FINAL REPORT

to

THE FLORIDA DEPARTMENT OF TRANSPORTATION
SYSTEMS PLANNING OFFICE

on Project

"Impact of Trucks on Arterial LOS and Freeway Work Zone Capacity"

FDOT Contract BD-545, RPWO #51 (UF Project 00054954)

Part A: Impact of Trucks on Arterial LOS

July 2007

University of Florida
Transportation Research Center
Department of Civil and Coastal Engineering

Disclaimer

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data published herein. The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Impact of Trucks on Arterial LOS and Freeway Work Zone Capacity (Part A)		5. Report Date July 2007
		6. Performing Organization Code UF-TRC
7. Author(s) Scott S. Washburn and Carlos O. Cruz-Casas		8. Performing Organization Report No. TRC-FDOT-54954-a-2007
9. Performing Organization Name and Address Transportation Research Center University of Florida 512 Weil Hall / P.O. Box 116580 Gainesville, FL 32611-6580		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. FDOT Contract BD-545 RPWO #51
12. Sponsoring Agency Name and Address Florida Department of Transportation 605 Suwannee St. MS 30 Tallahassee, Florida 32399 (850) 414 - 4615		13. Type of Report and Period Covered Final Report
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

Large trucks have considerably different size and performance characteristics than passenger cars. Consequently, these trucks can have a significant impact on traffic operations. It is therefore essential to properly account for this impact in the traffic operations analysis in order to reflect the operational quality of the roadway as accurately as possible. Signalized intersections are one roadway facility that can be particularly sensitive to the presence of commercial truck traffic.

The most common method used for the analysis of signalized intersections is contained in the Highway Capacity Manual (HCM). In this method, the base saturation flow rate of the signalized intersection is defined in units of passenger cars per hour green per lane (pc/hg/ln). To account for the presence of large trucks in the traffic stream, the HCM includes a Passenger Car Equivalency (PCE) value. In the current edition of the HCM, a PCE value of 2.0 is applied for all large trucks, with no distinction between different sizes of trucks.

Some transportation professionals have questioned the validity of this PCE value recommended by the HCM. They are concerned that the impact of trucks at signalized intersections is being under-estimated. If this is the case, then capacity is being over-estimated and intersections are not being adequately designed.

The primary objective of this research was to determine appropriate truck PCE values to apply for signalized intersection analysis. These PCE values were classified by three different categories of truck sizes and performances. Additionally, a general PCE value with only one truck category was developed for planning purposes and/or a less detailed analysis. The development of the PCE values was based on the relative headway concept, as defined in the HCM. The results of this study are based primarily on data generated from a custom simulation program. However, a considerable amount of field data was collected for the purpose of simulation calibration. The PCE values determined from this study are 1.8, 2.2, and 2.8 for small trucks, medium trucks, and large trucks, respectively. Additionally, an equation was developed to calculate start-up lost time that accounts for the impact of trucks at the front of the queue, as opposed to the standard 2.0 seconds recommended by the HCM. Furthermore, based on the field data collected, it was found that the base saturation flow rate value of 1900 pc/hg/ln recommended by the HCM appears to be quite optimistic.

17. Key Words Signalized Intersections, Trucks, Passenger Car Equivalent Values, Saturation Flow Rate	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA, 22161

19 Security Classif. (of this report) Unclassified	20.Security Classif. (of this page) Unclassified	21.No. of Pages 107	22 Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

Acknowledgments

The authors would like to express their sincere appreciation to Ms. Gina Bonyani and Mr. Doug McLeod of the Florida Department of Transportation (Central Office) for the support and guidance they provided on this project. We would also like to thank the staff of the Gainesville Public Works department and the staff of the Brooksville office of FDOT for their assistance with field data collection.

Report Organization

The content for this report is essentially the master's thesis prepared by Mr. Carlos Cruz-Casas under the supervision of Dr. Scott Washburn. The front matter that was relevant only to the graduate school of the University of Florida was deleted, and some minor editorial and formatting revisions were also performed.

DEVELOPMENT OF PASSENGER CAR EQUIVALENCY VALUES FOR TRUCKS AT SIGNALIZED INTERSECTIONS

TABLE OF CONTENTS

page

LIST OF TABLES iii

LIST OF FIGURES iv

CHAPTER

1 INTRODUCTION 1

Background 1
Problem Statement 2
Research Objective and Tasks 3
Document Organization 3

2 LITERATURE REVIEW 5

Overview of HCM Treatment of Heavy Vehicles 5
Passenger Car Equivalency Factor at Signalized Intersections 7
Car-Following Models 12
Summary of HCM 2000 PCE Guidelines 20
Summary of Previous PCE Studies 20
Summary of Car-Following Models 21

3 RESEARCH APPROACH 23

Methodological Approach 23
Field Data Collection 26
Simulation Modeling 38

4 ANALYSIS OF FIELD DATA AND CALIBRATION OF SIMULATION MODEL 45

Summary of Data Reduction 45
Time Headways 46
Start-up Reaction Time (SRT) 49
Start-up Lost Time (SLT) 51
Calibration of Simulation Model 52
Execution of Experimental Design 54

5 ANALYSIS OF SIMULATION DATA 56

6 CONCLUSIONS AND RECOMMENDATIONS 73

APPENDIX

A DATA COLLECTION EQUIPMENT SETUP 79

B PICTURES OF VEHICLE TYPES BY CATEGORY 81

C SIMULATION PROGRAM SCREENSHOTS 86

D QUEUE COMPOSITION 87

E HEADWAY STATISTICS 89

F CALIBRATION RESULTS AND MSE CALCULATIONS 91

LIST OF REFERENCES 94

LIST OF TABLES

Table		page
2-1	Car-following models comparison	21
3-1	Possible pairing combinations for four vehicle types*	25
3-2	Data collection cites	27
3-3	Data collection periods for Williston Rd/34th Street site (method 1)	37
3-4	Data collection periods for other sites (method 2)	37
4-1	Average headway and frequencies for each leader-follower combination	46
4-2	Time headways from field data	47
4-3	Results from STATISTICA for model 1	47
4-4	Results from STATISTICA for model 2	48
4-5	Results from STATISTICA for model 3	48
4-6	Results from STATISTICA for model 4	49
4-7	Distribution of trucks by site and position in queue	51
4-8	Final choice of calibration parameter values	54
5-1	Model estimation results for additional headways of 16 vehicle pair combinations	62
5-2	Headways and PCE values for 16 vehicle pair combinations	64
5-3	Vehicle type's headway and their Δh	65
5-4	Time consumed and PCE values for each vehicle type	66
5-5	Results from STATISTICA for model with vehicle types	67
5-6	PCE factors for three vehicle types	68
5-7	Model estimation results for f_{HV} equation with 16 vehicle pairs	70
D-1	Vehicle type per position in queue	87
E-1	Headway statistics for 16 lead-trail vehicle combinations	89
F-1	Simulation model calibration results	91

LIST OF FIGURES

Figure page

2-1 Measured headways including start-up lost time (dark shade) 7

3-1 SW Williston Rd / SW 34th St aerial view (eastbound on Williston Rd) 27

3-2 SW Williston Rd / SW 34th St ground level view (eastbound on Williston Rd) 28

3-3 University Ave. / Waldo Rd aerial view (northbound on Waldo Rd) 28

3-4 University Ave. / Waldo Rd ground level view (northbound on Waldo Rd) 29

3-5 US 41 / SR 50 aerial view (westbound on SR50) 29

3-6 US 41 / SR 50 ground level view (westbound on SR50) 30

3-7 US 301 / SR 50 aerial view (eastbound on SR 50) 30

3-8 US 301 / SR 50 ground level view (eastbound on SR 50) 31

3-9 CR 326 / SR 200A ground level view (westbound on CR 326) 31

3-10 John Young Parkway / Colonial Dr aerial view (southbound on John Young Pkwy) 32

3-11 John Young Pkwy / Colonial Dr ground level view (southbound on John Young Pkwy) 32

3-12 Preferred video camera mounting location (plan view) for method 1 34

3-13 Screen capture (low resolution) of video image from Williston Rd/34th Street site in Gainesville (method 1) 34

3-14 Camera setup for data collection (method 2) 35

3-15 Screen capture of the composite video image (method 2) 36

3-16 Simulation program user interface 41

4-1 Start-up reaction time frequencies 50

4-2 Start-up reaction time frequencies for trimmed data set 50

4-3 User-adjustable vehicle, driver, and model parameters 53

5-1 Queue sections for analysis 56

5-2 Impact of trucks in the first part of the queue to the saturation headway 57

5-3 SLT example 61

5-4 Total time for vehicles 1-8 using vehicle pairs in positions 5-8 63

5-5 Time consumed by a large truck 64

5-6 Total time for vehicles 1-8 using vehicle types in positions 5-8 67

A-1 Data collection equipment setup for method 1 79

A-2 Signal controller cabinet with data collection equipment installed for method 1 80

B-1 Small trucks. A) Panel truck. B) Garbage truck. C) Two-Axle Single-unit dump truck. D) Small delivery truck. E) Passenger cars with trailers 81

B-2 Medium trucks. A) Three-Axle Single-unit dump truck. B) Concrete Mixer. C) Passenger car with trailer using fifth wheel. D) Delivery truck. E) Single-unit cargo truck. 83

B-3 Large trucks. A) Tractor plus trailer. B) Tractor plus flatbed. C) Buses. 84

C-1 Simulation screenshot. A) Before signal turns green. B) Once the queue starts to discharge 86

CHAPTER 1
INTRODUCTION

Background

Large trucks have considerably different size and performance characteristics than passenger cars. Consequently, these trucks can have a significant impact on traffic operations. It is therefore essential to properly account for this impact in the traffic operations analysis in order to reflect the operational quality of the roadway as accurately as possible. This study focuses only on those trucks that are considerably larger than pick-up trucks.

Signalized intersections are one roadway facility that can be particularly sensitive to the presence of commercial truck traffic. Like other facilities, the length of trucks has a negative impact on the capacity of the signalized intersection; however, the reduced performance characteristics of these trucks has an even greater impact on signalized intersection than uninterrupted flow facilities due to the need for many of the trucks traveling through an intersection to decelerate to a stop and re-accelerate to cruise speed. Given the longer time it takes trucks to re-accelerate to cruise speed, when compared to passenger cars, the presence of trucks can have implications on signal coordination. This effect may be greater when trucks are present at the front of a discharging queue.

When the traffic stream stops at the intersection, the inter-vehicle spacing decreases, resulting in increased vehicle density. At this higher density, it is clear that trucks occupy more space than passenger cars due to their physical characteristics. Once the signal turns green, and after a short period of start-up lost time, vehicles start departing at the saturation flow rate and it is here where the heavy vehicles have the greater impact due to their operational capabilities. Large trucks have poorer acceleration than passenger cars; therefore it will take them more time to reach their desired speed. With poorer acceleration characteristics, heavy vehicles slow down the traffic stream and increase their time headway. Furthermore, with their longer length, heavy vehicles increase the time headway of the vehicle following them. In arterial roads where the Free Flow Speed is low, the cruising speed of all vehicles may be similar. Once the vehicles reach their desired speed and have a constant cruising speed, the impact of trucks to the traffic will be again mostly due to their length.

Problem Statement

The most common method used for the analysis of signalized intersections is contained in the Highway Capacity Manual (HCM) [1]. In this method, the base saturation flow rate of the signalized intersection is defined in units of passenger cars per hour green per lane (pc/hg/ln). To account for the presence of large trucks in the traffic stream, the HCM includes a Passenger Car Equivalency (PCE) value. This factor is based on the relative headway of a truck to that of a passenger car. In the current edition of the HCM, a single PCE value of 2.0 is used for all trucks passing through a signalized intersection. Thus, this implies that a single large truck is equivalent to two passenger cars for capacity analysis purposes; that is, an intersection can only accommodate half as many trucks as cars.

Transportation professionals with the Florida Department of Transportation (FDOT) have recently questioned the validity of this PCE value recommended by the HCM. This concern has stemmed from observations that the failure rate of intersections in Florida seems particularly high when any significant percentage of trucks is present in the traffic stream. Their observations are especially disturbing in light of the fact that Florida's population is growing rapidly, and complimentary to this population growth is a corresponding growth in commercial truck traffic. Thus, FDOT officials are concerned that the impacts of trucks at signalized intersections in Florida are being under-estimated. If this is the case, then capacity is being over-estimated and intersections are not being adequately designed.

Research Objective and Tasks

The primary objective of this research was to determine appropriate truck PCE values to apply for signalized intersection analysis. These PCE values were classified by three different categories of truck sizes and performances. Additionally, a general PCE value with only one truck category was developed for planning purposes and/or a less detailed analysis. The results of this study are based primarily on data generated from a custom simulation program. However, a considerable amount of field data was collected for the purpose of simulation calibration.

The following tasks were conducted to support the accomplishment of these objectives:

- Conduct a literature review
- Collect preliminary field data
- Develop data collection methods
- Determine appropriate criteria to select data collection sites
- Identify appropriate field data collection sites
- Collect video data from field
- Process the video using the RLRAP [2] to obtain signal status and time
- Reduce field data
- Develop a simulation program
- Calibrate the simulation program
- Develop an experimental design
- Generate simulation data set from experimental design
- Perform analysis and modeling of the generated simulation data set
- Develop new PCE factors

Document Organization

Chapter 2 presents an overview of the relevant studies found in the literature. This review includes the state of the practice with regard to incorporating the effects of large vehicles into traffic analyses, previous studies focused on the development of PCE values on arterial roads or at intersections, and a review of car-following models that might be applicable to modeling queue discharge at signalized intersections. Chapter 3 describes the research approach that was used to accomplish the objectives of this study including the methodological approach, field data collection, simulation model development, and the simulation experiments. Chapters 4 and 5 contain the analysis and results of the field and simulation data, respectively. Conclusions and recommendations are contained in Chapter 6.

CHAPTER 2
LITERATURE REVIEW

An extensive literature review has been conducted in three areas. The first area is a review of the state of the practice with regard to incorporating the effects of heavy vehicles into traffic analyses, namely the Highway Capacity Manual [1, 3]. The second area targets previous studies that focused on the development of PCE values at intersections. While other studies have been done on the development of PCEs for other facilities, they were not covered in this project due to the unique influence of signals on traffic flow. A custom simulation program was developed for use in this project for several reasons, which are discussed in Chapter 3. Therefore the third area of this chapter deals with a review of car-following models that might be applicable to modeling queue discharge at signalized intersections.

Overview of HCM Treatment of Heavy Vehicles

The Highway Capacity Manual (HCM) first introduced the term "passenger car equivalent" in the 1965 version of this publication [3] as "the number of passenger cars displaced in the traffic flow by a truck or bus, under the prevailing roadway and traffic conditions." The HCM 2000 [1] definition states that the passenger car equivalent is "the number of passenger cars displaced by a single, heavy vehicle of a particular type under specified roadway, traffic, and control conditions." Currently, a PCE value of 2.0 is specified for all heavy vehicles. The refining of the original definition emphasizes the importance of traffic controls and their effect on PCE values, and considers the subjectivity of what can be considered as heavy vehicles by not specifically citing those vehicles.

As defined by the HCM 2000, a heavy vehicle is any vehicle which has more than four tires in contact with the driving surface. There is no distinction between trucks, recreational vehicles, and buses in the calculation of the adjusted saturation flow rate at signalized intersections. The HCM also recommends that if no data exist for a particular intersection, a value of 2% heavy vehicles should be used for urban streets. The PCE is implemented through a heavy-vehicle factor (f_{HV}), which is used to adjust the base saturation flow rate. This heavy-vehicle factor is one of several adjustment factors for the base saturation flow rate (S_0). In Equation 2-1 is shown how the base saturation flow rate is adjusted by the as it is defined in Equation 16-4 of the HCM 2000.

$$S = S_0 \times f_{HV} \qquad [2\text{-}1]$$

The form of the equation for f_{HV} (Equation 2-2) is:

$$f_{HV} = \frac{1}{\left(1 + P_T \times \left(E_T - 1\right)\right)} \qquad [2\text{-}2]$$

Where:

P_T = percentage of trucks in the traffic stream
E_T = passenger car equivalency factor

The standard procedure for measuring saturation flow rate (HCM2000, 16-158 in appendix H) prescribes that the headways of the first four to six vehicles in queue are not considered as saturation headway because this time is usually considered to be part of the start-up lost time. The HCM 2000 recommends a default start-up lost time (SLT) of 2.0 seconds if field measurements are not available. It is not specified for what traffic stream composition this value is based upon (e.g., passenger car only stream). This start up lost time can be determined by adding the difference between saturation headway and the headway measured for those first few vehicles that do not depart under the saturation headway.

Figure 2-1 illustrates this concept, where the saturation headway has a value of 2.0 seconds per vehicle (light shade) and the lost time is shown in this example as applying to the first four vehicles (dark shade).

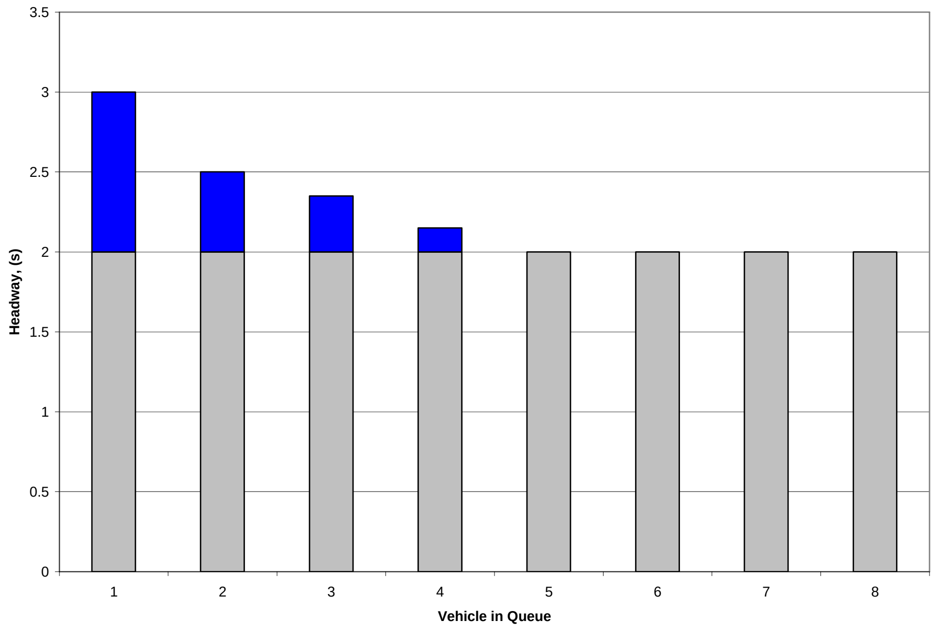


Figure 2-1. Measured headways including start-up lost time (dark shade)

Passenger Car Equivalency Factor at Signalized Intersections

In 1987, Molina [4] derived a PCE model based on the assumption that passenger cars depart at a constant saturation flow headway, and thus the basis for his model is the headway method. Molina collected field data from one site in each of three cities in Texas, and obtained a total of 13,000 observations. During the data collection, Molina considered vehicles to form a part of the queue if they came to a complete, or near stop. He recorded the time that these vehicles crossed the stop line. He classified the 13,000 vehicle observations into four vehicle classes and within each vehicle class into ten queue positions. He used the regression analysis method to develop a model of the collected data. An expression was derived for the additional effect of a heavy vehicle in the first position of a queue. This is a modified expression of the headway ratio method, and derived relationships consider only one heavy vehicle in the queue with its position varying from one to ten.

In addition, the analysis was limited to through movements only, and other factors, such as percentage of trucks, vehicular volumes, and headway increase of the eighth-positioned vehicle behind the truck are not considered. Molina found that position in queue did not have a pronounced effect when dealing with two- and three-axle and single-unit trucks, but had a very pronounced effect with five-axle combination trucks. His recommendations included using different methods to distinguish between light and heavy vehicles when analyzing capacity at signalized intersections, as these truck types can have significantly different effects.

Benekohal and Zhao [5] performed a study on the additional delay to the passenger cars behind a heavy vehicle. This delay is produced from both longer headways as well as additional headway increases of those vehicles behind the heavy vehicle that causes the delay. This study introduces a new PCE value labeled the D-PCE, meaning a delay-based calculation of passenger car equivalents. The delay-based passenger car equivalent is defined by Benekohal and Zhao as the ratio of delay caused by a heavy vehicle to the delay of a car in an all-passenger car traffic stream. The calculation of these D-PCE values considers the traffic volume as well as the percentage of heavy vehicles in the traffic stream. Data were collected at ten approaches of seven intersections in Central Illinois, where sites possessed as many ideal features as possible. Vehicle headways, delays for all-passenger car streams, number of queued and non-queued vehicles, the position of heavy vehicles in the queue, signal timing information, and geometric data were all collected in the process. The headway time of the first vehicle was defined from the moment the signal turned green to the point when the rear wheels of the vehicle crossed the stop line. This implies that reaction time was included in the start-up lost time.

