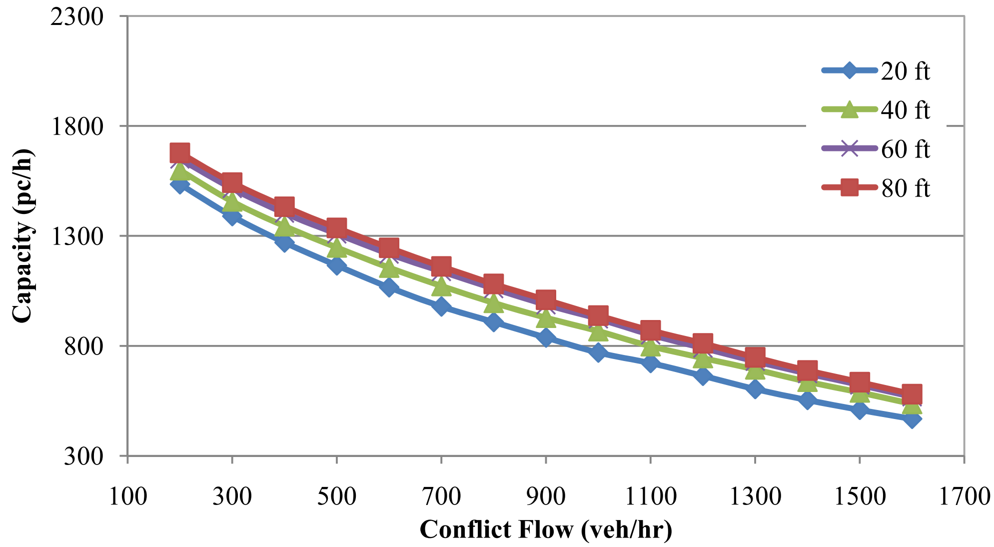


Figure 7.10 Approach capacity with 300 p/h.

Figure 7.10: illustrates the approach capacity with 300 p/h crossing the entry approach.

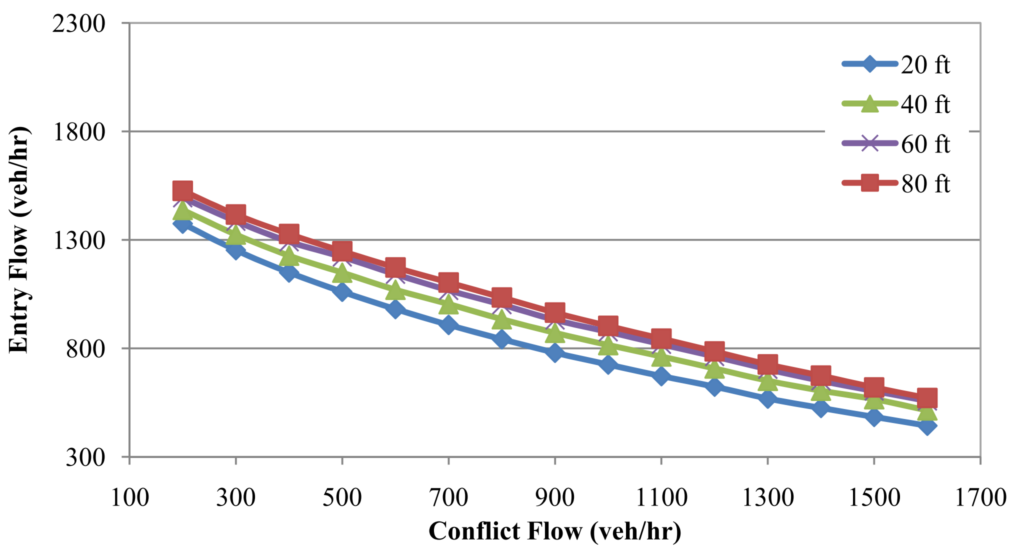


Figure 7.11: Approach capacity with 400 p/h.

7.3 Pedestrian Capacity Adjustment Factors

The pedestrian capacity adjustment factors were estimated using the benchmark data of no crosswalk no pedestrians. The adjustment factors for different pedestrian volumes are plotted in the following figure. Figure 7.14 shows the reduction factor models for pedestrian crosswalk located at 80 ft.