Because TRAF-NETSIM queue delay was used for comparison, queue delay was the performance measure recorded in the field. Benekohal and Zhao concluded that the position of the truck in queue is not as important as how many vehicles are behind the truck. Also, comparison with the HCM values for single unit trucks indicates that the HCM overestimates the effect by which capacity is reduced by these types of vehicles at signalized intersections. The D-PCE increases with the number of vehicles behind a heavy vehicle, and PCE values for signalized intersections should be determined based on additional delay caused by large trucks.

Kockelman and Shabih [6] performed a study of the impact of light-duty trucks (LDTs) on the capacity of signalized intersections. Three factors were identified as influencing vehicle headways: length, performance, and driver behavior. Kockelman and Shabih used the headway method to arrive at PCE values for five different categories of light-duty trucks, where the additional time it takes for a passenger car behind an LDT to enter the intersection relative to being behind a passenger car is considered. The start-up lost time in their developed model did not include the reaction time of the first driver, as measurements began only when the first vehicle began to move (i.e., they did not record the start of green). In addition, only those vehicles that came to a complete stop before the signal changed to green were considered to have been part of the queue. Field data were collected from sites that met the following criteria:

- High traffic volumes and significant queuing
- Level terrain
- Exclusive left turn lane and protected signal phase for left turns
- Exclusive right turn lane
- Ease of data collection equipment setup
- Mix of vehicle types
- No parking zones along streets
- Insignificant disturbance from bus stops

The time elapsed from when the first vehicle in the queue began to move to when the rear axle of the last vehicle in the queue crossed the stop line was measured. After a statistical analysis, Kockelman and Shabih concluded that vehicle length is a significant factor on following-vehicle headways. As a result, the impacts of LDTs should be given special consideration, as sport-utility vehicles as well as vans have the ability to reduce the capacity at a signalized intersection in a statistically significant manner. Kockelman and Shabih recommended PCE values of 1.07, 1.41, 1.34, and 1.14 for small SUV, long SUV, vans and pick-up trucks respectively.

Bonneson and Messer [7] performed a study in which several models were developed that can be used to predict the saturation flow rate and start-up lost time of through movements at signalized interchange ramp terminals and other closely spaced intersections. The minimum discharge headway method is used, and it is typically reached by the vehicle in the sixth position of the queue. In their models they include the term "traffic pressure", defined as the tendency for vehicle headways to decrease as queue lengths increase and aggressive driving is present. Bonneson and Messer indicate that other authors (Stokes et al.) have independently identified this occurrence and call it "headway compression." Data for the study were collected at twelve interchanges in five states. The sites selected for analysis contained the two basic forms of interchanges, partial cloverleaf and diamond. Video cameras and computer-monitored tape switch sensors, mounted at the upstream end of each of two street segments, were used for data collection. The data were collected during weekdays, between 7:00 a.m. and 7:00 p.m.

The study concluded that there exists a strong correlation between start-up lost time and saturation flow rate, and that the distance to a downstream queue as well as traffic pressure has a significant effect on the saturation flow rate of a signalized traffic movement. Therefore, start-up lost time is not a constant value as it is commonly used in practice, but rather dependent upon the saturation flow rate. Bonneson and Messer recommended that an ideal saturation flow rate of 2000 pc/h/ln should be used for high volume intersections in urban areas.

Bonneson et al. [8] studied the formulation of the equation 16-4 of the HCM 2000 for saturation flow rate at intersections. The research resulted in the development of some new adjustment factors such as area population, number of lanes, and right-turn radius and the revision of some existing factors such as right turn, traffic pressure, and heavy vehicle. These adjustment factors were developed from data collected five different counties in Florida. Data were collected at 12 intersections, which included a total of 38 approaches and 2901 cycles in overall.

While not a focus of this study, a truck PCE factor was included in the model, as some trucks were present in the data set. Their estimated PCE value was 1.74, which interestingly is smaller than the HCM recommended value of 2.0. However, the authors indicate that truck percentages were not significant in their data set, and they specifically recommend a more thorough investigation of this specific factor.

Perez-Cartagena and Tarko [9] in their study focused in the development of local values of the base saturation flow rate and lost times used in capacity analysis of signalized intersections in Indiana. In their study, it was considered that the first four vehicles in queue were carrying the SLT. They found that some of the default values recommended by the HCM were adequate for their location. These factors included the heavy vehicle factor (f_{HV}). They also found that the saturation flow rate was not the same for all their sites even though they were almost identical in terms of geometry and traffic conditions. This indicated that there are some other factors affecting the saturation flow rate that are not considered in the HCM. Perez-Cartagena and Tarko proposed population adjustment factors of 0.92 for medium towns and 0.79 for small towns to be added to the adjusted saturation flow equation 16-4 in HCM 2000.

Li and Prevedouros [10] examined saturation headway and start-up lost times of traffic discharging from a signalized intersection. Their study was done using data collected from one through movement and one protected left turn at a single intersection. They proved that the assumption of that the saturation headway of short and long queues is the same was overlooking other factors that might be present. It was observed how the last few vehicles in a longer queue can produce either compressed or elongated headways.

Compressed headways were observed when vehicles bunch together to be able to cross the stop bar before the clearance interval is over. Elongated headways were observed when the queues were long enough to allow the vehicles to exceed speeds of 40 mph. Additionally, they found that the minimum headway was not reached until the 9^{th} to 12^{th} vehicle in queue. In addition Li and Prevedouros recommended a mean start-up reaction time of 1.76 seconds with a standard deviation of 0.61.

Car-Following Models

Cohen investigated the issue of simulating queue discharge at a signalized intersection through the application of the modified Pitt car-following model [11]. This study was based on a single intersection with no restrictions on the departing flow. Some queue-discharge mechanisms are based on the assumption of every vehicle in queue departing from the intersection at equal time headways. These assumptions are neglecting the start-up delay brought from the first few cars in queue and other issues such as varying vehicle and driver characteristics, among others.

The basic form of the modified Pitt car-following model is shown in Equation 2-3.

$$a_f(t+T) = \frac{K \times \left\{ \begin{array}{l} s_l(t+R) - s_f(t+R) - L_l - h \times v_f \\ + \left[v_f(t+R) - v_l(t+R) \right] \times T - \dfrac{1}{2 \times a_l(t+R) \times T^2} \end{array} \right\}}{T \times \left(h + \dfrac{1}{2 \times T} \right)} \qquad [2\text{-}3]$$

This model estimates an acceleration for a following vehicle, subject to three constraints: a_f has to be between a_{min} and a_{max}; the speed at any time t has to be less than the free-flow speed; and a_f has to be less than the acceleration computed for safe following (Equation 2-4):

$$a_{fe}(t+T) = \frac{-0.5 \times v_f^2(t+R)}{s_l(t+T) - s_f(t+T) - L_l - \frac{v_{l\min}^2(t+T)}{2 \times a_{l\min}}} \qquad [2\text{-}4]$$

In these two equations the variables are defined as:

$a_f(x)$	= acceleration trailing vehicle at time x, computed from car-following (ft/s^2)
t	= current simulation time (sec)
T	= simulation time-scan interval (sec)
K	= sensitivity parameter used in modified Pitt car-following model
$s_l(x)$	= position of lead vehicle at time x as measured from upstream (ft)
R	= perception-reaction time (assumed to be equal for all vehicles) (sec)
$s_f(x)$	= position of follower vehicle at time x as measured from upstream (ft)
L_l	= length of lead vehicle plus a buffer based on jam density (ft)
h	= time headway parameter in Pitt car-following model (buffer headway) (sec)
$v_f(x)$	= speed of follower vehicle at time x (ft/s)
$v_l(x)$	= speed of lead vehicle at time x (ft/s)
$a_l(x)$	= acceleration of lead vehicle at time x (ft/s^2)
a_{fe}	= acceleration of follower vehicle as computed from safe following (ft/s^2)
v_{lmin}	= minimum speed of lead vehicle (ft/s)
$a_{lmin}(v_f)$	= minimum acceleration of lead vehicle (maximum deceleration) (ft/s^2)

For purposes of the research, the initial movement of the vehicles was defined once they reached the speed of one foot per second. Furthermore, it was found that an appropriate value for the parameter K should be greater than 1. This is taking into consideration the assumption that a driver in a queue behaves significantly different than a driver in free-flowing traffic. The best fit for this value in the study was 1.25.

Greenshields [12] indicates that the vehicle headways become fairly constant after the fifth vehicle. This indicates that the first four cars should not be used for the discharge headway calculations. These cars are the major carriers of the start-up lost time. The results of this research include only passenger cars.

In the study it was found, as expected, that longer vehicle length results in longer discharge headways. Furthermore, it was found that the expansion wave speed is independent of the Free

Flow Speed (FFS) but it increases when the following distances are closer. The expansion wave may slow down when the vehicles' average speed reaches 30 mph. This is reasonable since acceleration decreases at higher speeds.

This research indicates that the impact of a truck is greater if the truck is positioned in the first few positions. Additionally, it is recommended that the *K* parameter should be calibrated with vehicles in the second and third position since they are the vehicles most affected.

Bonneson [13] studied and summarized previous researches of modeling discharge headways at signalized intersections. Bonneson selected a model based on vehicle and driver capabilities, including driver reaction time, driver acceleration, and vehicle speed. Then, he used field data from five different sites to calibrate the model.

The Briggs headway model [14] assumes that the acceleration for each vehicle in queue remains constant. This model separates the vehicles not by position but by distance to the stop bar in comparison with the distance they need to reach their desired speed. This distance is denoted as d in Equation 2-5.

If

$$n \times d < d_{\max} \qquad [2\text{-}5]$$

Then

$$h_n = T + \frac{\sqrt{2 \times d \times n}}{A} - \frac{\sqrt{2 \times d \times (n-1)}}{A} \qquad [2\text{-}6]$$

Otherwise

$$h_n = T + \frac{d}{V_q} \qquad [2\text{-}7]$$

With

$$d_{\max} = \frac{V_q^{\,2}}{2 \times a} \qquad [2\text{-}8]$$

Where:

n = queue position (n = 1, 2, 3…)
d = distance between vehicles in a stopped queue (ft)
d_{max} = distance traveled to reach speed V_q (ft)
h_n = headway of the nth queued vehicle (sec)
T = driver starting response time (sec)
a = constant acceleration of queued vehicles
V_q = desired speed of queued traffic (ft/s)

Messer and Fambro [15] found that regardless of the position of the driver, the response time was 1.0 sec. Although it was found that an additional delay of 2.0 sec should be allocated to the first driver. They also found that an average length for each queue position is 25 ft.

To these values it is important to add the additional length and delay due the vehicle type. In this report the vehicle composition of the queue is not specified.

Buhr et al. [16] conducted a study of driver acceleration characteristics on freeway ramps. This study considers only passenger cars from a stopped condition. The authors determined that acceleration decreased linearly with increasing speed according to Equation 2-9.

$$a = A_{\max} \times \left(1 - \frac{V}{V_{\max}}\right) \qquad [2\text{-}9]$$

Where:

a = instantaneous acceleration (ft/s^2)
A_{max} = maximum acceleration (ft/s^2)
V = velocity of vehicle (ft/s)
V_{max} = maximum speed corresponding to zero acceleration (ft/s)

For on-ramps at level terrain, the authors found an A_{max} = 15 ft/s^2 and V_{max} = 60 ft/s. A later study conducted by Evans and Rothery [17], studied the acceleration and speed characteristics of queued drivers. The results show an exponentially increasing speed to a desired speed of approximately 50 ft/s. This speed is reached by the vehicle in 25th position, which can be considered as normal cruising speed when crossing the stop bar.

Previous studies indicate that the discharge headway (h) between the n^{th} vehicle and the $(n-1)^{th}$ vehicle has two components. First, the time that it takes from the beginning of the green time to the driver to start moving. This can be estimated as $\tau + n \times T$, where τ is the additional time for the first driver and T is the individual response time. The second part is the time that it takes the vehicle to reach the stop bar. Briggs assumed that each vehicle in queue will occupy the same amount of space. The final form of the proposed headway model is shown in Equation 2-10.

$$h_n = \tau \times N_1 + \mathrm{T} + \frac{d}{V_{\max}} + \frac{V_{st(n)} - V_{st(n-1)}}{a_{\max}} \qquad [2\text{-}10]$$

Where:

h_n = headway of the nth queued vehicles (sec)
τ = additional response time of the first queued driver (sec)
$N_1 = \begin{cases} 1 \text{ if it is the first vehicle} \\ 0 \text{ otherwise} \end{cases}$
T = driver starting response time (sec)
d = distance between vehicles in a stopped queue (ft)
V_{max} = maximum speed (ft/s)
$V_{st}(n)$= stop line speed of the nth queued vehicle (ft/s)
A_{max} = maximum acceleration (ft/s2)

This approach does not consider any kind of mixed traffic. Another limitation in this model is that it does not consider any variation in the inter-vehicle length or any other buffer length between the two vehicles.

In conclusion, this study says that the minimum discharge headway is dependent on driver response time, desired speed and traffic pressure for each movement. It states that the minimum discharge headway is not reached until the eighth vehicle. This model also suggests that under ideal conditions the discharge headway should be shorter than 2.0 sec/veh.

Akçelik et al. [19] describe an exponential queue discharge flow and speed model. In this study they model queue discharge speed in addition to the headway. By including speed it is easier to develop relationships for traffic parameters like vehicle spacing, density, time and space occupancy ratios, gap time, occupancy time, space time and acceleration characteristics.

Equations 2-11, 2-12, and 2-13 show the models for speed, flow and headway developed by Akçelik, et al [19]:

$$v_s = v_n[1 - e^{-m_v(t-tr)}] \qquad [2\text{-}11]$$

$$q_s = q_n[1 - e^{-m_q(t-tr)}] \qquad [2\text{-}12]$$

$$h_s = \frac{h_n}{[1 - e^{-m_q(t-tr)}]} \qquad [2\text{-}13]$$

Where:

v_s = queue discharge speed at time t *(km/h)*
v_n = maximum queue discharge speed (km/h)
m_v = parameter
t = time since the start of green (seconds)
t_r = start response time (constant) average from all drivers in 1st position
q_s = queue discharge flow rate at time t (veh/h)
q_n = maximum queue discharge flow rate (veh/h)
m_q = parameter
h_s = queue discharge headway at time t (seconds)
h_s = $3600/q_s$
h_n = minimum queue discharge headway (seconds)
h_q =$3600/q_n$

Pipes [20] describes the ideal space or distance headway as one car length for every ten miles per hour of speed at which the follower vehicle is traveling. The resulting equation is shown in Equation 2-14.

$$d_{\min} = \left[x_n(t) - x_{n+1}(t)\right]_{MIN} = L_n\left[\frac{v_{n+1}(t)}{(1.47)(10)}\right] + L_n \qquad [2\text{-}14]$$

Where:

d_{min} = distance between vehicles at time t (ft)
x_n = location of lead vehicle at time t (ft)
x_{n+1} = location of follower vehicle at time t (ft)
L_n = length of lead vehicle (ft)
v_{n+1} = speed of the following vehicle at time t (mph)

In this case the headway depends more on the length of the lead vehicle. For example, say a passenger car is following a truck, the ideal distance will be using the lead vehicle's length. In this case it will result in an extremely large headway. On the other hand, if a truck is following a passenger car, the model gives a smaller headway. It is obvious this model is not taking in consideration the driver behavior and the braking capabilities of the vehicles. Previous studies have found that it takes longer to a truck to stop. Therefore, the model is not compatible with real life scenarios.

This model is relatively easy to use, the mathematical operations are simple. The big limitation of this model is that it does not take into consideration any kind of interaction between vehicles. The acceleration is considered to be constant and there is no way to allocate heavy vehicles.

As appear in May [21], Forbes' theory approaches the car-following by considering the reaction time of the follower. The minimum gap between the vehicles should be greater than the reaction time of the drivers. In addition to the minimum gap, this model considered the length of the vehicles. In Equation 2-15 is expressed this mathematical relationship.

$$h_{MIN} = \Delta t + \frac{L_n}{v_n(t)} \qquad [2\text{-}15]$$

Where:

h = time headway (seconds)
Δt = reaction time, assumed to be 1.5 sec (seconds)
L_n = length of lead vehicle (ft)
v_n = speed of the lead vehicle at time t (mph)

May [21] also presents the studies developed by a group of researchers from General Motors about car-following theories. These studies were more extensive than the studies made for Pipes' model and Forbes' model. It also has a particular importance because it is based on an empirical model. After four previous intents or versions, GM came up with the 5th and final model. This eliminates the discontinuities in the previous versions. This final model is shown in Equation 2-16.

$$a_{n+1}(t+\Delta t)=\frac{\alpha_{l,m}\left[v_{n+1}(t+\Delta t)\right]^{m}}{\left[x_{n}(t)-x_{n+1}(t)\right]^{l}}\left[v_{n}(t)-v_{n+1}(t)\right] \qquad [2\text{-}16]$$

Where:

$a_{n+1}(x)$	= acceleration of follower vehicle at time x (ft/s2)
Δt	= reaction time (seconds)
α	= sensitivity parameter with speed and distance exponents m and l respectively
m	= speed exponent
l	= distance exponent
$v_{n+1}(x)$	= speed of follower vehicle at time x (ft/s)
$v_n(x)$	= speed of lead vehicle at time x (ft/s)
$x_n(x)$	= position of lead vehicle at time x as measured from upstream (ft)
$x_{n+1}(x)$	= position of follower vehicle at time x as measured from upstream (ft)

This model includes the acceleration characteristics of the following vehicle and yet it does not include a variation for vehicle type or size. However, the parameters m and l can be calibrated by individual vehicle types to adjust the acceleration of the following vehicle. Another limitation of this model is the lack of consideration for the vehicle's length.

Long [22] found that passenger cars, SUVs, and vans length averaged around 15 ft. These vehicles length vary from 10.9 to 19.7 ft, with approximately two-thirds of the sample range between 13 and 16 ft. It was also found that when any of these cars were pulling a trailer, their total length increase in average to 35 ft.

The truck distribution was not as close to a normal distribution as the PC's were. Additionally, only an approximated 12 % of the trucks were as long as the WB-50 design length (55 ft) or shorter. Long [22] found that combination trucks typically have a length of 65 ft.

Using the grouping that AASHTO has follow for years regarding acceleration capabilities, typical lengths of 15, 65 and 30 ft can be used for PCs, combinations trucks and all others vehicle respectively. This research also specifies some other types of trucks that can potentially be used as guidance.

Then an expected average vehicle length (EVL) could be estimated with weighted average using the expected proportion for each one of the groups. Long also recommend a reasonable inter vehicle spacing of 12 ft in contrast to the 3 ft default value in CORSIM.

Summary of HCM 2000 PCE Guidelines

- Currently, a PCE value of 2.0 is specified for all large vehicles.
- No distinction is made between trucks, recreational vehicles, and buses in the calculation of the adjusted saturation flow rate at signalized intersections. It applies to any vehicle with more four tires in contact with the driving surface.
- The headways of the first four to six vehicles in queue are not considered as part of the saturation headway.
- A default start-up lost time (SLT) of 2.0 seconds, if field measurements are not available, is recommended.

Summary of Previous PCE Studies

There are no recent studies that have examined or determined heavy vehicle PCE values for signalized intersections. There are previous studies that demonstrate how heavy vehicles have an impact on traffic streams at an intersection. Of all the research reviewed, headway was by far the most common performance measures used to base PCE values on. Significant findings from these studies are as follows:

- Molina derived a different expression for the additional effect of a heavy vehicle in the first position of a queue. Molina also found that position in queue did not have a pronounced effect on two- and three- axle and single-unit trucks.
- Benekohal and Zhao concluded that other than being in the first position, the position of the heavy vehicle in the rest of the queue does not matter, but yet the number of cars behind the heavy vehicle has a significant impact.
- Benekohal and Zhao concluded that at signalized intersections, the additional delay caused by larger trucks should be used for calculating PCEs. They introduced a new term, D-PCE, meaning a delay-based calculation for PCE.

- Kockleman and Shabih found that three factors influence the vehicle headways: length, performance and driver behavior. In addition they recommended PCE values of 1.07, 1.41, 1.34, and 1.14 for small SUV, long SUV, vans and pick-up trucks respectively.

- Bonneson et al. developed new and revised adjustments to the saturation flow equation. They estimated a PCE value 1.74 for heavy vehicles, but that was based on a data set with minimal truck observations.

- Perez-Cartagena and Tarko stated that the f_{HV} is adequate for use in Indiana, although they recommend an additional factor for population.

- Li and Prevedouros found that the minimum headway is not reached until the 9^{th} vehicle crosses the stop bar. They also recommended a mean start-up reaction time of 1.76 seconds with a standard deviation of 0.61.

Summary of Car-Following Models

There were several car-following models reviewed in this chapter including from the first few models developed to the most recent ones. Table 2-1 summarizes the details that were taken into consideration at the moment of selecting which car-following model was going to be used.

Table 2-1. Car-following models comparison

	Pipes	Forbes	GM	Modified Pitt
Constant acceleration	Yes	Yes	No	No
Acceleration of the leading vehicle	No	No	No	Yes
Speed at stop bar	No	No	Yes	Yes
Following distance	Fixed through vehicle type	Fixed	Fixed	Fixed but can be adjusted with vehicle's length
Vehicle length	Yes	Yes	No	Yes
Length variation for vehicle type	No but can be adjusted	No but can be adjusted	No	Yes but have to be adjusted
Calibration difficulty	High	High	High	Medium
Implementation difficulty	Low	Low	Low	Low
Computational efficiency	High	High	Medium	Medium

Of these five car-following models studied, the modified Pitt model fits better to apply the model to a signalized intersection with heavy vehicles in the stream. In the model, the summation of the length of each vehicle plus a buffer length determined at jam density is known as L. For passenger cars, a value of 20 ft is used for this length L. This value should be reconsidered due the findings in Long's study [22]. This value should be the sum of two parts.

The first part should be the length of the leading vehicle, and second the minimum or allowable inter-vehicle spacing. By doing this, a more detailed composition of the traffic stream can be reached.

CHAPTER 3
RESEARCH APPROACH

This chapter describes the research approach that was used to accomplish the objectives of this study. More specifically, it will discuss the methodological approach, field data collection, simulation model development, and the simulation experiments.

Methodological Approach

As discussed in the literature review chapter, two different methodological approaches have been used in previous PCE related studies. One was based on the concept of a delay-based passenger car equivalent for trucks. The other was based on the concept of a time headway passenger car equivalent. The latter is the approach that is currently used in the Highway Capacity Manual [1]. One of the main objectives of this project was to update the PCE values used for large trucks in FDOT's ARTPLAN software [23]. Since the ARTPLAN calculation methodologies are largely based on the HCM analysis framework, it was decided to use the time headway approach for purposes of consistency.

In the headway based approach to PCE calculation, it is important to recognize that the time headway value between successive vehicles is a function of both the leading vehicle and the trailing vehicle. Thus, a PCE value that is determined for a particular type of vehicle must account for a variety of different vehicles that may precede it in a queue. For example, the headway between a passenger car (leader) and large truck (follower) will be different than that between a large truck and a large truck, all else being equal. Of course, it is not reasonable to expect practitioners to collect the exact sequence of vehicles in queue during data collection activities. Generally, the data collection is limited to just a percentage of trucks in the traffic stream, not their actual positions in the traffic stream as well. Furthermore, this sequence will usually be different from cycle-to-cycle. Thus, the PCE values that are derived must be somewhat generalized so that the sequence of vehicles is not an input to the selection of a PCE value.

While it might seem desirable to develop the PCE values based on a large number of the different leader-follower vehicle combinations, there are some practical concerns with this approach. First, certain combinations of vehicles will occur very rarely in the traffic stream; for example, a recreational vehicle pulling a trailer that is following a motorcycle, or vice versa, as the individual frequencies of occurrence for these vehicle types is quite small. Second, from a data collection perspective again, it is only reasonable to expect practitioners to classify vehicles into just a few different categories. Therefore, for this project, it was decided to consider just three different truck types, in addition to the passenger vehicle.

Different types of trucks have different impacts on the traffic stream; for example, one single-unit truck does not take up the same amount of space as one tractor+semi-trailer, and their acceleration capabilities are most likely different as well. Therefore, heavy vehicles were categorized as either a small, medium or large truck depending on their size and operational characteristics. The details of each category will be described in a later section in this chapter.

The end result of this project is the development of three different PCE values applicable to three categories of heavy vehicles which can be used to calculate a heavy vehicle factor (f_{HV}) analogous to the current HCM method, as shown in Equation 3-1.

$$f_{HV} = \frac{1}{\left(1 + P_{ST} \times (E_{ST} - 1) + P_{MT} \times (E_{MT} - 1) + P_{LT} \times (E_{LT} - 1)\right)} \qquad [3\text{-}1]$$

Where:
f_{HV} = adjustment factor for heavy vehicles in traffic stream
P_i = proportion of truck type i in traffic stream
E_i = PCE factor for truck type i
i = LT for large truck, MT for medium truck, and ST for small truck

According to the procedure in the HCM to obtain the saturation flow rate, it was necessary to measure the saturation headway at each intersection with at least eight vehicles in queue. Since it was established that the headway of each vehicle depends on the leading and trailing vehicle and that there were three different truck types under study, the number of possible leader-follower combinations is 16 (4^2). Table 3-1 enumerates these 16 combinations.

Table 3-1. Possible pairing combinations for four vehicle types*

PC – PC	*ST – PC*	*MT – PC*	*LT – PC*
PC – ST	*ST – ST*	*MT – ST*	*LT – ST*
PC – MT	*ST – MT*	*MT – MT*	*LT – MT*
PC – LT	*ST – LT*	*MT – LT*	*LT – LT*

* *PC* = passenger car, *ST* = small truck, *MT* = medium truck, *LT* = large truck

Since the focus of this study was on truck PCE values, it was obviously necessary to find sites with a relatively high percentage of large trucks. For sites with less than 10% trucks in the traffic stream, a very large percentage of the queues would have only zero or one truck in it. Thus, the number of cycles that would need to be collected to obtain a significant number of queues with multiple trucks present would be extreme, thus leading to a very lengthy and inefficient data collection process. Thus, only sites with a truck percentage of at least 10% were considered for data collection. However, sites with persistent queues of at least 8 vehicles and 10% or more large trucks are not very common. Sites with a lot of traffic (thus generating the necessary queue lengths) typically have small truck percentages, and sites with high truck percentages (such as along truck routes) do not typically have high overall volumes. With the data collection resources available for this project (i.e., time, money, labor), it was not feasible to obtain enough data from enough sites to facilitate the objectives of this project from field data alone. Therefore, the decision was made to collect as much field data as project resources would allow, and then use these field data to calibrate a simulation model that would be used to provide the full data set upon which to base the development of the truck PCEs. The collection of the field data is described in the next section, and the development and application of the simulation model in the section after that.

Field Data Collection

In order to reduce the impact of additional factors that could affect the effectiveness of the intersection, and to obtain the necessary amount of data, it was essential for the sites selected to contain certain characteristics. The criteria to select these sites for field data collection are outlined below.

Site Selection Criteria

Geometry

- Typical four leg intersection (turning radii at or close to 90°)
- At least one site with only one through lane. The other sites should have two or three travel lanes in the through direction
- The approach must have an exclusive left turn lane(s), but the queue cannot spill back onto through lanes
- Sites with an exclusive right turn lane are preferred, but not absolutely necessary if the site has a small percentage of right turning vehicles
- Level terrain is preferred, but sites with small grades are acceptable
- No curbside parking or bus stops near the intersection, or other external factors that will significantly influence the saturation flow rate

Traffic

- Queue lengths of at least 10 veh/lane at the beginning of green should be regularly present during the data collection period. This is a function of the signal g/C ratio and cycle length, in addition to the overall traffic demand. For example, for a g/C ratio of 0.4 and a cycle length of 60 seconds, an average hourly flow rate of at least 1000 veh/hr/lane is needed. Additionally, if progression is favorable, then the volumes would need to be adjusted upward slightly, as a higher percentage of vehicles would be arriving on green.
- A heavy vehicle percentage of 10% or higher in the traffic stream.

- The operations of the observed approach must not be impacted by a downstream queue. That is, vehicles must be able to depart freely from the subject approach.

From these criteria, and assistance from FDOT personnel, six sites were ultimately chosen for field data collection. These sites are listed in Table 3-2.

Table 3-2. Data collection cites

Site #	Intersection street names	City
1	SW Williston Rd / SW 34th St	Gainesville
2	Waldo Rd / University Ave	Gainesville
3	US 41 / SR 50	Brooksville
4	US 301 / SR 50	Brooksville
5	SR 326 / CR 200A	Ocala
6	John Young Parkway / Colonial Drive	Orlando

Sites Selected

Herein are shown aerials and ground level views from each site selected.

Figure 3-1. SW Williston Rd / SW 34th St aerial view (eastbound on Williston Rd)

Figure 3-2. SW Williston Rd / SW 34th St ground level view (eastbound on Williston Rd)

Figure 3-3. University Ave. / Waldo Rd aerial view (northbound on Waldo Rd)

Figure 3-4. University Ave. / Waldo Rd ground level view (northbound on Waldo Rd)

Figure 3-5. US 41 / SR 50 aerial view (westbound on SR50)

Figure 3-6. US 41 / SR 50 ground level view (westbound on SR50)

Figure 3-7. US 301 / SR 50 aerial view (eastbound on SR 50)

Figure 3-8. US 301 / SR 50 ground level view (eastbound on SR 50)


Figure 3-9. CR 326 / SR 200A ground level view (westbound on CR 326)

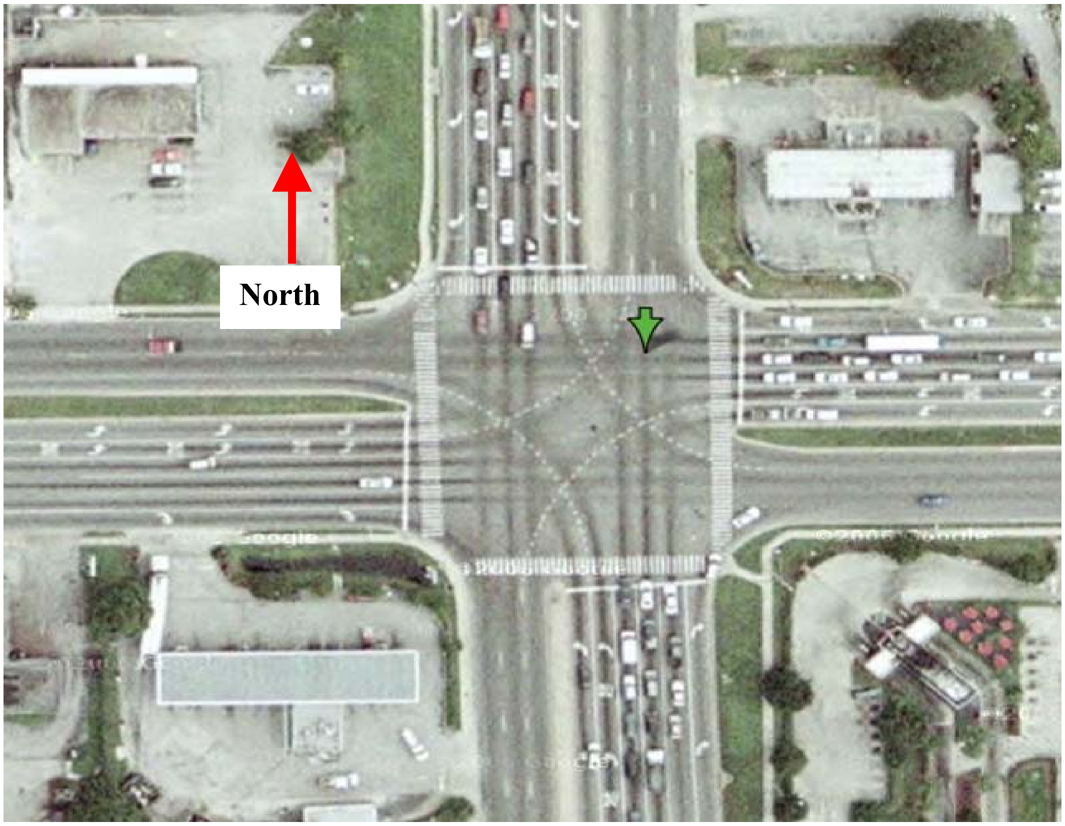


Figure 3-10. John Young Parkway / Colonial Dr aerial view (southbound on John Young Pkwy)


Figure 3-11. John Young Pkwy / Colonial Dr ground level view (southbound on John Young Pkwy)

Data Collection Methods

Two different methods for data collection were used. One method used a single camera along with equipment installed in the signal controller cabinet to obtain signal status information concurrent with the video signal. This method is referred to as "method 1," and was used for only the Williston Rd/34th St site. The other method used two cameras to obtain both traffic and signal status information, and is referred to as "method 2." Each of these methods is described in the following sections.

Data collection equipment for method 1

For this method, it was necessary to be able to set up the following devices inside of the signal controller cabinet: a VCR, a signal encoding device, and current sensors that attach (passively) to each of the green bulb/LED power wires. Figure A-1 illustrates the necessary connections. A picture of the installation of the equipment in a signal controller cabinet is shown in Figure A-2.

A video camera was mounted on the mast arm facing the approach of interest. An additional constraint for this setup was that the camera be mounted on a mast arm that is in the same quadrant as the signal controller cabinet to easily facilitate running the video/power cable from the video camera to the controller cabinet. Figure 3-12 illustrates the camera setup at the Williston Rd/34th St intersection.

The stop bar and back of queue must be visible within the camera field-of-view (FOV), as demonstrated in Figure 3-13. Figure 3-13 also shows how the video image and the signal status information are combined into a composite view to facilitate data reduction. This is accomplished through special processing hardware and software that is too detailed to explain here. For more information on this system, see Washburn et al. [2].

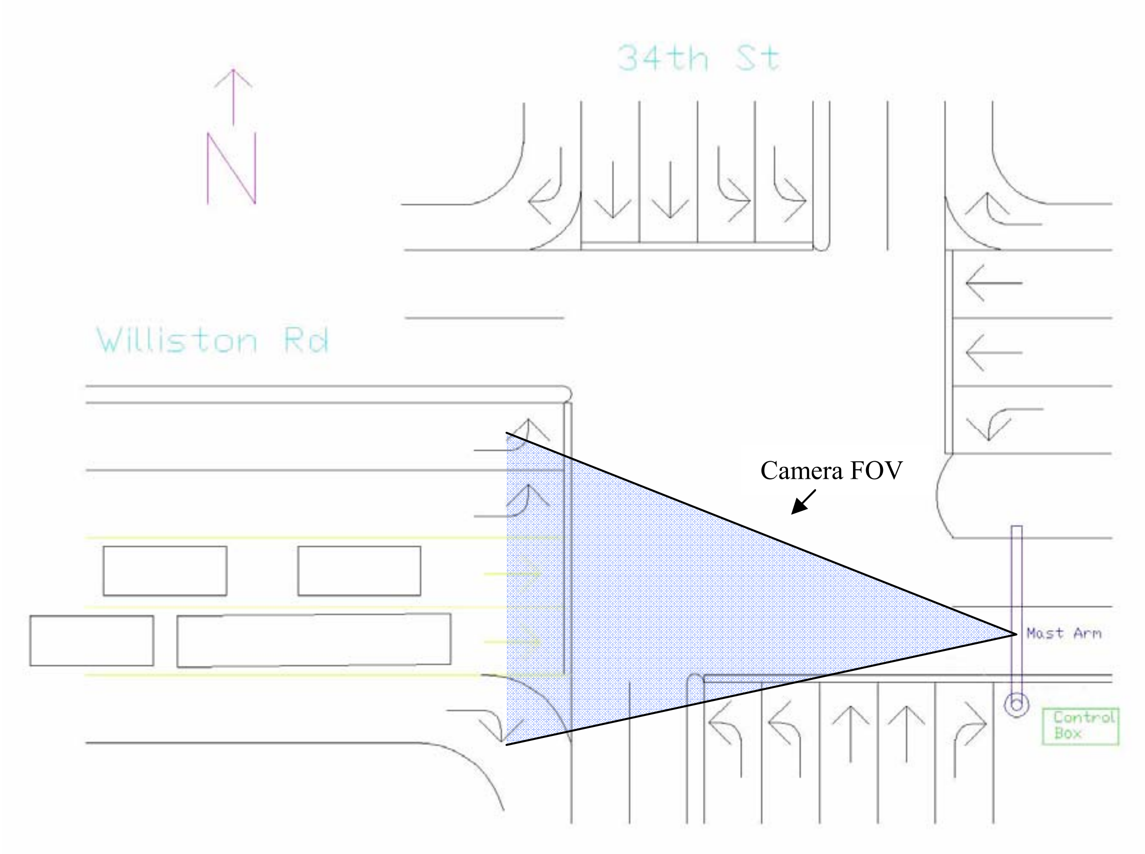


Figure 3-12. Preferred video camera mounting location (plan view) for method 1

Figure 3-13. Screen capture (low resolution) of video image from Williston Rd/34th Street site in Gainesville (method 1)

Data collection equipment for method 2

To be able to implement the first method for data collection requires the cooperation of the local agency that maintains the signals. Due to an existing relationship with the City of Gainesville, this cooperation was easily facilitated. However, it was not so easily facilitated in the other jurisdictions. Thus, it was decided to use a different data collection approach for sites outside of Gainesville to avoid this complication. This second method requires no access to the control cabinet or the need of the presence of local agency staff. This method consists of having two cameras at the site at ground level. One camera is placed in a position where it can focus on the traffic signal head and at the same time see a fair amount of the queue. The second camera is placed where it has a FOV that includes the stop bar and the first couple of cars in the queue. This setup is illustrated in Figure 3-14.

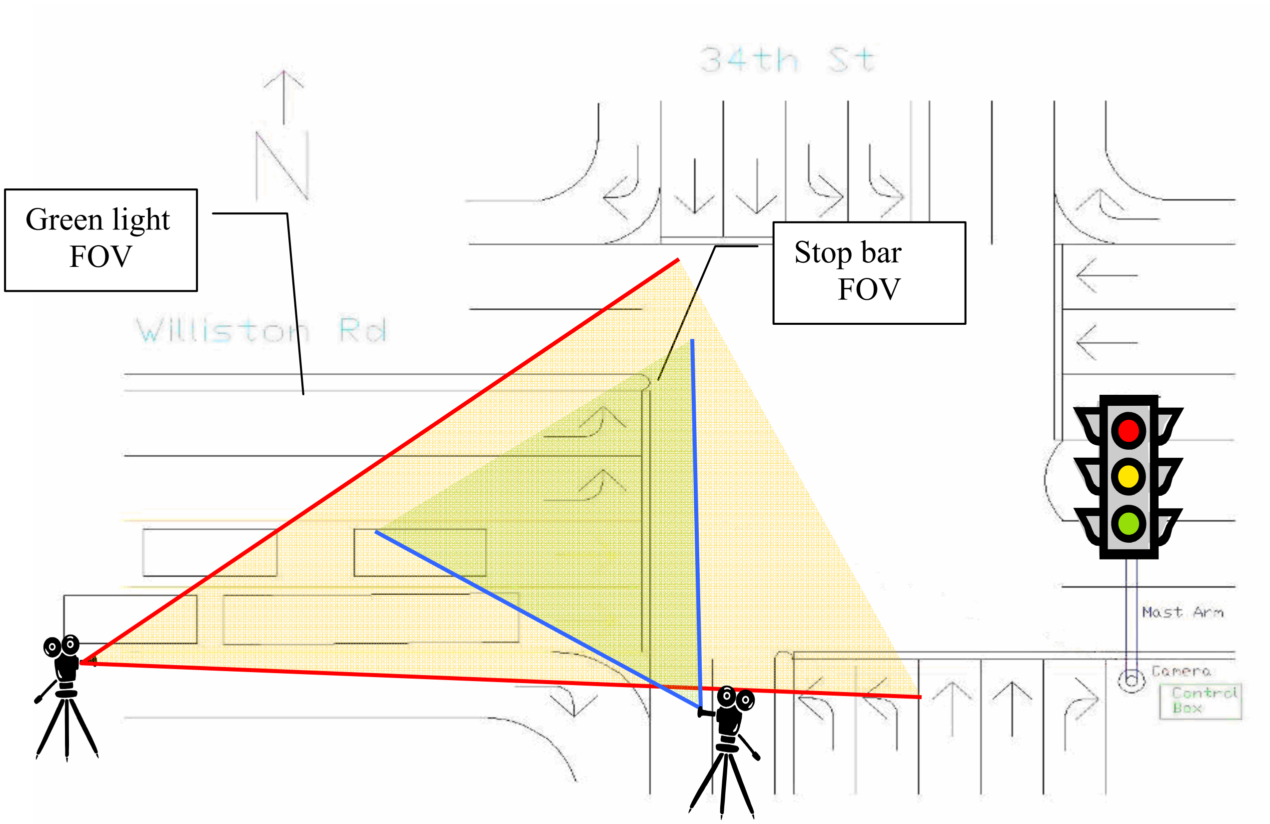


Figure 3-14. Camera setup for data collection (method 2)

For this method, if the back of queue was not visible within the camera FOV for some cycles, it was recorded manually. Method 2 was used for data collection at the other five sites. While one of the other sites was in Gainesville, method 1 could not be used at this site due to complications of running the necessary cables from the video camera through the in-ground conduit to the signal controller cabinet.