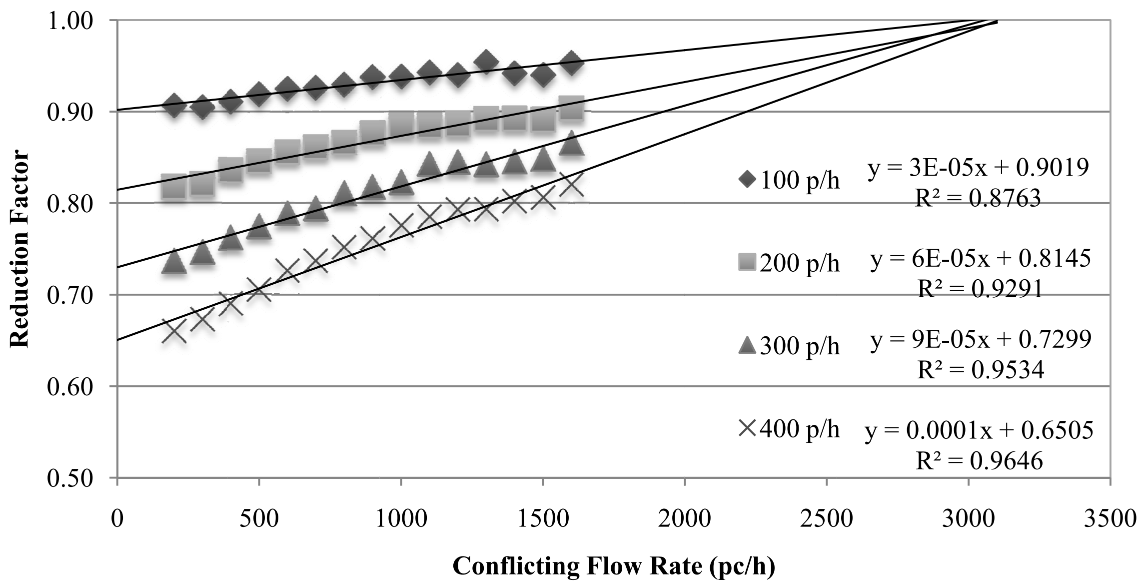


Figure 7.12: Capacity adjustment curves for pedestrian crosswalk at 20 ft upstream from the yield line.

Figure 7.13 shows the reduction factor models for pedestrian crosswalk located at 40 ft upstream from the yield line.

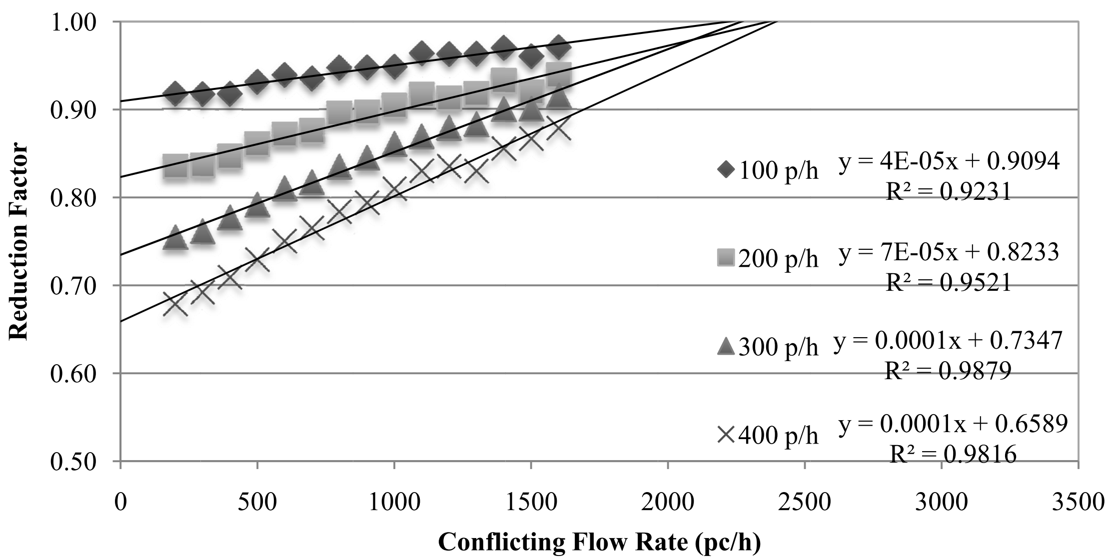


Figure 7.13: Capacity adjustment curves for pedestrian crosswalk at 40 ft upstream from the yield line.

Figure 7.14 shows the reduction factor models for pedestrian crosswalk located at 60 ft upstream from the yield line.

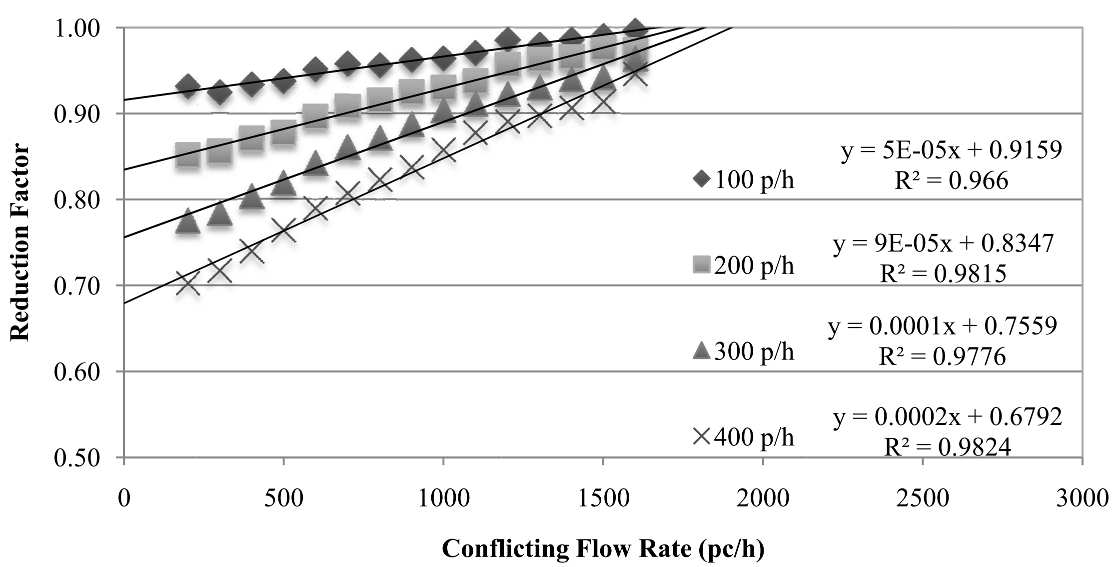


Figure 7.14: Capacity adjustment curves for pedestrian crosswalk at 60 ft upstream from the yield line.

Figure 7.15 shows the reduction factor models for pedestrian crosswalk located at 80 ft upstream from the yield line.

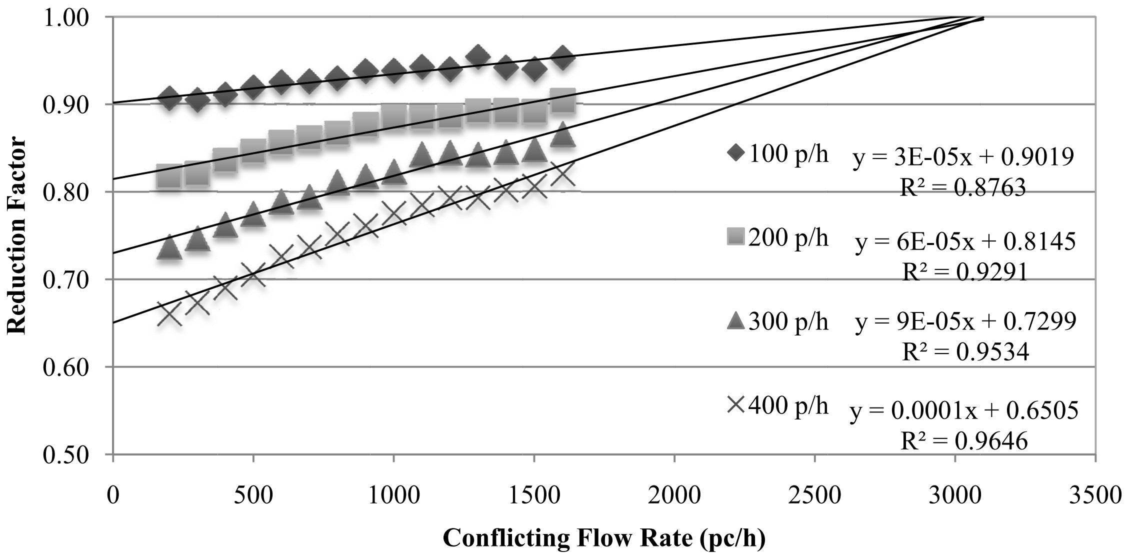


Figure 7.15: Capacity adjustment curves for pedestrian crosswalk at 80 ft upstream from the yield line.

Table 7-1 and tables in Appendix C show an easy way to read the reduction factors. The values provided in this table are based on the VISSIM outputs. For example for a conflicting volume of 900 pc/h with 100 p/h at the entry lanes, the capacity will be reduced by 4% or the approach will operate at 96% of its original capacity.

Table 7-1: Capacity adjustment factor for pedestrian crosswalk located at 20 ft upstream from the yield line, VISSIM results.

Conflicting Volume (pc/h)	Pedestrian Volume 100 p/h	200 p/h	300 p/h	400 p/h
200	0.91	0.82	0.74	0.66
300	0.90	0.82	0.75	0.67
400	0.91	0.84	0.76	0.69
500	0.92	0.85	0.77	0.70
600	0.93	0.86	0.79	0.73
700	0.93	0.86	0.79	0.74
800	0.93	0.87	0.81	0.75
900	0.94	0.88	0.82	0.76
1000	0.94	0.88	0.82	0.78
1100	0.94	0.88	0.84	0.78
1200	0.94	0.89	0.84	0.79
1300	0.95	0.89	0.84	0.79
1400	0.94	0.89	0.85	0.80
1500	0.94	0.89	0.85	0.81
1600	0.95	0.90	0.87	0.82

Table 7-2: Capacity adjustment factor for pedestrian crosswalk located at 20 ft upstream from the yield line, regression model.

Conflicting Volume (pc/h)	Pedestrian Volume 100 p/h	200 p/h	300 p/h	400 p/h
200	0.91	0.83	0.75	0.67
300	0.91	0.83	0.76	0.68
400	0.91	0.84	0.77	0.69
500	0.92	0.84	0.77	0.70
600	0.92	0.85	0.78	0.71
700	0.92	0.86	0.79	0.72
800	0.93	0.86	0.80	0.73
900	0.93	0.87	0.81	0.74
1000	0.93	0.87	0.82	0.75
1100	0.93	0.88	0.83	0.76
1200	0.94	0.89	0.84	0.77
1300	0.94	0.89	0.85	0.78
1400	0.94	0.90	0.86	0.79
1500	0.95	0.90	0.86	0.80
1600	0.95	0.91	0.87	0.81

Chapter 8: Summary, Conclusions and Recommendations

8.1 Summary

The effects of pedestrian crossing on roundabout capacity were evaluated using microscopic traffic simulation models. The goal of this research is to analyze the approach capacity of a two-lane roundabout with (i) four pedestrian volumes; and (ii) four different pedestrian crosswalk locations. The pedestrian capacity adjustment factors developed reflect the capacity reduction associate with pedestrian volume, crosswalk location and conflicting volume. The microscopic traffic simulation model for a roundabout using VISSIM, which has two-lane circulating lanes and a two entry lane was developed. The roundabout simulation model was calibrated and validated with two different set of data (morning and afternoon peak hours observed at the site at Olathe, Kansas). This allowed the simulation model to estimate the entry capacity comparable to the entry capacity estimated when using HCM2010 models without pedestrians.