To facilitate data reduction with this method, the two camera views had to be combined into a single composite image, along with a timer, as shown in Figure 3-15. This was accomplished with a video mixer, a software program that generated the timer, and another software program that merged the timer with the output of the two camera images from the mixer.

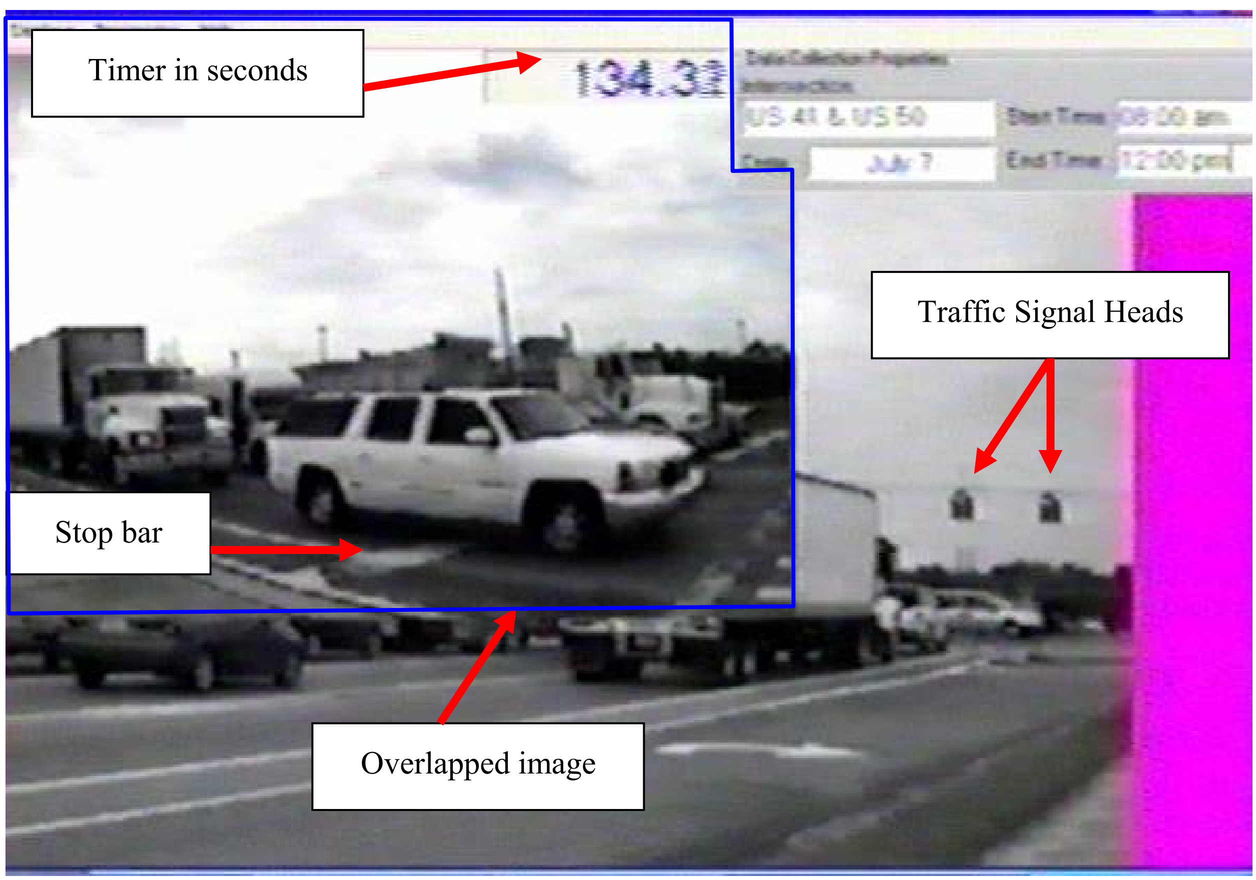


Figure 3-15. Screen capture of the composite video image (method 2)

Data Collection Periods

For the data collection performed at Williston Rd/34th St site, there was more flexibility in how much data could be collected, and at what times data could be collected. This was because with method 1, the VCR in the controller cabinet could be programmed to record any time period, and the City of Gainesville allowed the research team to access the cabinet to change VCR tapes at our discretion. For data collection with method 2, it was necessary to be present on site during the entire duration of the data collection. Thus, due to this issue and travel costs and constraints, data collection was limited to a 4-hr period at the sites outside of Gainesville.

The data collection periods for the Williston Rd/34th St site (using method 1) are shown in Table 3-3.

Table 3-3. Data collection periods for Williston Rd/34th Street site (method 1)

Tape #	Date	Time(s)
2	3/2/2006	6:30 - 9:30 am; 11:30 am - 1:30 pm; 3:30 - 6:30 pm
5	3/31/2006	8:30 am - 4:30 pm
6	4/12/2006	6:30 am - 2:30 pm

The data collection periods for the sites using method 2 are shown in Table 3-4.

Table 3-4. Data collection periods for other sites (method 2)

Site	City	View[1]	Lanes	Tape num	Time interval	Total time hr:min	Date
1	Gainesville	Stop bar	2	1	3:30 – 5:30 pm	2	6/26/06
2	Ocala	Stop bar	1	1	8:00 – 12:00 pm	4	6/29/06
		Green		2	8:00 – 12:00 pm	4	6/29/06
3	Orlando	Stop bar	3	1	8:00 – 12:00 pm	4	6/30/06
		Green		2	8:00 – 12:00 pm	4	6/30/06
1	Gainesville	Stop bar	2	1	2:00 – 6:00 pm	4	7/05/06
		Green		2	2:00 – 6:00 pm	4	7/05/06
4	Brooksville	Stop bar	2	1	8:00 – 12:00 pm	4	7/07/06
		Green		2	8:00 – 12:00 pm	4	7/07/06
5	Brooksville	Stop bar	1	1	9:05 – 11:45 pm	2:40	7/07/06
		Green		2	9:35 – 11:45 pm	2:10	7/0706

[1] This indicates the field-of-view perspective of the cameras used for data collection at the site.

Data Reduction

The composite image videos were manually processed. The information that was recorded from each video included the time when the signal turned green and the time when the front axle of each vehicle in the queue crossed the stop bar. Also recorded was the type of each vehicle in queue, according to a predetermined vehicle classification scheme (either passenger car, small truck, medium truck, or large truck). Examples of the types of vehicles that fall into each of the truck categories are shown in Appendix B The individual headway data and lost times were used to develop the truck passenger car equivalent values for the three different classifications of trucks, as discussed in Chapter 4.

Due to equipment problems, tapes 2 and 6 at the Williston Rd/34th St site unfortunately did not have the start of green information. However, all other information could still be obtained from these videos.

It should be noted that the precision of the timing measurements is limited to 0.0333 seconds. This is a result of using video cameras and recording equipment that utilize the standard frame rate of 30 frames per second.

Simulation Modeling

This section describes the development of the simulation program, which was used for the generation of a much larger data set to base the development of the PCE values upon. While there are a variety of commercial simulation programs on the market that can simulate traffic flow at signalized intersections, it was decided to develop a custom simulation program. This decision was made for several reasons: 1) many commercial software programs do not readily provide detailed documentation about the underlying car movement models; 2) the car movement models contained in some programs are not very robust, particularly with respect to heavy vehicles, or do not provide for user modification of some key parameters within those models; and 3) with a custom simulation program, custom pre- and post-processing routines can also be developed for increased efficiency. Thus, by developing a custom simulation program, all aspects of the car movement models and other features of the program can be completely controlled.

The development of the simulation program involved aspects such as creating a user interface for specification of simulation scenarios and run settings and implementation of a car-following model. These efforts are described in the following sections.

Car-Following Model

The foundation of a traffic simulation program is the underlying mathematical models that describe the movement of vehicles along the roadway system. Thus, the first task in the development of the simulation program was the selection of a car-movement model that was suitable to a queue discharge situation at a signalized intersection.

Based upon the literature review in Chapter 2 for car-following models, and the needs of this research project, the Modified Pitt model was selected for implementation [11].

The Modified Pitt car-following model calculates an acceleration value for the trailing vehicle based on intuitive parameters, such as the speed and acceleration of the lead vehicle, the speed of the trailing vehicle, the relative position of the lead and trail vehicles, as well as a desired headway. Car-following models are generally based on a 'driving rule', such as a desired following distance or following headway. The Modified Pitt model is based on the rule of a desired following headway.

As indicated before, the headway of each vehicle depends of the leading and trailing vehicle and this model takes into consideration the physical and operational characteristics of both. The main form of the model is shown again in Equation 3-2.

$$a_f(t+T)=\frac{K\times\left\{\begin{array}{l}s_l(t+R)-s_f(t+R)-L_l-h\times v_f\\+\left[v_f(t+R)-v_l(t+R)\right]\times T-\frac{1}{2\times a_l(t+R)\times T^2}\end{array}\right\}}{T\times\left(h+\frac{1}{2\times T}\right)} \qquad [3\text{-}2]$$

Where:

$a_f(t+\mathrm{T})$ = acceleration of follower vehicle at time $t+T$, in ft/s^2
$a_l(t+R)$ = acceleration of lead vehicle at time $t+R$, in ft/s^2
$s_l(t+R)$ = position of lead vehicle at time $t+R$ as measured from upstream, in ft
$s_f(t+R)$ = position of follower vehicle at time $t+R$ as measured from upstream, in ft
$v_f(t+R)$ = speed of follower vehicle at time $t+R$, in ft/s
$v_l(t+R)$ = speed of lead vehicle at time $t+R$, in ft/s
L_l = length of lead vehicle plus a buffer based on jam density, in ft
h = time headway parameter (refers to headway between rear bumper plus a buffer of lead vehicle to front bumper of follower), in seconds
T = simulation time-scan interval, in seconds
T = current simulation time step, in seconds
R = perception-reaction time, in seconds
K = sensitivity parameter (unit less)

For application to this project, the value of the L parameter varied based on one of the four different vehicle types. The time headway parameter (h) was set up as a random variable, rather than a constant value, to introduce an additional stochastic element to the model. Its value was based on a normal distribution to represent the more realistic scenario that desired headways vary by driver. The mean and standard deviation for this distribution could be specified for each of the four vehicle types. Thus, desired headways can vary by driver, as well as by vehicle category.

Additional details on the Modified Pitt car-following model can be found in Cohen [24].

Program Development

The simulation program was written in Visual Basic 6. A screen capture of the main user interface is shown in Figure 3-16.

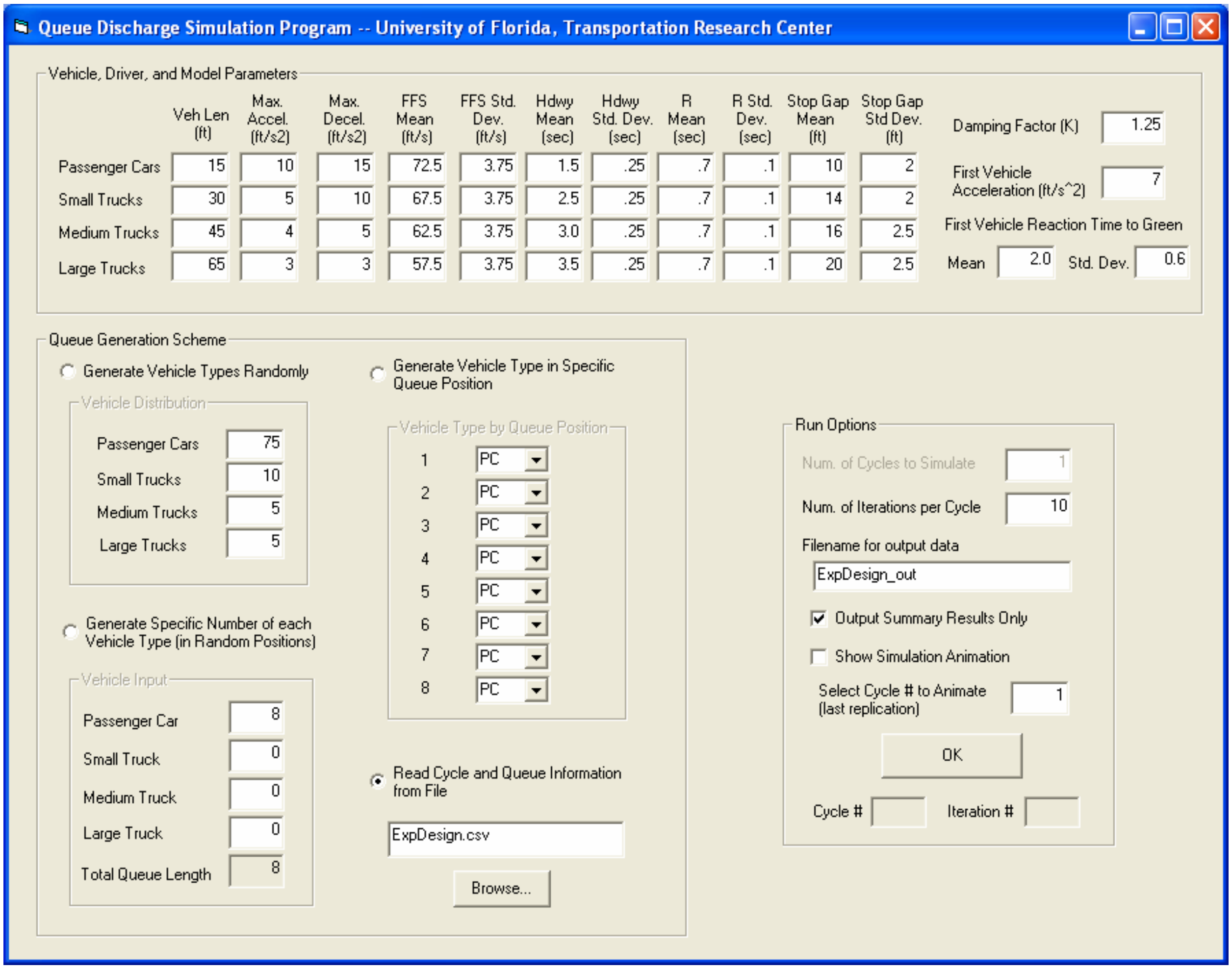


Figure 3-16. Simulation program user interface

The top of the user interface is where all of the vehicle characteristics and model parameters are specified. The lower part of the user interface is where the simulation run options were specified. Four options can be specified:

1. Generate a queue of vehicles randomly, based on the specified vehicle proportions (the queue length is a constant 8 vehicles).
2. Generate a specific number of each vehicle type in the queue, in random positions.
3. Generate a specific vehicle type in a specific queue position, for each of 8 total queue positions.
4. Reading any number of pre-specified queue configurations from an input file.

The first three options were used primarily for model testing purposes. The fourth option was used to facilitate the running of a large number of pre-specified simulation scenarios according to the experimental design, described in a later section.

The program includes a traffic animation component. This component animates the vehicle trajectory information recorded during the simulation process. It updates the position, speed, and acceleration of each vehicle in the queue every tenth of a second. This screen includes the signal status and the elapsed time from the beginning of the green. Additionally, it includes a table that displays key vehicle properties during each time step of the animation, which is used primarily for diagnostic purposes. Screen captures of this animation screen are shown in Appendix C.

Experimental Design

As mentioned previously, the main purpose of the field data was for calibration of the simulation program. The simulation program was used to generate the data for the development of the passenger car equivalent values for trucks, as well as revised lost time estimates. One obvious limitation with the field data is that a limited number of vehicle type-queue position combinations were observed, and of the combinations that were observed, some were observed a very limited number of times, some only once. With the use of simulation, a wide variety queue combinations (i.e., different vehicle types in different queue positions) can be generated, as well as any number of replications of a specific queue combination.

One of the first decisions to make regarding the experimental design is which variables to include. The study on saturation flow rate by Bonneson [8] identified several factors that affect this value, such as speed limit, number of lanes, and traffic pressure. As discussed in Chapter 4, none of these factors were found to be significant, or at least were inconclusive, in the field data. Thus, none of these factors were included in the experimental design. Some of these variables are still inputs to the simulation program, such as speed limit, but these were fixed at one "average" value. Furthermore, given the anticipated size of the simulation analysis data set, it was felt that simulation and analysis resources would be better spent focusing on just the truck-specific aspects of the queue discharge mechanism, which was the primary concern for this project. With this approach, any revised PCE values resulting from this study could be used in combination with the other factors developed as part of the Bonneson study. While the Bonneson study did identify a single revised truck PCE value, the authors admit that the examination of the effects of trucks on capacity was not a variable of primary concern; thus, truck percentages were quite low in the field data they collected for their project. In fact, one of the specific recommendations from that study was to further investigate truck PCE values.

Therefore, the experimental design consisted of just varying the vehicle type by queue position, for a fixed queue length, and leaving other factors fixed at representative values. Note that certain factors in the simulation program, such as maximum acceleration, are varied randomly by vehicle/driver for each queue generation, according to a mean value and standard deviation. However, these factors are part of the stochastic simulation process, and not factors to be estimated as part of the analysis process.

In working within the framework of the HCM guidance for measuring lost time and saturation flow rate, the queue length for the experimental design was set at eight vehicles for each combination. Thus, with the four different vehicle types, and a queue length of eight vehicles, the number of possible combinations is 65,536 (4^8). In order to reduce the computational burden due to such a large number of combinations, and to better reflect reality with the specific combinations, the number of combinations was reduced. Among the 65,536 possible combinations for four vehicle types and eight queue positions are many combinations that include a high percentage of trucks in the queue. Since queues with a high percentage of trucks were extremely rare in the field, it was decided to eliminate all queue combinations that consisted of a truck percentage of more than 50%. Of all the field data queues, very few queues had more than three trucks (out of a queue length of eight vehicles), and only one queue had five trucks. No queues had more than 5 trucks (out of eight). Elimination of all queue combinations with more than four trucks resulted in a total of 7,459 combinations, which comprised the final experimental design.

CHAPTER 4
ANALYSIS OF FIELD DATA AND CALIBRATION OF SIMULATION MODEL

This chapter describes the reduction and analysis of the field data, as well as the development of a model to fit the traffic flow data obtained from the field. It also describes in detail the calibration of the simulation program using the collected field data

Summary of Data Reduction

From the reduced field data, only signal cycles that had queues with at least 8 vehicles were retained for data analysis. A summary of this data set is included in Appendix D. This table shows the different queue compositions observed in the field along with their respective frequencies. Note that for notational convenience, vehicle types are expressed in a numbered format, where 1 is a passenger car, 2 is a small truck, 3 is a medium truck and 4 is a large truck.

This data set consisted of a total of 403 cycles, where 174 cycles were passenger cars only, 126 cycles had 1 truck, 68 cycles had 2 trucks, 28 cycles had 3 trucks and 7 cycles had 4 trucks in queue. Some of these queue compositions per cycle were repeated; therefore there were a total of 110 different observations. In addition, only one queue was observed to have 5 trucks but it was left out of the calibration process since the experimental design was constrained to no more than 4 trucks in queue. This will be explained in more detail later in this chapter.

In chapter 3 it was discussed that the time headway value between successive vehicles is a function of both the leading vehicle and the trailing vehicle. A summary of the average time headway for the 16 possible vehicle pairs (for 4 vehicle types) is shown in Appendix E. In Table 4-1 is a summary of those tables for vehicles in queue positions 2-8 and 5-8. Headways of vehicles in position 1 are not included in these tables since only 231 out of 403 cycles included the start of green.