The pedestrian capacity adjustment factors developed in this research allow for researchers and practitioner to better estimate the roundabout capacity at entry lanes. The following section concludes the important findings from the simulation results.

8.2 Conclusions

The main conclusions of this research are:

- The capacity at each of the scenarios is affected by the number of pedestrian crossing. At higher pedestrian volume, the entry capacity is decreased by a greater factor.
- The models follow a similar trend and show that pedestrian reduce their impact at high volumes of circulating flow rates as estimated in the HCM2010 models.
- The reduction factors estimated with the simulation model are greater than the ones described in the HCM2010. It is important to note that the factors described in HCM2010 are based on research conducted in Germany. The data collected to feed these models could reflect the driving behavior, and familiarity of drivers when using a roundabout with pedestrians.
- The simulation models also demonstrate a lower capacity reduction when the pedestrian crosswalk is located farther away from the yield line.

8.3 Recommendations for future research

The model limitation is affected by the availability of data and observation. The following recommendations are made for future research:

- Collect pedestrian crossing data to verify model results, and compare capacity reduction associate with pedestrians
- More data should be collected in the U.S. when more roundabouts are available. The available data collected under NCHRP project 3-65 is limited for researchers to draw conclusions on the effect of pedestrians on roundabout capacity.

- The integration of blind or handicapped pedestrian should be considered when estimated the capacity reduction associate with these users.
- .Future research should survey if pedestrian are willing to walk more distance for crosswalk located at farther distance away from the standard location 20 ft upstream from the yield line.

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Appendix A: Calibration and Validation

Afternoon peak hour data (calibration)

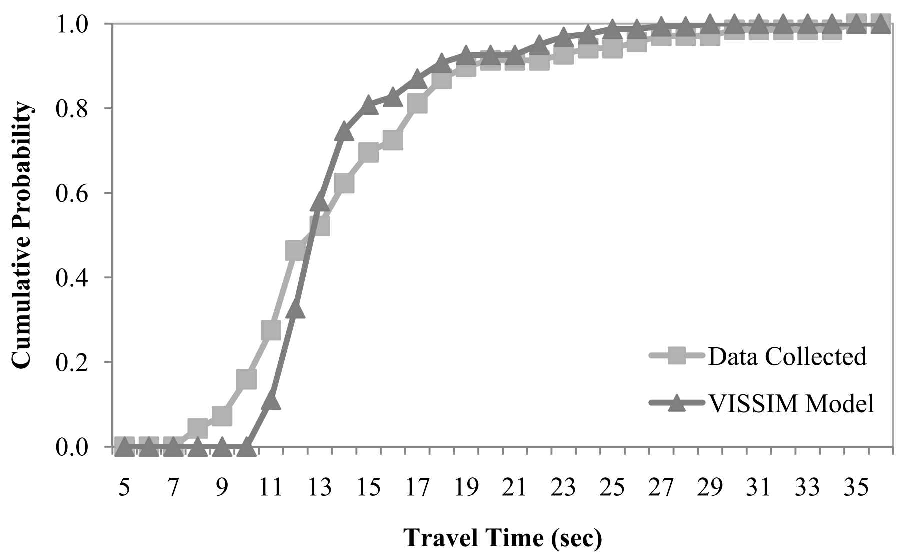

Figure A- 1: K-S test southbound, afternoon peak.

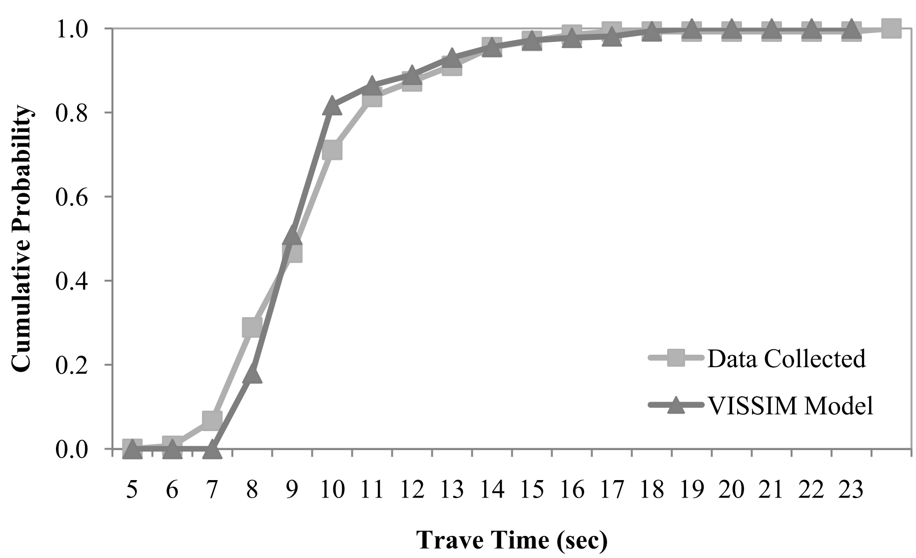

Figure A- 2: K-S test eastbound, afternoon peak.