Table 4-1. Average headway and frequencies for each leader-follower combination

Average headways of vehicle pairs in positions 2 through 8																
Trail vehicle[1]	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Lead vehicle[1]	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Frequency	417	51	37	61	45	8	5	7	36	4	4	1	65	6	7	16
Mean	2.40	3.13	3.34	4.70	3.01	3.85	4.92	5.61	3.67	4.86	5.70	4.24	4.14	4.42	4.97	5.09

Average headways of vehicle pairs in positions 5 through 8																
Trail vehicle[1]	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Lead vehicle[1]	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Frequency	232	29	21	38	28	3	4	3	21	2	3	1	39	2	3	11
Mean	2.19	2.82	2.72	4.13	2.86	3.08	4.88	4.22	3.74	5.61	4.59	4.24	4.13	4.50	4.46	5.23

[1] Vehicle Types: 1 = passenger car, 2 = small truck, 3 = medium truck, 4 = large truck

Time Headways

Time headways are often used to estimate the impact of trucks in the traffic stream, but to specifically calculate the saturation flow rate it is necessary to use the average headway of vehicles in positions 5 through 8 ($\bar{h}_{5-8}$) or also known as the saturation headway (h_{SAT}). It should be noted that this definition is consistent with the HCM, although some studies have found that the saturation headway is not achieved until the sixth or later vehicle in the queue. The concept of the saturation headway, consistent with the HCM definition, is illustrated mathematically with the following equations.

1. Saturation headway (i.e., the average headway for vehicles in positions 5-8)

$$h_{SAT} = \frac{(T_8 - T_4)}{4} \qquad [4\text{-}1]$$

2. Average headway for vehicles in positions 1-8

$$\bar{h}_{1-8} = \frac{T_8}{8} \qquad [4\text{-}2]$$

3. Average headway for vehicles in positions 2-8

$$\bar{h}_{2-8} = \frac{(T_8 - T_1)}{7} \qquad [4\text{-}3]$$

Where:

T_i = the time it takes for vehicle i to cross the stop bar

Table 4-2 summarizes the headways calculated from the field data.

Table 4-2. Time headways from field data

	All vehicles	All vehicles with start-up time	Passenger cars only	Passenger cars only with start-up time
h_{SAT}	2.87	2.83	2.18	2.23
h_{1-8}	N/A	2.98	N/A	2.48
h_{2-8}	3.11	N/A	2.36	N/A

Several models were specified in the statistical software package STATISTICA [25] to attempt to reasonably describe the impacts of trucks on queue discharge for these field data. These models considered queue composition, truck percentages, and site characteristics. The specifications were done using a non-linear regression analysis with a confidence level of 95%. A summary of this model development is described in the following paragraphs.

Model 1

The first model (Equation 4-4) used an alternate form of the f_{HV} equation (HCM chapter 16) to estimate the average saturated headway (h_{SAT}). This analysis used the data that did not contain start-up reaction time values.

$$h_{SAT} = h_{SPC} \times \left(1 + Pct_{ST} \times (b_1 - 1) + Pct_{MT} \times (b_2 - 1) + Pct_{LT} \times (b_3 - 1)\right) \qquad [4\text{-}4]$$

Where:

h_{SAT} = saturation headway
h_{SPC} = saturation headway of passenger cars in a passenger car only queue
Pct_{ST} = percentage of small trucks in queue
Pct_{MT} = percentage of medium trucks in queue
Pct_{LT} = percentage of large trucks in queue

In this model, the coefficients b_1, b_2 and b_3 represent the E_T values for small, medium and large trucks respectively. The estimated coefficient values are shown in Table 4-3. An R^2 value of 0.3591 was obtained for this model.

Table 4-3. Results from STATISTICA for model 1

Coefficients	Estimate	Standard error	t-value
h_{SPC}	2.216044	0.033277	66.59442
b_1	1.655451	0.165439	10.00640
b_2	1.771816	0.196516	9.01615
b_3	2.677482	0.132386	20.22480

Model 2

The second model (Equation 4-5) is the same as Model 1, but this model estimates the average headway for vehicles in position 2 through 8, where h_{PC} is the average headway for passenger cars. Since some of the data did not include the start of green time (thus the headway for vehicle 1 could not be determined), this analysis was based on a larger data set. The estimated coefficient values are shown in Table 4-4. An R2 of 0.5923 was obtained for this model.

$$\overline{h}_{2-8} = h_{PC} \times \left(1 + Pct_{ST} \times (b_1 - 1) + Pct_{MT} \times (b_2 - 1) + Pct_{LT} \times (b_3 - 1)\right) \qquad [4\text{-}5]$$

Table 4-4. Results from STATISTICA for model 2

Coefficients	Estimate	Standard error	t-value
h_{PC}	2.381300	0.021310	111.7433
b_1	1.765421	0.099050	17.8236
b_2	2.062019	0.118059	17.4660
b_3	2.508828	0.077991	32.1682

Model 3

The third model (Equation 4-6) has the same characteristics as *Model 1* but using only the data that contain the start-up time. The estimated coefficient values are shown in Table 4-5. An R^2 of 0.3798 was obtained for this model.

$$h_{SAT} = h_{SPC} \times \left(1 + Pct_{ST} \times (b_1 - 1) + PctM_{ST} \times (b_2 - 1) + Pct_{LT} \times (b_3 - 1)\right) \qquad [4\text{-}6]$$

Table 4-5. Results from STATISTICA for model 3

Coefficients	Estimate	Standard error	t-value
h_{SPC}	2.204078	0.040422	54.52684
b_1	1.705201	0.189231	9.01121
b_2	2.148228	0.217222	9.88953
b_3	2.639080	0.174277	15.14304

Model 4

The fourth model (Equation 4-7) has the same characteristics as *Model 2* but using only the data that contain the start-up time. The estimated coefficient values are shown in Table 4-6. An R^2 of 0.5309 was obtained for this model.

$$\overline{h}_{1-8} = h_{PC} \times \left(1 + PctST \times (b_1 - 1) + PctMT \times (b_2 - 1) + PctLT \times (b_3 - 1)\right) \qquad [4\text{-}7]$$

Table 4-6. Results from STATISTICA for model 4

Coefficients	Estimate	Standard error	t-value
h_{PC}	2.479479	0.024889	99.61976
b_1	1.414669	0.102169	13.84634
b_2	1.882030	0.117720	15.98733
b_3	2.239560	0.093155	24.04133

These models resulted to have a low value for R^2 and it was due to the variance inherent in the data. This could be due to insufficient field data. In addition, there were specified several other models that included additional variables such posted speed limit, area type, and number of lanes. These models were not found to be statistically significant.

Start-up Reaction Time (SRT)

The Start-up Reaction Time (SRT) is the time from start of green until the first vehicle begins to move. This measurement is sensitive to the timing precision of the video frame rate (0.0333 frames/sec). If vehicles in the first position had their front axle on the stop bar, their headways were set equal to the SRT. Cycles that had vehicles in the first position with their front axle beyond the stop bar were dropped out of the data set.

These times were obtained from the videos and were measured only from site 1. Site 1 has two through lanes, but these data were collected during a time when one through lane was closed. After careful observation and comparison between the queue discharge at this site for two through lanes and one through lane, it was concluded that this particular lane closure set-up did not have any adverse impact on start-up reaction times or queue discharge rates. Right turning vehicles were not included in these measurements.

A frequency graph of the field-measured SRT values measured is shown in Figure 4-1.

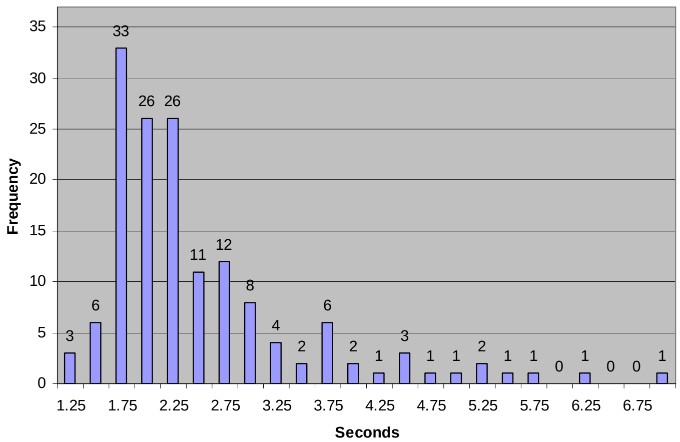


Figure 4-1. Start-up reaction time frequencies

Based on the observations, it was decided that a maximum of 3.5 seconds can be considered as a realistic maximum start-up reaction time. SRTs greater than 3.5 seconds were typically due to an obvious hesitation (related to distraction or similar) of the drivers. Therefore, SRT values above 3.5 seconds were trimmed from the data set. Figure 4-2 shows a frequency distribution for only those drivers with a SRT of 3.5 seconds or less.

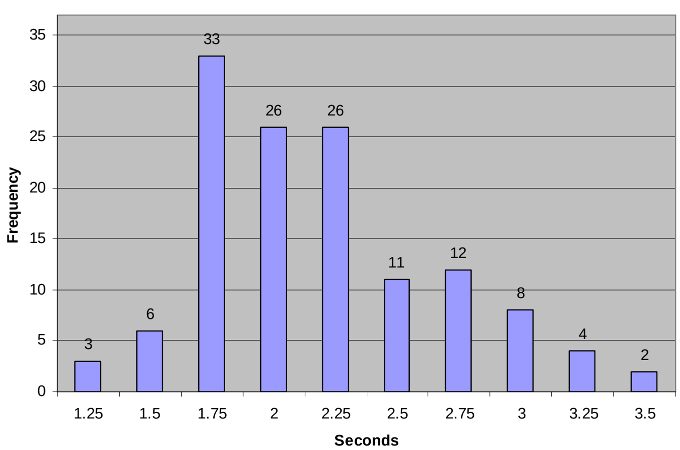


Figure 4-2. Start-up reaction time frequencies for trimmed data set

Using this trimmed data set with 134 observations, a mean start-up reaction time of 2.04 seconds and a standard deviation of 0.47 were calculated. This mean value was somewhat consistent with previous studies reported in the literature. Kockelman [6] reported a mean SRT value of 1.79 sec and Li and Prevedouros [10] reported a mean SRT value of 1.76 and standard deviation of 0.61. They also concluded that these values were normally distributed.

For simulation purposes, it was assumed that the first vehicle started at the stop bar and therefore the headway of the first vehicle in queue was equal to the SRT regardless of vehicle type. From field data and other studies it was decided to the set SRT equal 2.0.

Start-up Lost Time (SLT)

The Start-up lost time is the difference between elapsed time for first four vehicles (1-4) to cross the stop bar and the time for the last four vehicles (5-8) in queue assuming passenger car only queue. The SLT can be calculated as it is shown in Equation 4-8.

$$SLT = TT_{1-4} - \left(h_{SAT} \times 4\right) \quad [4\text{-}8]$$

Where:

TT_{1-4} = total time required for first four vehicles in queue to cross the stop bar

h_{SAT} = average saturation headway for a passenger cars only queue

The field data were examined to obtain some sense of whether or not trucks are more or less likely to be in the first four positions of the queue. Table 4-7 summarizes these data.

Table 4-7. Distribution of trucks by site and position in queue

	Site indicator									
	1	2	3	4	5	6-1	6-2	6-3	6-4	Total
Total cycles	50	11	24	17	22	52	66	18	143	403
Cycles with trucks	41	11	8	14	20	20	25	9	81	229
Cycles with trucks up-front*	6	1	3	8	11	9	10	7	40	95
% of trucks up-front*	15%	9%	38%	57%	55%	45%	40%	78%	49%	41%

*up-front = in the first four positions

Based on the field collected data, it was not possible to draw a definitive conclusion about whether trucks are more or less likely to be at the front of the queue. Thus, for the purposes of this study, it was assumed that trucks were randomly distributed throughout the entire queue, for each cycle. Additionally, it was observed that the Start-up Lost Time increased proportionally with an increase in overall truck percentage.

Calibration of Simulation Model

As mentioned before, for simulation to be effective, it is necessary to calibrate a simulation program against field data. An extensive calibration process was performed in order to identify the parameter values that resulted in the simulation data providing the best match with the field data. The quantitative measure used in this process was the Mean Square Error (MSE). The MSE is commonly used to compare a predicted value with an observed value. In this case, it was used to compare the observed average headway from the field data with the average headway estimated from the simulation data. Equation 4-9 shows the mathematical form for MSE.

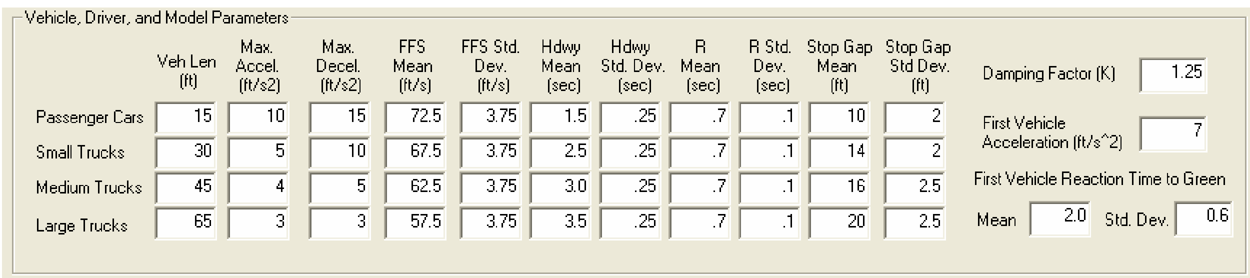
$$MSE = \frac{\sum \left(\bar{h}_{Field} - \bar{h}_{Simul}\right)^2}{N} \qquad [4\text{-}9]$$

Where:

MSE = Mean Square Error
$\bar{h}_{Field}$ = average headway from the field data
$\bar{h}_{Simul}$ = average headway from the simulation data
N = total number of observations

The calibration process consisted of comparing valid field data cycles with simulation cycles with the same configuration. Valid cycles were those cycles with at least 8 vehicles in queue and that showed a normal behavior. Cycles with implicit hesitation of the drivers, lane changing, first vehicle beyond the stop bar, motorcycles and right turns were considered as not valid or non-normal behavior.

In this process there were a couple of parameters that remained fixed. The simulation time-scan interval (*T*) is not a direct calibration parameter and was not changed throughout the process. Also parameters as R and K were fixed at 0.7 and 1.25 respectively. These values were recommended by Cohen [11]. The vehicles' lengths were fixed using values that were based on observations from this study's field data, and were also consistent with values recommended by Long [22]. The rest of the parameters were adjusted during the process. A screen capture of the program with the final set of parameters is shown in Figure 4-3.


Vehicle, Driver, and Model Parameters

	Veh Len (ft)	Max. Accel. (ft/s2)	Max. Decel. (ft/s2)	FFS Mean (ft/s)	FFS Std. Dev. (ft/s)	Hdwy Mean (sec)	Hdwy Std. Dev. (sec)	R Mean (sec)	R Std. Dev. (sec)	Stop Gap Mean (ft)	Stop Gap Std Dev. (ft)
Passenger Cars	15	10	15	72.5	3.75	1.5	.25	.7	.1	10	2
Small Trucks	30	5	10	67.5	3.75	2.5	.25	.7	.1	14	2
Medium Trucks	45	4	5	62.5	3.75	3.0	.25	.7	.1	16	2.5
Large Trucks	65	3	3	57.5	3.75	3.5	.25	.7	.1	20	2.5

Damping Factor (K) 1.25
First Vehicle Acceleration (ft/s^2) 7
First Vehicle Reaction Time to Green
Mean 2.0 Std. Dev. 0.6

Figure 4-3. User-adjustable vehicle, driver, and model parameters

In the process of calibration and trying to achieve a minimum MSE value, it was necessary to keep in mind that the final values of the parameters should stay within a reasonable range. For example, if a minimum MSE value is obtained by using a passenger car free flow speed of 30 ft/sec (~20 mi/h), it is not realistic to say that a passenger car will cruise at such a low speed.

Process of Calibration

An input file was created and read into the program to facilitate the process. This file contained each queue composition observed in the field. A first attempt included ten replication runs of each queue scenario. The variance within the runs was too high and the required sample size was on the order of 60 replications, for that reason the number of replications was increased to 100.

During the process of calibration the parameters that were primarily experimented with were the lead vehicle's acceleration and the maximum acceleration, free-flow speed, minimum desired headway and inter-vehicle spacing per vehicle type. A total of 99 different parameter combinations were run in an effort to obtain the lowest MSE. In those iterations, the value of the MSE fluctuated from 0.097 to 0.415 for the average headway of vehicles 2 through 8 and from 0.142 to 0.761 for the average saturation headway. The final set of parameters had 0.118 and 0.142 for the average headway of vehicles 2 through 8 and the average saturation headway respectively. The final set of parameters is shown in Table 4-8.

Table 4-8. Final choice of calibration parameter values

	Vehicle length (ft)	Maximum acceleration (ft/s^2)	FFS (ft/s)	FFS std. dev. (ft/s)	Headway (sec)	Headway std. dev. (sec)	Stop gap (ft)	Stop gap std. dev. (ft)
PC	15	10	72.50	3.75	1.50	0.25	10	2.0
ST	30	5	67.50	3.75	2.50	0.25	14	2.0
MT	45	4	62.50	3.75	3.00	0.25	16	2.5
LT	65	3	57.50	3.75	3.50	0.25	20	2.5

Execution of Experimental Design

Once the experimental design was complete, an input file was generated with the 7459 queue combinations. The simulation program was run with this file performing 100 replications of each queue combination. Based on some testing, it was decided to perform 100 replications to account for the considerable variance inherent in this process. Since there was considerable variance in the field data, the calibration process of the simulation still allows for a large amount of variance in each individual simulation run. The choice of 100 replications was somewhat conservative, but not extremely conservative.

An average of each of the 100 replications was calculated and used in the analysis database. This new data set was inputted in the statistical software package STATISTICA. Subsequently, variables were created as needed to run the models. These variables include, but are not limited to, indicator variables for each vehicle type per position, indicator variables for vehicle pairs per position, frequencies and percentages of each one of theses scenarios.

Additional experiments were run to verify certain numerical results, such as the following.

- 10,000 passenger car-only queues to verify the base saturation headway
- 400 passenger car-only queues, changing the vehicle in position 4 to estimate the effect of different truck types on the headway of the vehicle in position 5.
- 256 cases with 100 replications of trucks in positions 1-4 and *PCs* in positions 5-8 to isolate the effect of trucks on the start-up lost time section of the queue.
- A data set with fabricated fixed values for each vehicle pair, regardless of their queue position, to verify model formulations.

CHAPTER 5
ANALYSIS OF SIMULATION DATA

This chapter describes how the data generated from the simulation experimental design process were used to obtain the new PCE factors. For the purposes of this research, the queue was analyzed in sections that were consistent with the guidance given in the HCM 2000 with regard to the components of start-up lost time and saturation headway. That is, the first four headways are included in the start-up lost time, and the headways of the remaining vehicles represent the saturation headway. Thus, the queue was subdivided into two sections for purposes of the analysis. Figure 5-1 illustrates these two queue sections.

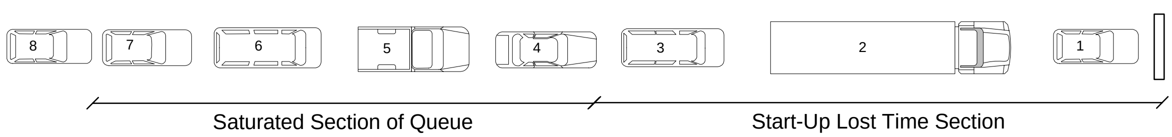


Figure 5-1. Queue sections for analysis

In reality, the lost time does not necessarily end with the headway of the fourth vehicle, but for purposes of this research it was decided to be consistent with the HCM framework. Furthermore, the field data collected in this study were not conclusive with regard to this issue.

As previously mentioned, the analyses were done assuming trucks were distributed randomly within each queue. That is, for any given cycle, a given number of trucks may appear at any location throughout the queue. In some queues, a given number of trucks may be at the front of the queue, in some queues this same number may be in the middle of the queue, and so on. But over the length of the analysis period, for a fixed percentage of trucks in the traffic stream, it is assumed that any position within the queue is as likely to have a truck in that position as any other position in the queue.