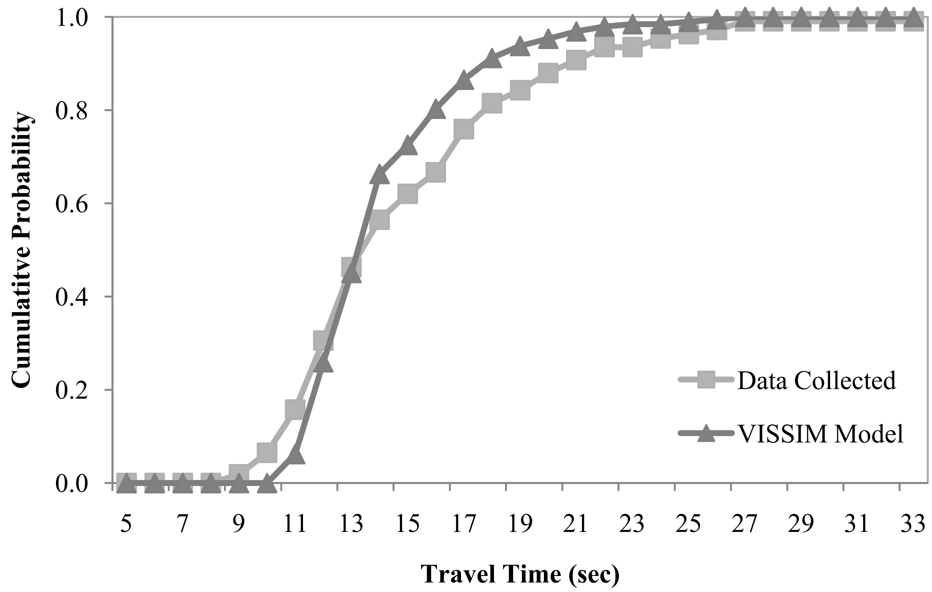

Figure A- 3: K-S test westbound, afternoon peak.

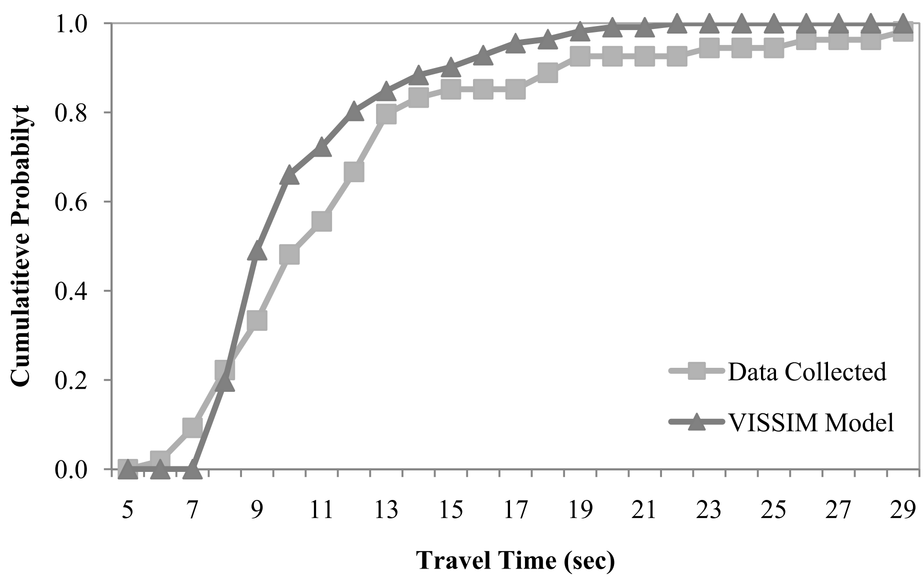

Figure A- 4: K-S test northbound, afternoon peak.

Morning peak hour data (validation)