The first approach to find the PCE values was to analyze the saturated part of the queue, as it is recommended in the HCM. A simple model using only the percentages of each type of truck in positions 5 through 8 was specified. The results of this model did not reflect exactly the behavior of the queue. This was expected since this model does not explicitly account for the headways being a function of the specific leader-follower combination. Subsequently, another model was specified using the frequency of each vehicle pair in the second half of the queue. This approach assumed that each vehicle pair had the same headway regardless of their position in the queue. The estimates from this model did not replicate the performance of the queue as closely as hoped. To confirm that the model was specified correctly, a different program was developed to generate fixed headway values that were independent of queue position. When the same model was run on this data set, it provided a better fit, but still not exact. Thus the next step was to incorporate indicator variables for each one of the 16 different vehicle pair combinations, for positions 5 through 8. This model demonstrated that each vehicle pair has a different headway value depending of their position in queue.

It was also noted that the type of vehicle in position 4 had a significant impact on the calculation of saturation headway, as expected. Furthermore, it was also observed that trucks present in the first few positions of the queue also influence the saturation headway. Figure 5-2 shows a diagram with three different scenarios reflecting these impacts.

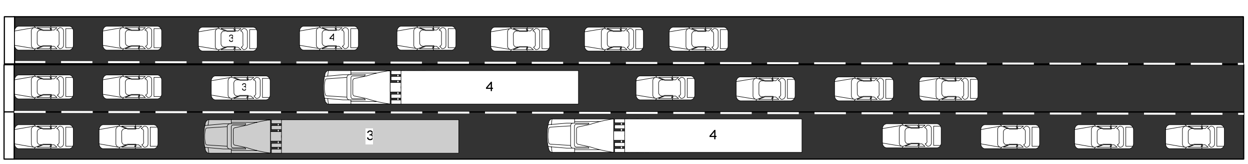


Figure 5-2. Impact of trucks in the first part of the queue to the saturation headway

The first case (on top) has only passenger cars, the second case (middle) has a large truck in position 4, and the third case (bottom) has large trucks in positions 3 and 4. Although the second part of the queue was the same for all the cases (passenger cars in positions 5-8), the saturation headways were significantly different. The saturation headways for these three scenarios were 2.033, 2.795 and 2.641 seconds respectively. The only difference between the first and second cases is the presence of a large truck in the fourth position. The size of this vehicle impacts directly the headway of the vehicle in position 5. In addition, large trucks have a lower acceleration rate when compared to passenger cars, which affect the acceleration of the vehicles behind. In this case, vehicles in positions 5 through 8 were directly impacted by the truck. These vehicles were constrained to the acceleration of the large truck since lane changing was not allowed. This comparison confirms that the saturation headway is affected by the vehicle in position 4.

Moreover, it was observed how the impact on saturation headway due to the number of trucks in the first few queue positions might be overestimated. The impact on saturation headway is just as much a function of the position of the trucks in the queue as it is the total number of trucks in the first four positions of the queue. These data showed that the queue with two large trucks had a smaller saturation headway than the one with only one truck. This confirms that not only the vehicle type in position 4 impacts the saturation headway, but also the presence of other trucks preceding the truck in position 4 provide an additional impact. In this example, the difference results from the length of the trucks rather than their acceleration capabilities. Vehicles in positions 5 through 8 were constrained to the acceleration rate of the large trucks, but in the third case the vehicles were farther back in the queue. The distance to the stop bar of each vehicle was larger and with the same acceleration the vehicles reached higher speeds prior to crossing the stop bar. The spacing was very similar between vehicles in the two cases, so with a higher speed and similar spacing, the result is lower headways. This result is consistent with some studies that have claimed that the headways between vehicles at the back of a long queue will be lower than those near the front because the vehicles near the back have more time accelerate to a higher speed. However, the Li and Prevedouros [10] study found that this is not always the case because headway expansion may occur toward the end of long queues

In light of these findings, it was desired to test a model that included indicator variables for each one of the 16 different vehicle-pair combinations for each queue position. However, this results in a model with an extremely large number of parameters, which was beyond the capabilities of the statistical software package. Therefore, it was decided to treat each part of the queue separately. A new data set was created with only passenger cars in the second part of the queue (positions 5-8). This data set included 256 (4^4) different queue combinations, as it was only the first four positions of the queue that varied by vehicle type. This new data set was used to estimate the impact of trucks to the SLT.

The more accurate method to analyze the impact of trucks to the SLT is to use indicator variables for the 16 different vehicle-pair combinations in each one of the first four positions. But again, this approach was not practical. Therefore, since the simulation program provides the same headway value for the first vehicle regardless of what type it is, it was decided to run a narrower model with only indicator variables (for the 16 combinations) for the first two vehicles. This model isolates the impact of the first vehicle in queue. The headway of the first vehicle was the same for each vehicle type, but the effect to the trailing vehicle due to the length and acceleration of the first vehicle varied. After the results from this model were obtained, this model was included in another model that included indicator variables of vehicle types for in positions 2 through 4. This model is shown in Equation 5-1.

$$TT_{1-4} = TT_{1-2} + TT_{PC3-4} + \sum_{k=2}^{4}\left(b_{2k} \times ST + b_{3k} \times MT + b_{4k} \times LT\right) \qquad [5\text{-}1]$$

Where:

$TT_{1\text{-}4}$ = clearance time for the first four vehicles in queue
$TT_{1\text{-}2}$ = clearance time for the first two vehicles in queue
$TT_{PC3\text{-}4}$ = clearance time for passenger cars in positions 3 and 4
b_{ik} = additional time for vehicle type i in position k to clear
ST = indicator variable for vehicle type small truck (1-yes, 0-no)
MT = indicator variable for vehicle type medium truck (1-yes, 0-no)
LT = indicator variable for vehicle type large truck (1-yes, 0-no)
k = indicator for queue position
i = ST for small truck, MT for medium truck, LT for large truck

And the clearance time for the first two vehicles was defined as is shown in Equation 5-2.

$$TT_{1-2} = 2.0 + h_{PC-PC} + \sum_{i,j=1}^{4} \left(b_{ij} \times I_{ij}\right) \qquad [5\text{-}2]$$

Where:

$TT_{1\text{-}2}$ = clearance time for the first two vehicles in queue
$h_{PC\text{-}PC}$ = headway of a passenger car in position 2 following a passenger car
b_{ij} = additional time for vehicle i following vehicle type j to clear
I_{ij} = indicator for vehicle type i following vehicle type j
i = indicator for vehicle type
j = indicator for vehicle type

The final form of the equation to estimate the total time to clear the first four vehicles in queue (Equation 5-1) is shown in Equation 5-3.

$$TT_{1\text{-}4} = 10.59 + \begin{bmatrix} 1.62 \times b_{212} + 3.00 \times b_{213} + 5.04 \times b_{214} \\ + 1.38 \times b_{221} + 3.09 \times b_{222} + 4.53 \times b_{223} \\ + 6.65 \times b_{224} + 2.47 \times b_{231} + 4.19 \times b_{232} \\ + 5.65 \times b_{233} + 7.80 \times b_{234} + 4.22 \times b_{241} \\ + 5.96 \times b_{242} + 7.36 \times b_{243} + 9.47 \times b_{244} \end{bmatrix} + \begin{bmatrix} 1.47 \times b_{32} + 2.49 \times b_{33} \\ + 4.03 \times b_{34} + 0.83 \times b_{42} \\ + 1.21 \times b_{43} + 1.72 \times b_{44} \end{bmatrix} \qquad [5\text{-}3]$$

Where the total time to clear the first four vehicles, if are passenger cars, is 10.59 seconds and the b_{kij} and b_{ij} follow the format described earlier in this chapter.

From Equation 4-8, the SLT can be calculated as the difference of this time to clear the first four vehicles and the equivalent of four times the saturation headway of passenger cars. The SLT calculated for passenger cars only queue is equal to 2.47 seconds, which is higher than the

2.0 seconds recommended in the HCM 2000 [1], but somewhat consistent with the findings of the field data.

It is also important to account for the impact of trucks to the SLT since the presence of these vehicles increased significantly the SLT. In example, if the composition of the queue is as it is shown in Figure 5-3.

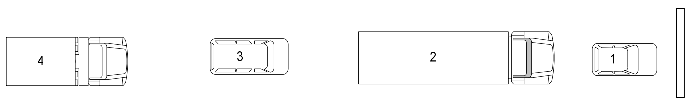


Figure 5-3. SLT example

where a large truck is in the second position, a medium truck in the fourth position, and the rest are passenger cars, the SLT can be estimated as indicated in Equation 5-4.

$$SLT = \left[10.59 + \left(4.22 \times b_{241}\right) + \left(1.72 \times b_{44}\right)\right] - \left(4 \times 2.03\right) \qquad [5\text{-}4]$$

Then, the SLT for this case of mixed traffic is 8.41 seconds. This value is considerably higher when compared to the 2.47 seconds for passenger cars only queue and it needs to be treated separately from the saturation headway.

Since it is not realistic to expect the practitioner to collect every vehicle type in queue and their exact position, an equation was developed to estimate the SLT based on the percentage of trucks in the queue. As discussed earlier, it is assumed that the trucks are distributed randomly throughout the queue. Therefore, over many cycles with trucks distributed randomly within the queue for each cycle, it can be assumed that on average, trucks will be distributed evenly throughout the queue. The SLT equation is given by,

$$SLT = 2.5 + 5.0 \times Pct_{ST} \times 9.0 \times Pct_{MT} + 15.0 \times Pct_{LT} \qquad [5\text{-}5]$$

Where:

Pct_i = percentage of truck type i

After analyzing the two different sections of queue, a compound model was developed. This model was used to obtain an estimate of the total time elapsed for clearing a queue length of eight vehicles, using the experimental design data set. A general form of this model is shown in Equation 5-6.

$$TT_{1-8} = TT_{1-4} + 4 \times h_{PC-PC} + \sum (b_{ij} \times F_{ij}) \qquad [5\text{-}6]$$

Where:

TT_{1-8} = clearance time for the first eight vehicles in queue
TT_{1-4} = clearance time for the first four vehicles in queue
h_{PC-PC} = headway of passenger car following a passenger car
b_{ij} = additional headway for vehicle pair (vehicle *i* following vehicle *j*)
F_{ij} = frequency of vehicle pair (vehicle *i* following vehicle *j*) in positions 5 through 8

The model estimation output is shown in Table 5-1 where b_0 is h_{PC-PC} and the remaining betas follow the "b_{ij}" format described before.

Table 5-1. Model estimation results for additional headways of 16 vehicle pair combinations

Coefficients	Estimate	Std. error	t-value
h_{PC-PC}	2.028586	0.004183	484.9803
b_{12}	0.590274	0.012212	48.3346
b_{13}	1.046166	0.012212	85.6654
b_{14}	1.852489	0.012212	151.6913
b_{21}	1.024162	0.012212	83.8636
b_{22}	1.531207	0.016902	90.5956
b_{23}	2.035979	0.020033	101.6312
b_{24}	2.993811	0.020033	149.4440
b_{31}	1.407661	0.012212	115.2665
b_{32}	1.920917	0.020033	95.8877
b_{33}	2.393117	0.016902	141.5915
b_{34}	3.377253	0.020033	168.5845
b_{41}	1.823759	0.012212	149.3388
b_{42}	2.426886	0.020033	121.1444
b_{43}	2.835042	0.020033	141.5186
b_{44}	3.573751	0.016902	211.4451

These estimates were substituted in Equation 5-6 and the final form of the model is defined in Equation 5-7.

$$TT_{1-8} = TT_{1-4} + 4 \times 2.029 + \begin{Bmatrix} 0.590 \times F_{12} + 1.046 \times \mathrm{F}_{13} + 1.852 \times \mathrm{F}_{14} + 1.024 \times \mathrm{F}_{21} \\ + 1.531 \times \mathrm{F}_{22} + 2.036 \times \mathrm{F}_{23} + 2.994 \times \mathrm{F}_{24} + 1.407 \times \mathrm{F}_{31} \\ + 1.921 \times \mathrm{F}_{32} + 2.393 \times \mathrm{F}_{33} + 3.377 \times \mathrm{F}_{34} + 1.824 \times \mathrm{F}_{41} \\ + 2.427 \times \mathrm{F}_{42} + 2.835 \times \mathrm{F}_{43} + 3.574 \times \mathrm{F}_{44} \end{Bmatrix} \quad [5\text{-}7]$$

Figure 5-4 shows a plot of this equation against the observed simulation data.

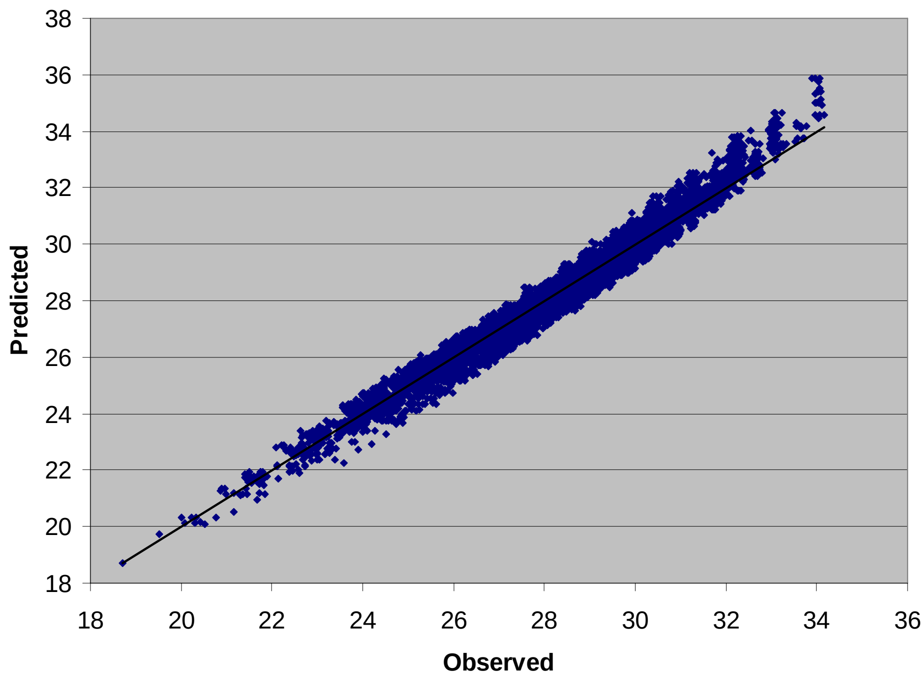


Figure 5-4. Total time for vehicles 1-8 using vehicle pairs in positions 5-8

This model provides the headway for a passenger car following a passenger car ($h_{PC\text{-}PC}$) in positions 5 through 8 and also the additional time headway taken by any other vehicle pair in these positions. Therefore, it was possible to calculate the headway for each one of these pairs. Table 5-2 shows these headways. In this table the PCE value for each vehicle pair is also shown. These PCE values were calculated as a relative headway to the $h_{PC\text{-}PC}$ per the definition in the HCM 2000.

Table 5-2. Headways and PCE values for 16 vehicle pair combinations

Vehicle pair trailing→leading	h_{PC-PC} (seconds)	Additional headway* (seconds)	Headway (seconds)	PCE values of each 16 vehicle-pair
PC → PC	**2.029**		**2.029**	**1.000**
PC → ST		**0.590**	**2.619**	**1.291**
PC → MT		**1.046**	**3.075**	**1.516**
PC → LT		**1.852**	**3.881**	**1.913**
ST → PC		**1.024**	**3.053**	**1.505**
ST → ST		**1.531**	**3.560**	**1.755**
ST → MT		**2.036**	**4.065**	**2.004**
ST → LT		**2.994**	**5.022**	**2.476**
MT → PC		**1.408**	**3.436**	**1.694**
MT → ST		**1.921**	**3.950**	**1.947**
MT → MT		**2.393**	**4.422**	**2.180**
MT → LT		**3.377**	**5.406**	**2.665**
LT → PC		**1.824**	**3.852**	**1.899**
LT → ST		**2.427**	**4.455**	**2.196**
LT → MT		**2.835**	**4.864**	**2.398**
LT → LT		**3.574**	**5.602**	**2.762**

$*\Delta h = (Veh_i \rightarrow Veh_j)\text{-}(PC \rightarrow PC)$

Although these PCE values reflected more accurately the behavior of the queue, it is not practical to have a PCE value for each possible vehicle-pair combination. In an effort to simplify this model, a new set of PCE factors was derived for each one of the truck vehicle types. This new set takes into consideration their headway and the time each vehicle type adds to the following vehicle's headway. This concept can be described as the *time consumed* by any vehicle type in queue. Figure 5-5 illustrates this concept with an example of a Large Truck in queue. This example compares the headway of a *PC* following a *LT* with the headway of a *PC* following a *PC*.

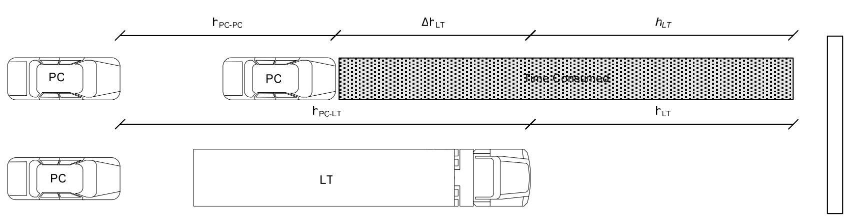


Figure 5-5. Time consumed by a large truck

As shown, a LT adds an additional headway (Δh) to the *PC*. In this case the additional time can be defined as it is shown in Equation 5-8.

$$\Delta h_{LT} = h_{PC \to LT} - h_{PC \to PC} \qquad [5\text{-}8]$$

From the values in Table 5-2, 1.852 seconds is added to the headway of the passenger car when following a large truck. This additional time should be considered part of the *LT* headway rather than the *PC* since this additional time it is not present in a *PC* only queue. Also shown is the *time consumed* by a *LT* in queue (dark shade) which was defined as the time headway of a LT plus the additional headway this vehicle type adds to the following vehicle.

The Δh of each vehicle type is not only added to the *PC*. These impacts are present regardless of what type of vehicle is following. Table 5-3 includes a summary of each vehicle type's headway when following a PC and the Δh they add to any other vehicle type.

Table 5-3. Vehicle type's headway and their Δh

Trailing	Leading Headway when following PC	Δh ST	Δh MT	Δh LT
PC	2.029	0.590	1.046	1.853
ST	3.053	0.507	1.012	1.970
MT	3.436	0.513	0.986	1.970
LT	3.852	0.603	1.011	1.750
Average Δh		0.553	1.014	1.885

In order to develop a model that does not take into consideration the position of each vehicle in queue, it was necessary to use the average Δh per vehicle type. By using the average of the Δh, the proportion of these vehicle pairs in queue are not being taken into consideration. To develop PCE values based on just three truck categories, rather than all 16 possible lead-follow vehicle pairings, the resulting PCE values will obviously be generalized. That is, with this approach, information about specific position in queue for each truck is not utilized. However, the reason, again, for this approach is that it is expected that practitioners will not collect data at this level of detail. With such a generalized approach, this loss of information requires some level of approximation. The implicit assumption is that each vehicle pair occurs with the same frequency in the traffic stream. While this may rarely be the case, the objective was to arrive at generalized PCE values that yielded an f_{HV} value that still tracked reasonably well with the f_{HV} value that results from using all 16 PCE values, for a varying range of overall truck percentage, as well as varying relative truck type percentages. This more general equation using the *time consumed* (H) by any vehicle i in queue is defined in Equation 5-9. Where vehicle j is following vehicle i.

$$H_i = h_{i \to PC} + \overline{\Delta h_i} \qquad [5\text{-}9]$$

Where:

$$\overline{\Delta h_i} = \frac{\sum_{i,j=1}^{4} \left(h_{j \to i} - h_{j \to PC} \right)}{4} \qquad [5\text{-}10]$$

And $1 = PC$, $2 = ST$, $3 = MT$ and $4 = LT$

Shown in Table 5-3 are the values for the *time consumed* in queue by each vehicle type. These values can be used to arrive at new PCE factors since this *time consumed* by each vehicle type can be considered as their time headway. Table 5-4 shows the PCE factors that were calculated as the relative time consumed per each vehicle type in queue when compared to a *PC*.

Table 5-4. Time consumed and PCE values for each vehicle type

Vehicle type	$h_{i \to PC}$ (seconds)	Avg. Δh_i (seconds)	H_i (seconds)	PCE_i
PC	2.029	0.000	2.029	1.000
ST	3.053	0.553	3.606	1.778
MT	3.436	1.014	4.450	2.194
LT	3.852	1.885	5.738	2.828

To assess the accuracy of these values, a model of total time needed to clear a queue with eight vehicles with only three types of trucks, instead of all vehicle-pair combinations, was tested. The model was specified in STATISTICA and the general form of this new model is shown in Equation 5-11.

$$TT_{1-8} = TT_{1-4} + 4 \times h_{PC5-8} + \left(b_{ST} \times F_{ST} + b_{MT} \times F_{MT} + b_{LT} \times F_{LT} \right) \qquad [5\text{-}11]$$

Where:

TT_{1-8} = time needed to clear the first eight vehicles in queue
TT_{1-4} = time needed to clear the first four vehicles in queue
h_{PC5-8}= headway of passenger cars following passenger cars in positions 5 through 8
b_i = additional headway for vehicle type i
F_i = frequency of truck type i in positions 5 through 8
i = ST for small trucks, MT for medium trucks, and LT for large trucks

The model estimation results are shown in Table 5-5, where b_0 represents the time of only PCs in positions 5 through 8 and the remaining betas follow the "b_i" format described before.

Table 5-5. Results from STATISTICA for model with vehicle types

Coefficients	Estimate	Std. error	t-value
h_{PC5-8}	2.270076	0.029382	309.0408
b_{ST}	1.228277	0.018823	65.2534
b_{MT}	1.952747	0.018823	103.7416
b_{LT}	2.985894	0.018823	158.6285

Figure 5-6 shows the plot of this equation against the observed data.

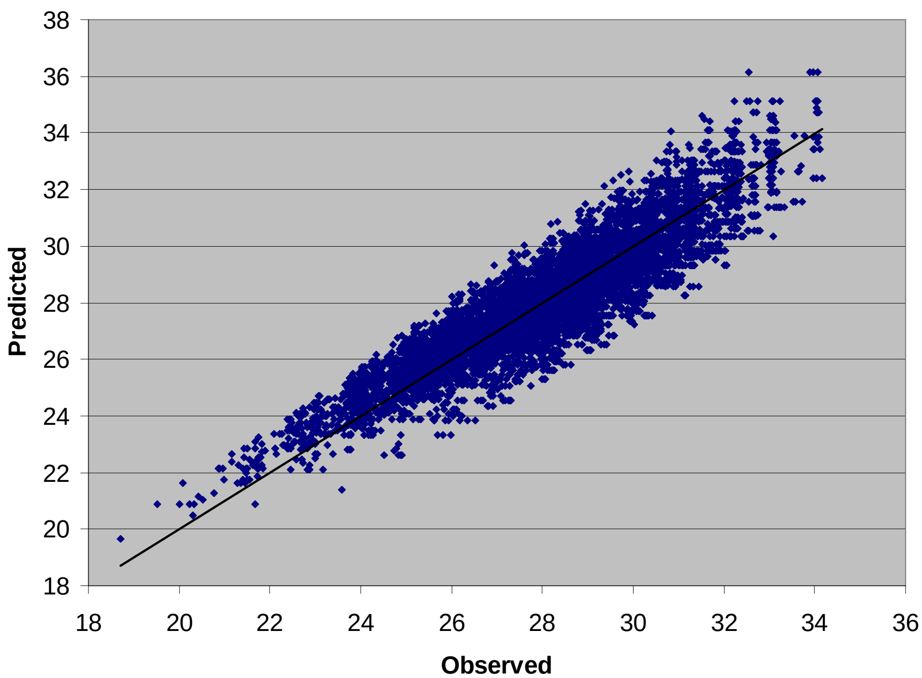


Figure 5-6. Total time for vehicles 1-8 using vehicle types in positions 5-8

This figure illustrates the increase in variance when compared to the plot based on all 16 vehicle pairs, as expected. Using just the three general categories of truck type instead of all 16 vehicle pairs obviously results in less precision of the saturation flow rate estimate. Table 5-6 shows the resulting headways and PCE values calculated from the results shown in Table 5-5.

Table 5-6. PCE factors for three vehicle types

Vehicle type	headway (seconds)	PCE
PC	2.270	1.000
ST	3.498	1.541
MT	4.223	1.860
LT	5.256	2.315

Note that these PCE values are considerably lower than the ones calculated from the other model shown in Table 5-4. These values demonstrate that the impact of the passenger cars is being overestimated and the impact of the trucks is being underestimated. As previously discussed, a truck in position four has a significant impact to the saturation headway, and this approach did not take that into consideration.

Another alternative was to use the alternate form of the f_{HV} equation from the HCM. The following derivation was made with the purpose of relating the current analysis to the HCM's approach. Starting with the model specified to estimate the total time needed to clear the first eight vehicles in queue Equation 5-11, and if $TT_{1-8} = TT_{1-4} + TT_{5-8}$ then,

$$TT_{5-8} = 4 \times h_{PC5-8} + \left(b_{ST} \times F_{ST} + b_{MT} \times F_{MT} + b_{LT} \times F_{LT}\right) \tag{5-12}$$

Again, the HCM defines the PCE as a relative headway. Since h_{PC5-8} is the headway of a passenger car and b_i is the additional headway of vehicle type i, then

$$PCE_i = \frac{b_i + h_{PC5-8}}{h_{PC5-8}} \Rightarrow b_i = \left(PCE_i \times h_{PC5-8}\right) - h_{PC5-8} \tag{5-13}$$

Then,

$$TT_{5-8} = 4 \times h_{PC5-8} + \begin{pmatrix} (h_{PC5-8} \times PCE_{ST} - h_{PC5-8}) \times F_{ST} + \\ (h_{PC5-8} \times PCE_{MT} - h_{PC5-8}) \times F_{MT} + \\ (h_{PC5-8} \times PCE_{LT} - h_{PC5-8}) \times F_{LT} \end{pmatrix} \qquad [5\text{-}14]$$

If,

$$h_{SAT} = \frac{TT_{5-8}}{4} \qquad [5\text{-}15]$$

and

$$Pct_i = \frac{F_i}{4} \qquad [5\text{-}16]$$

then simplifying the equation and dividing by 4 to obtain one average headway value yields,

$$h_{SAT} = h_{PC5-8} \times \left[1 + Pct_{ST} \times (PCE_{ST} - 1) + Pct_{MT} \times (PCE_{MT} - 1) + Pct_{LT} \times (PCE_{LT} - 1)\right] \qquad [5\text{-}17]$$

This model was used to generate the PCE values. The results were similar to those obtained when using vehicle types in the total time formulation (Equation 5-11). This model was also run with a fixed value for $h_{PC\text{-}PC}$ and the results were similar to the ones obtained when using vehicle pairs (Equation 5-7). From these results it was concluded that the impact of trucks in the first few positions of the queue was not accounted for. In addition, the data set was expanded adding passenger car only queues to reduce the overall truck percentage from 46% to 10%. Then the same model was run (Equation 5-11) and the results were the same as fixing the $h_{PC\text{-}PC}$. Increasing the amount of passenger car only queues in the data set had a significant change to the estimation of the saturation headway.

The alternate form of the equation for f_{HV} can also be derived from the model based on all 16 vehicle pairs. Equation 5-18 gives the final form of this equation.

$$h_{5-8} = h_{PC-PC} \times \left(1 + \sum_{i,j=1}^{4} \left(Pct_{ij} \times \left(PCE_{ij} - 1\right)\right)\right) \qquad [5\text{-}18]$$

Model estimation results are shown in Table 5-7, where b_0 estimates the $h_{PC\text{-}PC}$ and the remaining betas estimate the PCE for vehicle *i* following *j*.

Table 5-7. Model estimation results for f_{HV} equation with 16 vehicle pairs

Coefficients	Estimate	Std. error	t-value
$h_{PC\text{-}PC}$	2.099136	0.002592	809.6979
b12	1.255759	0.003755	334.4511
b13	1.474404	0.003898	378.2679
b14	1.860251	0.004181	444.9291
b21	1.431489	0.003869	370.0254
b22	1.672034	0.005277	316.8362
b23	1.910199	0.006270	304.6783
b24	2.364296	0.006512	363.0607
b31	1.614128	0.003996	403.9260
b32	1.855496	0.006243	297.2072
b33	2.079154	0.005505	377.6886
b34	2.545657	0.006620	384.5518
b41	1.813318	0.004145	437.5067
b42	2.097403	0.006365	329.5371
b43	2.290151	0.006470	353.9746
b44	2.638246	0.005874	449.1694

Although this form is consistent with the HCM, to base the new PCE values it was decided to use the model (Equation 5-7) that estimates the total time for the first eight vehicles in queue to clear with the 16 different vehicle pairs. Accounting for the total time provides a better understanding of what occurred in the entire queue and it isolated the impact of trucks to the SLT from the saturation headway. However, the main objective of this research was to develop new PCEs for three truck categories that result in an f_{HV} value that match reasonably well with the values based on all 16 PCEs. It is not possible to have only one set of PCE values (for just the three truck categories) and have them result in the same f_{HV} value as for the 16 PCE values across a range of truck percentages.

In order to obtain the new PCE values for the three trucks types from the sixteen vehicle pairs' headway, two different methods were implemented. The first was to use the relative time consumed in queue (*H*) by each vehicle type. This method considered only the headway of each vehicle *i* when following a passenger car plus the average additional headway of any vehicle *j* trailing vehicle *i*. The results of this method are shown earlier in this chapter in Table 5-4.

The second was implemented adjusting the additional time consumed by each vehicle in queue. This adjustment comes from the observation that the time consumed by a vehicle *i* it is not exactly the same across different trailing vehicles. A general for of this method is shown in Equation 5-19.

$$H_{adj-i} = h_{i,PC} + \overline{\Delta h}_i + \frac{\left(\sum_{i,j=2}^{4} \left(\overline{\Delta h}_{i,j} - \overline{\Delta h}_j \right) \right)}{3} \qquad [5\text{-}19]$$

Where;

H_{adj-i} = adjusted time consumed by vehicle *i* in queue
$h_{i,PC}$ = headway of vehicle *i* when following a PC
$\overline{\Delta h}_i$ = average additional headway by vehicle *i*
$\overline{\Delta h}_{i,j}$ = additional headway of vehicle *i* when following vehicle *j*

Using the same approach of relative time consumed, PCE values of 1.75, 2.16, and 2.80 were obtained for small trucks, medium trucks, and large trucks respectively.

These values were extremely similar, and to be consistent with these results, it was decided to use values of 1.8, 2.2, and 2.8 for small, medium and large trucks respectively. Tests done with these values across a wide range of overall truck percentage, as well as relative truck type percentages, showed that the resulting f_{HV} values tracked reasonably well with those yielded by applying all 16 possible vehicle lead-follow PCE values. For situations where the relative truck type percentages might be extremely skewed (such as an exit from a distribution warehouse that consists entirely of large trucks) the specific PCE value from Table 5-2 can be used directly (in this case, $PCE_{LT\text{-}LT}$).

Furthermore, when relative truck type distributions are not available, a general approximation based on a relatively balanced distribution of small, medium, and large trucks in the traffic stream can be assumed. This approximation consisted of the average of the PCEs for these three categories. From Table 5-4 PCE values of 1.8, 2.2 and 2.8 can be obtained for small, medium and large trucks respectively. The average of these values yields to 2.267. Thus, a single truck PCE value of 2.3 can be applied. Again, this rough estimate considered a general assumption that small trucks, medium trucks and large trucks are equally distributed in the traffic stream.

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

The signal analysis methodology (Chapter 16) of the Highway Capacity Manual (HCM) currently recommends a single truck PCE value of 2.0. There is not much research available to support this value, and some transportation professionals have questioned the validity of this value, feeling that it generally underestimates the impact of trucks on saturation flow. This issue was the focus of this study; thus, the objective was to either validate this specific PCE value from the HCM, or to recommend a more appropriate value, or combination of values.

The results of this study are based primarily on simulation data. While not as ideal as having the results based strictly on field data, for this type of study that was focused on the effects of large trucks, there were some additional constraints beyond the typical saturation flow rate study that does not consider the effect of trucks. The challenge of finding sites with long queues (eight or more vehicles) that also had a large percentage of trucks, the large variance of the effect of trucks, and the sample size required due to this variance made it prohibitive to collect sufficient field data in an efficient manner within the scope and resources of this project.

Nonetheless, a considerable amount of field data were still collected, which provided for a reasonable data set to use for simulation program calibration. It was felt that the simulation program was calibrated fairly well to the field data, and replicated the general trends observed in the field data quite well. Thus, it is felt that the results provided in this study, despite the heavy reliance on simulation data, are reasonably valid and reliable.

The results presented in this study were based on using the same definition for saturation headway as provided in the HCM; that is, the average headway of vehicles in position 5 through 8 of the queue. Some studies [10] have indicated that the saturation flow rate will increase (saturation headway will decrease) with longer queues; although the body of evidence at this point is inconclusive. Li and Prevedouros [10] also found that in some cases the saturation headway may increase, and in others it might decrease. They indicated that in the former, headways at the end of a long queue compress, and in the latter headways elongate, and this is a function of drivers' performance. In this study, it was found that trucks in the first few positions of the queue impact not only the start-up lost time, but also the saturation headway of short queues. This result likely reflects the fact that start-up lost time extends beyond the fourth vehicle in queue, as it is expected that saturation headway would eventually reach a stable value if the queue is sufficiently long enough. This result could not be confirmed directly, due to computational limitations of the simulation program.

As indicated before, for purposes of this research it was assumed that the saturation section of the queue started after the fourth vehicle. Therefore it was decided to exclude this increase in the saturation headway because it is most likely a result of the additional lost time caused by the trucks being absorbed into the saturation flow rate estimate for short queues. From a practical standpoint, the accuracy of capacity estimates is not critical for situations when queues at signalized intersections are consistently short.

As it was demonstrated, headways are a function of both the leading and following vehicle in the queue, not just the trailing vehicle. This study categorized vehicles into four different types, which led to a total of 16 (4^2) possible leading-trailing vehicle pairs. New PCE values were developed for these pairs; however, it is not realistic to expect a practitioner to collect such detailed data on vehicle pair frequencies to apply these PCE values. Therefore, in order to provide more practical results, PCE values were developed based on three general categories of truck type—small, medium, and large. The method by which this was accomplished was to consider the time each vehicle type consumed during the queue clearance process. This *time* *consumed* was defined as the headway of the vehicle plus an additional time it adds to the trailing vehicle, and was based on the values obtained from the model that considered all 16 possible lead-trail vehicle pairs. The final recommended values for these PCEs are listed below.

- 1.8 for small trucks
- 2.2 for medium trucks
- 2.8 for large trucks

In this study small trucks include those trucks that have only two axles and between four and six tires. It also includes passenger cars with a trailer and garbage trucks, regardless of their number of axles. Medium trucks include those with three axles and usually range in length from 40 to 55 ft. Passenger cars with a trailer using the fifth-wheel, recreational vehicles (RVs) and small trucks with a trailer are also included as medium trucks. Large trucks include those with four or more axles, RVs with trailers and buses.

For situations in which only an estimate of the overall percentage of trucks in the traffic stream is available, then the single truck PCE value of 2.3 can be applied. Again, this value is a very general approximation, based a relatively balanced distribution of small, medium, and large trucks in the traffic stream. Use of this single general truck PCE value is reasonable for planning applications, where the level of precision may not be as important, or where it may not be possible to know the percentages of different truck classifications in the traffic stream.

For situations where the truck percentages, by category, are very unbalanced, the values of Table 5-2 can be used instead of the generalized ones. For example, if an approach to a signalized intersection serves a warehouse distribution center, where almost all the trucks are large, then the PCE value for LT-LT could be applied in the f_{HV} equation.

The increase in SLT due to trucks in the first four positions of the queue can be estimated using the percentage of trucks in the traffic stream. This estimate was based on the assumption that the trucks are distributed evenly throughout the queue. The results of this study showed a SLT for passenger car-only queues of approximately 2.5 seconds, which is larger than the HCM recommended value of 2.0 seconds. This value increases accordingly with an increase in truck percentage, reaching a value of approximately 17.5 seconds for 100% large trucks in the traffic stream.

Another finding of this study relates to the HCM recommended base saturation flow rate value of 1900 pc/h/ln. For the field data obtained in this study that included queues of only passenger cars, this saturation flow was rarely ever reached. As the results indicated, the average saturation headway for passenger car only queues was approximately 2.03 seconds, which corresponds to a saturation flow rate of 1773 pc/hr. While it is certainly debatable as to whether any of the field sites had what would be considered truly "ideal" conditions, per the general HCM definitions, this was generally the case. Only under the most optimal driver behavior and vehicle composition (e.g., only small passenger cars) conditions were average saturation headways below 2.0 observed. Although this field data set was not extremely large, the HCM recommended value for base saturation flow rate appears to be quite optimistic. This would seem to be particularly the case where the percentages of SUVs, mini-vans, and pick-up trucks currently make up a significant percentage of passenger "cars". Research by Kockelman and Shabih [6] found average headway values for these vehicle types to be greater than those for sedan-type passenger vehicles.

From detailed observations of the field data, it was found that a very high percentage of discharging queues have one or more drivers that hesitate/lag during their start-up process. While difficult to conclude exactly why this happens from the video recordings, it appears to frequently be a result of general driver inattentiveness or distraction. That is, the phenomenon of drivers "falling asleep at the wheel" while waiting to start-up from a stop at a signalized intersection approach appears to be quite common—one or two drivers (within the first eight positions of the queue) almost every cycle. These hesitating/lagging drivers have a significant impact on the capacity of the intersection. Queues that contained obvious instances of this driver hesitation were not included in the calculated value of 2.03; thus, this value still represents more ideal driver behavior. Unfortunately, this "ideal" driver behavior, at least for an entire discharging queue, seems to be quite rare. So based on just this phenomenon, the HCM recommended base saturation flow rate of 1900 pc/hg/ln would appear to be essentially unattainable over any reasonable length of time.

Recommendation for further studies

Ideally, a very large field study should be conducted, including not only all the variables in Bonneson's study [13], but also a comprehensive consideration of trucks and other vehicle types affecting the performance of the intersections. For example, heavily loaded trucks obviously have poorer acceleration capabilities than an unloaded truck. However, it must be recognized that trying to incorporate truck weight information into the methodology may not be practical, due to the difficultly associated with obtaining this type of measurement. With respect to determining the distribution of trucks within the queue (i.e., by queue position), a larger field study might also provide for the ability to reach a statistically valid conclusion on this issue.

A study including longer queues should be done in order to better identify the length of queue over which the Start-up Lost Time applies when trucks are present at the front of the queue. From the results of this research, trucks in the first four queue positions impact the discharge rate of vehicles in positions five through eight, and to better estimate the saturation flow rate it was necessary to exclude this impact. Studying longer queues in the field will also provide more insight into the issue of queue compression or elongation.

It is also recommended to study the frequency of inattentive drivers within the queue. It was observed how these situations frequently resulted in large gaps between vehicles, therefore resulting in an effective reduction of intersection capacity. This driver inattentiveness phenomenon could be could be included as a factor in the adjusted saturation flow rate calculation, somewhat analogous to a ‘local adjustment factor’.