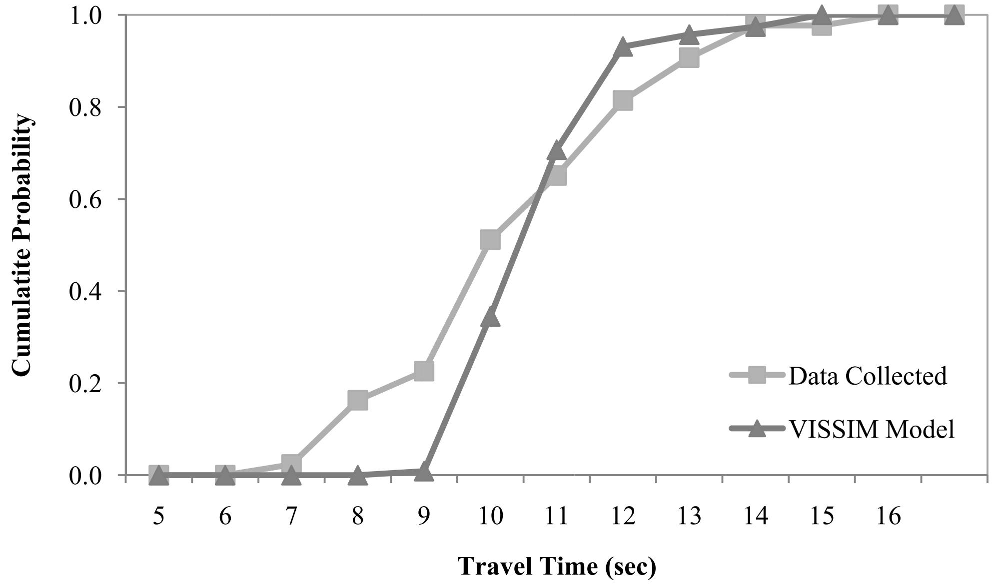

Figure A- 5: K-S test southbound, morning peak.

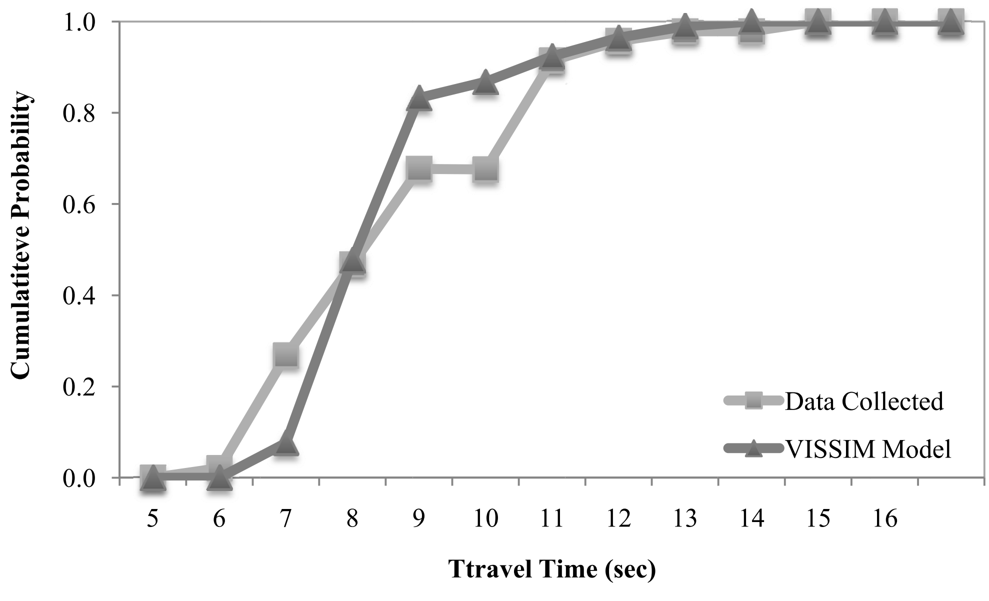

Figure A- 6: K-S test eastbound, morning peak.

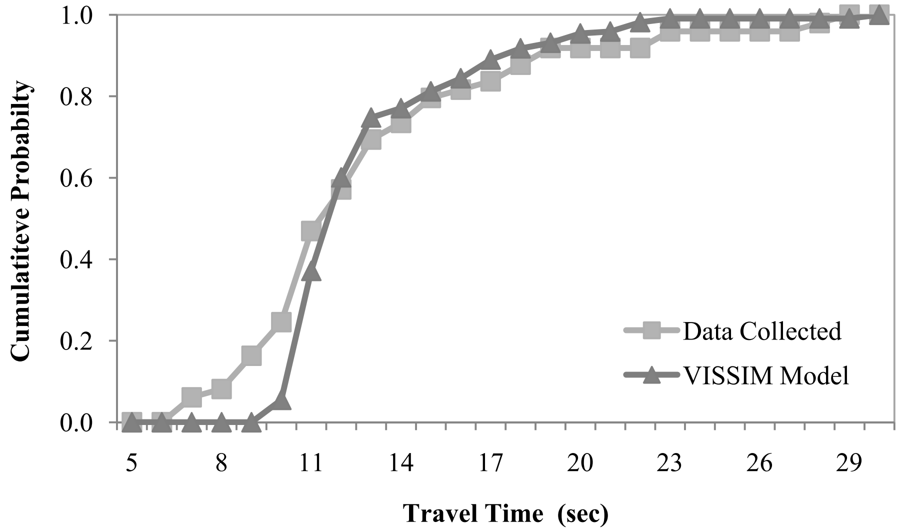

Figure A- 7: K-S test westbound, morning peak.

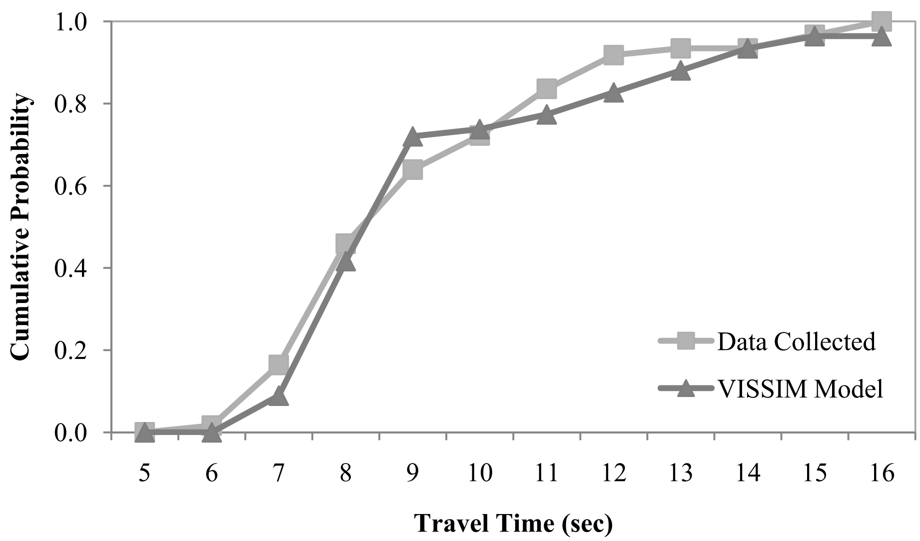

Figure A- 8: K-S test northbound, morning peak.

Appendix B: Reduction Factors

Table B- 1: Capacity adjustment factor for pedestrian crosswalk located at 20 ft upstream from the yield line.

Conflicting Volume (pc/h)	Pedestrian Volume 100 p/h	200 p/h	300 p/h	400 p/h
200	0.91	0.82	0.74	0.66
300	0.90	0.82	0.75	0.67
400	0.91	0.84	0.76	0.69
500	0.92	0.85	0.77	0.70
600	0.93	0.86	0.79	0.73
700	0.93	0.86	0.79	0.74
800	0.93	0.87	0.81	0.75
900	0.94	0.88	0.82	0.76
1000	0.94	0.88	0.82	0.78
1100	0.94	0.88	0.84	0.78
1200	0.94	0.89	0.84	0.79
1300	0.95	0.89	0.84	0.79
1400	0.94	0.89	0.85	0.80
1500	0.94	0.89	0.85	0.81
1600	0.95	0.90	0.87	0.82

Table B- 2: Capacity adjustment factor for pedestrian crosswalk located at 40 ft upstream from the yield line.

Conflicting Volume (pc/h)	Pedestrian Volume 100 p/h	200 p/h	300 p/h	400 p/h
200	0.92	0.84	0.75	0.68
300	0.92	0.84	0.76	0.69
400	0.92	0.85	0.78	0.71
500	0.93	0.86	0.79	0.73
600	0.94	0.87	0.81	0.75
700	0.94	0.88	0.82	0.77
800	0.95	0.90	0.84	0.78
900	0.95	0.90	0.84	0.79
1000	0.95	0.90	0.86	0.81
1100	0.96	0.92	0.87	0.83
1200	0.96	0.91	0.88	0.83
1300	0.96	0.92	0.88	0.83
1400	0.97	0.93	0.90	0.86
1500	0.96	0.92	0.90	0.87
1600	0.97	0.94	0.92	0.88

Table B- 3: Capacity adjustment factor for pedestrian crosswalk located at 60 ft upstream from the yield line.

Conflicting Volume (pc/h)	Pedestrian Volume			
	100 p/h	100 p/h	100 p/h	100 p/h
200	0.93	0.85	0.78	0.70
300	0.92	0.86	0.78	0.72
400	0.93	0.87	0.80	0.74
500	0.94	0.88	0.82	0.76
600	0.95	0.90	0.84	0.79
700	0.96	0.91	0.86	0.81
800	0.96	0.92	0.87	0.82
900	0.96	0.92	0.89	0.84
1000	0.96	0.93	0.90	0.86
1100	0.97	0.94	0.91	0.88
1200	0.99	0.96	0.92	0.89
1300	0.98	0.96	0.93	0.90
1400	0.99	0.97	0.94	0.91
1500	0.99	0.98	0.94	0.91
1600	1.00	0.97	0.96	0.95

Table B- 4: Capacity adjustment factor for pedestrian crosswalk located at 80 ft upstream from the yield line.

Conflicting Volume (pc/h)	Pedestrian Volume			
	100 p/h	200 p/h	300 p/h	400 p/h
200	0.94	0.86	0.79	0.72
300	0.93	0.87	0.80	0.74
400	0.94	0.88	0.82	0.76
500	0.95	0.90	0.84	0.79
600	0.96	0.91	0.86	0.81
700	0.96	0.92	0.87	0.83
800	0.97	0.93	0.89	0.85
900	0.97	0.94	0.91	0.87
1000	0.98	0.95	0.92	0.89
1100	0.98	0.96	0.94	0.91
1200	0.99	0.97	0.95	0.92
1300	0.99	0.97	0.95	0.92
1400	0.99	0.97	0.96	0.94
1500	0.99	0.98	0.96	0.94
1600	0.98	0.97	0.96	0.95

Appendix C: Capacity Adjustment Factors from Regression Model

Table C- 1: Capacity adjustment factor for pedestrian crosswalk located at 20 ft upstream from the yield line.

Conflicting Volume (pc/h)	Pedestrian Volume			
	100 p/h	200 p/h	300 p/h	400 p/h
200	0.91	0.83	0.75	0.67
300	0.91	0.83	0.76	0.68
400	0.91	0.84	0.77	0.69
500	0.92	0.84	0.77	0.70
600	0.92	0.85	0.78	0.71
700	0.92	0.86	0.79	0.72
800	0.93	0.86	0.80	0.73
900	0.93	0.87	0.81	0.74
1000	0.93	0.87	0.82	0.75
1100	0.93	0.88	0.83	0.76
1200	0.94	0.89	0.84	0.77
1300	0.94	0.89	0.85	0.78
1400	0.94	0.90	0.86	0.79
1500	0.95	0.90	0.86	0.80
1600	0.95	0.91	0.87	0.81

Table C- 2: Capacity adjustment factor for pedestrian crosswalk located at 40 ft upstream from the yield line.

Conflicting Volume (pc/h)	Pedestrian Volume			
	100 p/h	200 p/h	300 p/h	400 p/h
200	0.92	0.84	0.75	0.68
300	0.92	0.84	0.76	0.69
400	0.93	0.85	0.77	0.70
500	0.93	0.86	0.78	0.71
600	0.93	0.87	0.79	0.72
700	0.94	0.87	0.80	0.73
800	0.94	0.88	0.81	0.74
900	0.95	0.89	0.82	0.75
1000	0.95	0.89	0.83	0.76
1100	0.95	0.90	0.84	0.77
1200	0.96	0.91	0.85	0.78
1300	0.96	0.91	0.86	0.79
1400	0.97	0.92	0.87	0.80
1500	0.97	0.93	0.88	0.81
1600	0.97	0.94	0.89	0.82

Table C- 3: Capacity adjustment factor for pedestrian crosswalk located at 60 ft upstream from the yield line.

Conflicting Volume (pc/h)	Pedestrian Volume			
	100 p/h	200 p/h	300 p/h	400 p/h
200	0.93	0.85	0.78	0.72
300	0.93	0.86	0.79	0.74
400	0.94	0.87	0.80	0.76
500	0.94	0.88	0.81	0.78
600	0.95	0.89	0.82	0.80
700	0.95	0.90	0.83	0.82
800	0.96	0.91	0.84	0.84
900	0.96	0.92	0.85	0.86
1000	0.97	0.92	0.86	0.88
1100	0.97	0.93	0.87	0.90
1200	0.98	0.94	0.88	0.92
1300	0.98	0.95	0.89	0.94
1400	0.99	0.96	0.90	0.96
1500	0.99	0.97	0.91	0.98
1600	1.00	0.98	0.92	1.00

Table C- 4: Capacity adjustment factor for pedestrian crosswalk located at 80 ft upstream from the yield line.

Conflicting Volume (pc/h)	Pedestrian Volume			
	100 p/h	200 p/h	300 p/h	400 p/h
200	0.94	0.87	0.80	0.74
300	0.94	0.88	0.81	0.76
400	0.95	0.89	0.82	0.78
500	0.95	0.90	0.83	0.80
600	0.95	0.91	0.84	0.82
700	0.96	0.92	0.85	0.84
800	0.96	0.93	0.86	0.86
900	0.97	0.94	0.87	0.88
1000	0.97	0.94	0.88	0.90
1100	0.97	0.95	0.89	0.92
1200	0.98	0.96	0.90	0.94
1300	0.98	0.97	0.91	0.96
1400	0.99	0.98	0.92	0.98
1500	0.99	0.99	0.93	1.00
1600	0.99	1.00	0.94	1.02

Vita

Carlos Duran was born in Torreón, Coahuila, México the third son from Maria de Lourdes Laura Valderrama Tapia and Miguel Angel Duran Méndez. At the age of ten year old, his family moved to Chihuahua, Chihuahua, Mexico. After completing his high school education, he moved to El Paso, Texas, where he started his higher education at El Paso Community College (EPCC), and later at the University of Texas at El Paso (UTEP). Mr Duran completed his Associate of Arts at EPCC in 2005, Bachelor's Degree in Civil Engineering in 2007, and accomplished a Master Degree in Civil Engineering in 2010. While pursuing his bachelor's and master's degree, Mr Duran worked as a Research Assistant and Graduate Research Assistant to various professors and at different transportation centers, such as the Laboratory for Advance Dynamic Transportation and Urban System (LADTUS at UTEP) under the supervision of Dr Yi Chiu, Border Intermodal Gateway Transportation Laboratory (UTEP) under the supervision of Dr Kelvin Cheu, and recently at The Center for International Intelligent Transportation Research from the Texas Transportation Institute under the supervision of Dr Rafel Aldrete. Additionally Mr Duran has taken part of different organization as president of the Institute of Transportation Engineers (ITE), vice-present of Chi Epsilon Civil Engineering Honor Society, vice-president of the Civil Engineering Senior Class Organization and member of the American Society of Civil Engineering (ASCE). During his studies Mr Duran has been awarded with the UTEP Cd Juarez Alumni Chapter Scholarship, two times recipient of the Dwight D. Eisenhower Transportation Fellowship, and selected as the UTEP representative for the International Road Federation Fellowship in 2010. Recently Mr Duran lead a joint team formed by students from the University of Texas at El Paso and a Chinese research center, and obtain the

bronze medal at the Daimler-UNESCO Mondialogo Engineering Awards 2010 in Stuttgart, Germany.

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