APPENDIX A
DATA COLLECTION EQUIPMENT SETUP

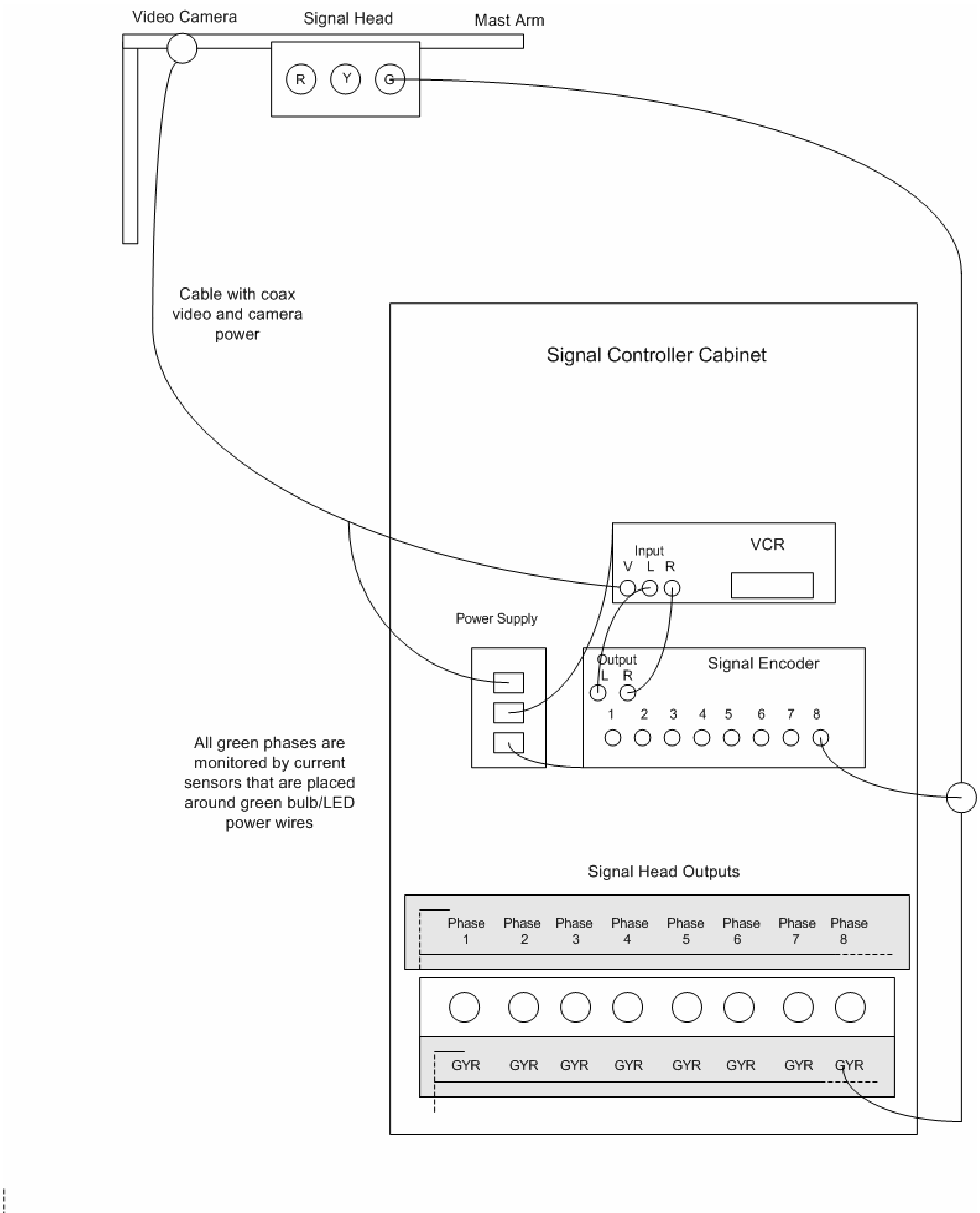


Figure A-1. Data collection equipment setup for method 1

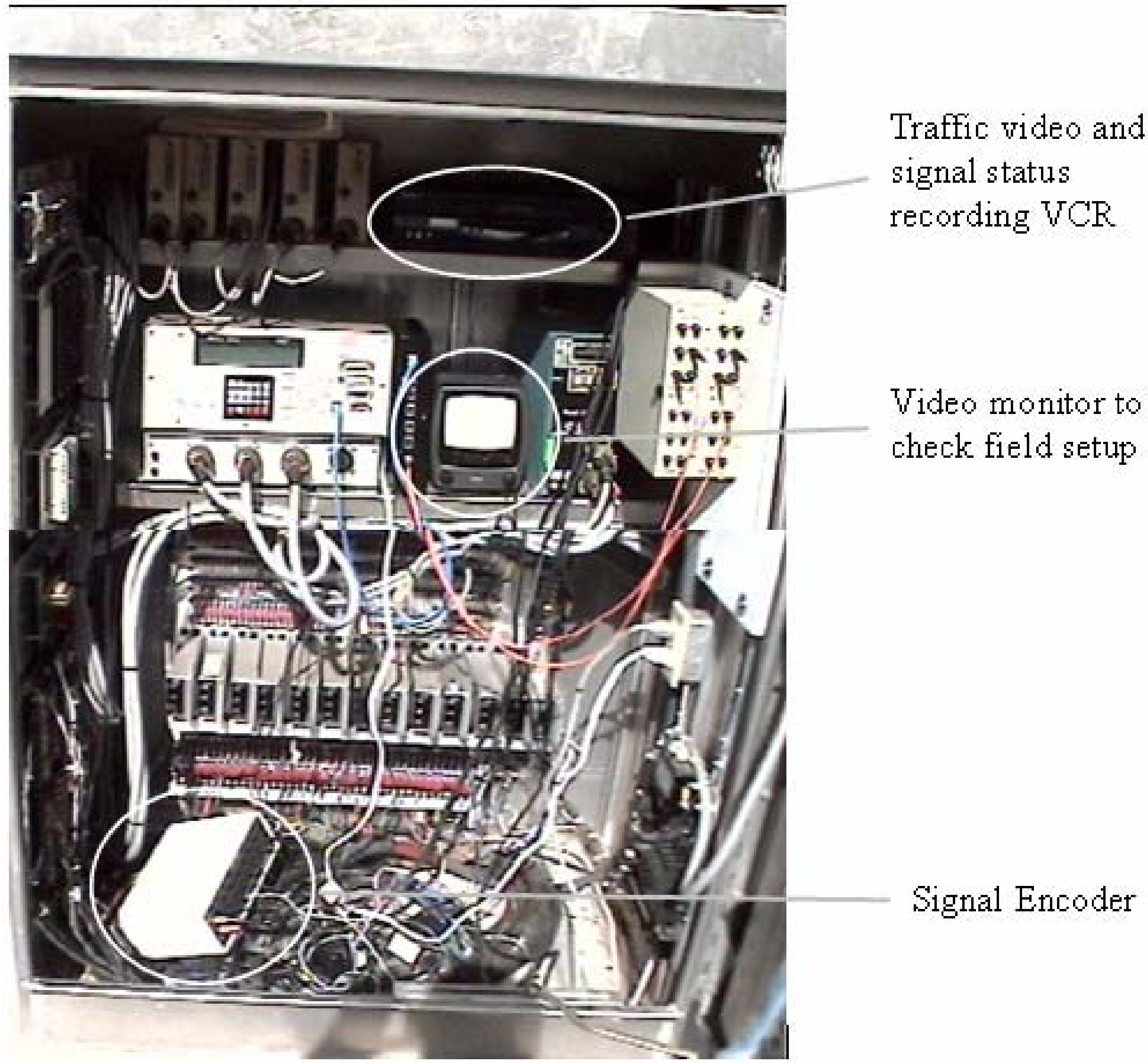


Figure A-2. Signal controller cabinet with data collection equipment installed for method 1

APPENDIX B
PICTURES OF VEHICLE TYPES BY CATEGORY

A

B

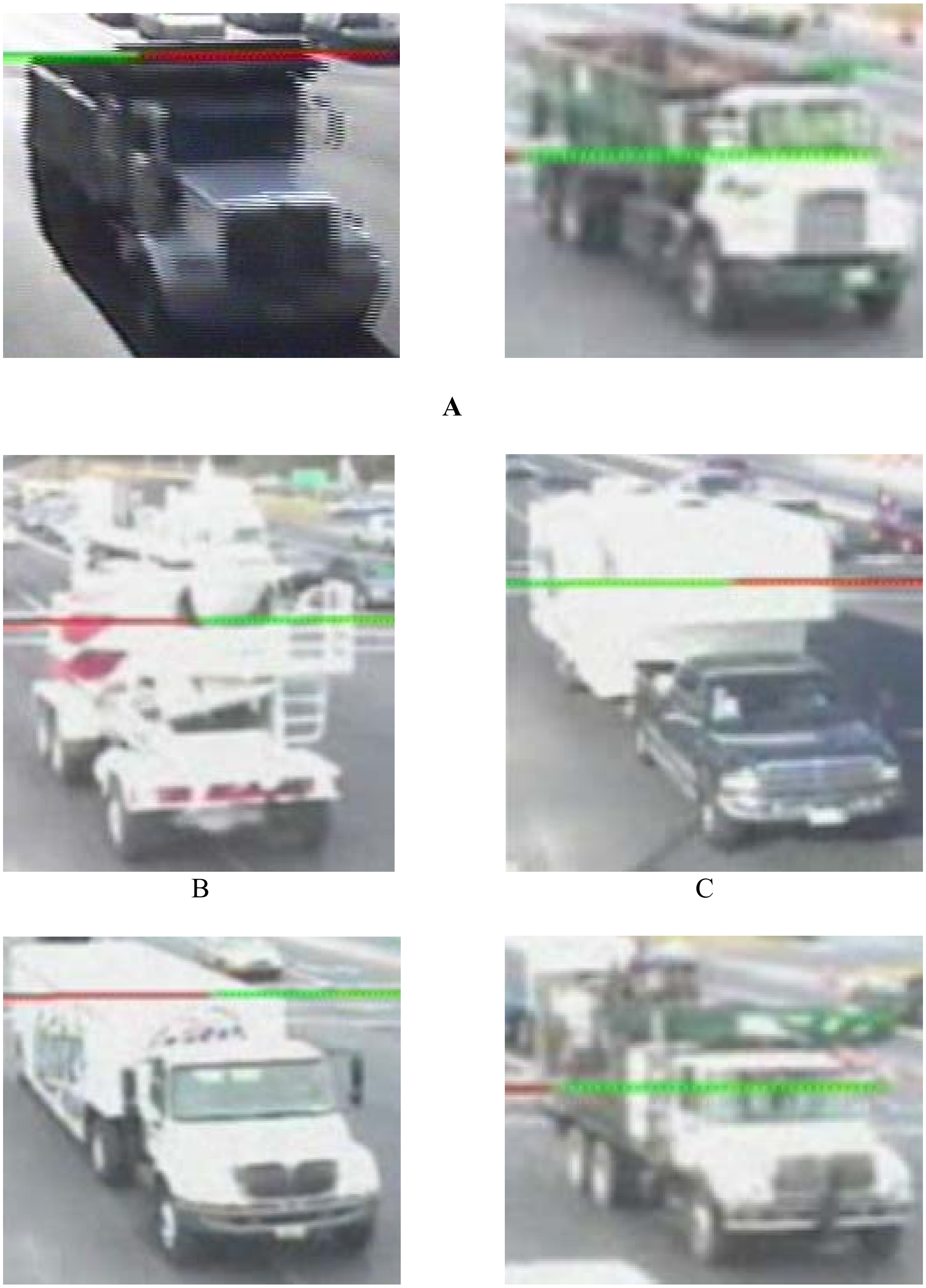
C

D

Figure B-1. Small trucks. A) Panel truck. B) Garbage truck. C) Two-Axle Single-unit dump truck. D) Small delivery truck. E) Passenger cars with trailers

E

Figure B-1. Continued



A

B C

D **E**

Figure B-2. Medium trucks. A) Three-Axle Single-unit dump truck. B) Concrete Mixer. C) Passenger car with trailer using fifth wheel. D) Delivery truck. E) Single-unit cargo truck.

A

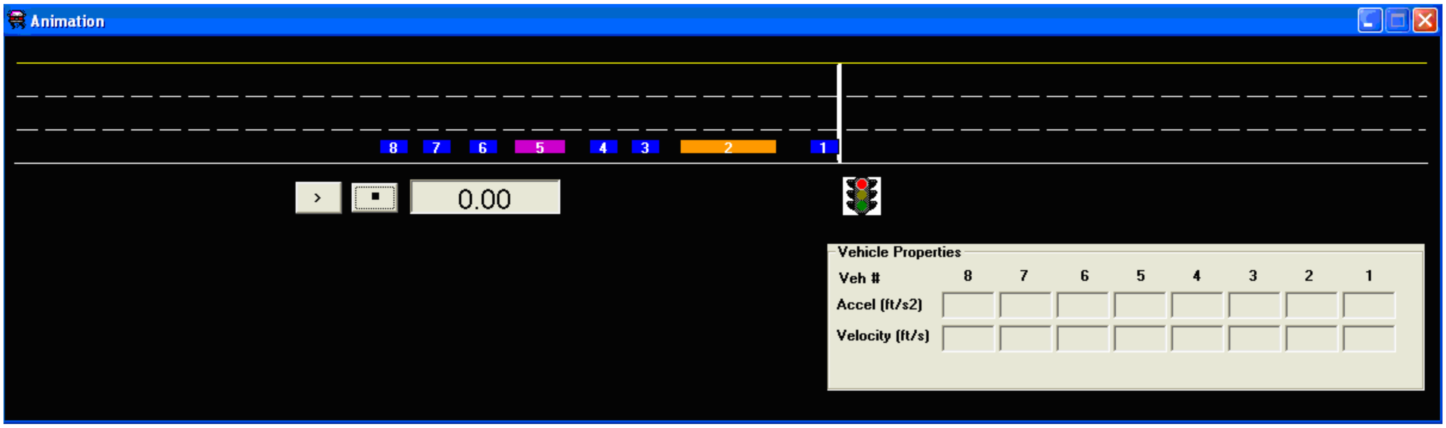
B

Figure B-3. Large trucks. A) Tractor plus trailer. B) Tractor plus flatbed. C) Buses.

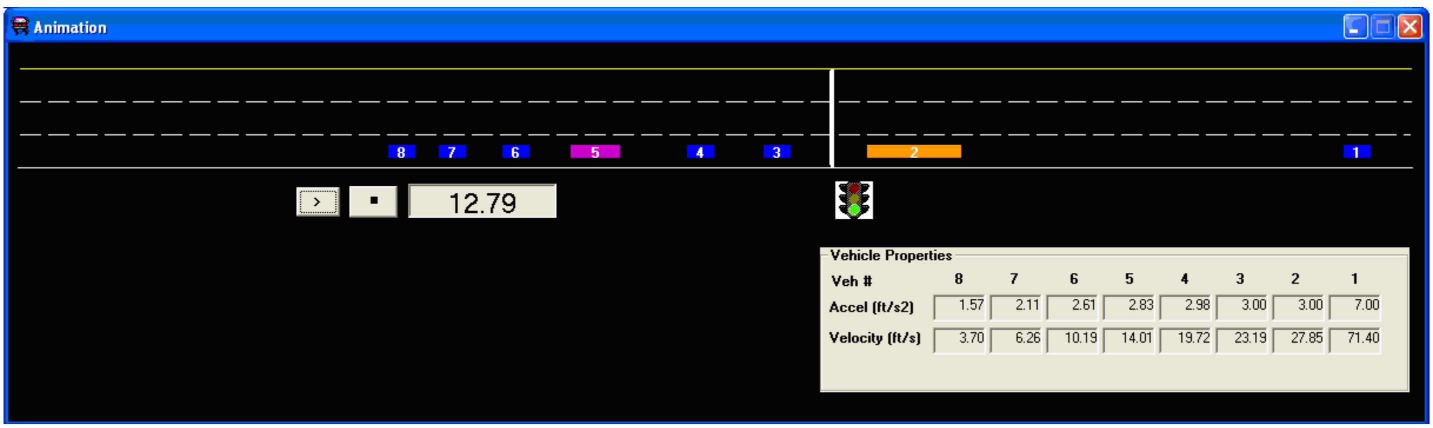
C

Figure B-3. Continued

APPENDIX C
SIMULATION PROGRAM SCREENSHOTS



A



B

Figure C-1. Simulation screenshot. A) Before signal turns green. B) Once the queue starts to discharge

APPENDIX D
QUEUE COMPOSITION

Table D-1. Vehicle type per position in queue

	Freq	Position in queue							
		1st	2nd	3rd	4th	5th	6th	7th	8th
1	174	1	1	1	1	1	1	1	1
2	2	1	1	1	1	1	1	1	2
3	3	1	1	1	1	1	1	1	3
4	9	1	1	1	1	1	1	1	4
5	9	1	1	1	1	1	1	2	1
6	6	1	1	1	1	1	1	3	1
7	6	1	1	1	1	1	1	4	1
8	5	1	1	1	1	1	2	1	1
9	5	1	1	1	1	1	3	1	1
10	11	1	1	1	1	1	4	1	1
11	5	1	1	1	1	2	1	1	1
12	6	1	1	1	1	4	1	1	1
13	6	1	1	1	2	1	1	1	1
14	1	1	1	1	4	1	1	1	1
15	3	1	1	2	1	1	1	1	1
16	3	1	1	3	1	1	1	1	1
17	10	1	1	4	1	1	1	1	1
18	7	1	2	1	1	1	1	1	1
19	3	1	3	1	1	1	1	1	1
20	7	1	4	1	1	1	1	1	1
21	6	2	1	1	1	1	1	1	1
22	6	3	1	1	1	1	1	1	1
23	7	4	1	1	1	1	1	1	1
24	1	1	1	1	1	1	1	2	2
25	1	1	1	1	1	1	1	3	2
26	1	1	1	1	1	1	1	3	4
27	6	1	1	1	1	1	1	4	4
28	1	1	1	1	1	1	2	3	1
29	1	1	1	1	1	1	2	4	1
30	1	1	1	1	1	1	2	1	4
31	1	1	1	1	1	1	3	2	1
32	1	1	1	1	1	1	4	2	1
33	1	1	1	1	1	2	1	2	1
34	2	1	1	1	1	2	1	1	4
35	2	1	1	1	1	2	2	1	1
36	1	1	1	1	1	3	1	2	1
37	1	1	1	1	1	3	1	4	1
38	1	1	1	1	1	3	3	1	1
39	1	1	1	1	1	4	1	2	1
40	1	1	1	1	1	4	1	1	2
41	1	1	1	1	2	1	1	2	1
42	1	1	1	1	2	1	1	4	1
43	1	1	1	1	2	1	1	1	4
44	1	1	1	1	2	3	1	1	1
45	1	1	1	1	4	1	1	2	1
46	1	1	1	1	4	1	1	3	1
47	1	1	1	1	4	1	1	4	1
48	2	1	1	1	4	1	1	1	3
49	1	1	1	1	4	1	4	1	1
50	1	1	1	2	1	1	1	3	1
51	2	1	1	2	4	1	1	1	1
52	1	1	1	4	1	1	1	1	2
53	1	1	1	4	1	1	3	1	1

Table D-1. Continued

Case	Freq	Position in queue 1st	2nd	3rd	4th	5th	6th	7th	8th
54	1	1	1	4	1	4	1	1	1
55	3	1	1	4	4	1	1	1	1
56	1	1	2	1	1	1	1	3	1
57	2	1	2	2	1	1	1	1	1
58	1	1	2	4	1	3	1	1	1
59	1	1	3	1	3	1	1	1	1
60	1	1	4	1	1	1	1	1	4
61	1	1	4	1	1	1	3	1	1
62	2	2	1	1	1	1	1	4	1
63	1	2	1	1	1	1	1	1	4
64	1	2	1	1	1	2	1	1	1
65	1	2	1	4	1	1	1	1	1
66	1	2	2	1	1	1	1	1	1
67	1	2	3	1	1	1	1	1	1
68	2	2	4	1	1	1	1	1	1
69	1	3	1	1	1	1	2	1	1
70	1	3	1	1	2	1	1	1	1
71	1	3	1	1	3	1	1	1	1
72	1	3	2	1	1	1	1	1	1
73	2	3	3	1	1	1	1	1	1
74	1	4	1	1	1	1	1	1	4
75	1	4	1	1	1	4	1	1	1
76	1	4	1	3	1	1	1	1	1
77	1	4	2	1	1	1	1	1	1
78	2	1	1	1	1	1	4	4	4
79	1	1	1	1	1	3	3	1	2
80	1	1	1	1	2	4	1	1	4
81	1	1	1	1	3	1	3	2	1
82	1	1	1	1	4	1	1	4	2
83	1	1	1	1	4	4	1	1	4
84	1	1	1	2	1	2	1	2	1
85	1	1	1	3	1	4	1	1	4
86	1	1	1	3	1	4	3	1	1
87	1	1	1	4	4	4	1	1	1
88	1	1	2	1	3	1	1	1	3
89	1	1	2	1	4	1	2	1	1
90	1	1	4	1	1	1	1	4	2
91	1	1	4	2	1	1	1	3	1
92	1	1	4	4	1	4	1	1	1
93	1	2	4	1	1	1	4	1	1
94	1	2	2	1	1	1	2	1	1
95	1	2	2	1	3	1	1	1	1
96	1	2	2	3	1	1	1	1	1
97	1	3	1	1	1	1	2	2	1
98	1	3	1	1	1	4	4	1	1
99	1	3	1	1	3	2	1	1	1
100	1	3	4	1	1	1	1	1	4
101	1	3	4	1	1	1	2	1	1
102	1	4	1	1	1	1	1	4	4
103	1	4	2	1	4	1	1	1	1
104	1	1	1	1	1	4	4	4	4
105	1	1	1	4	4	1	4	1	4
106	1	1	3	4	1	4	1	1	2
107	1	1	3	4	4	4	1	1	1
108	1	2	1	1	3	3	4	1	1
109	1	4	1	4	1	1	1	3	4
110	1	4	2	1	4	1	1	4	1

Table E-1. Headway statistics for 16 lead-trail vehicle combinations

	Vehicle in position 2															
Trailing	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Leading	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
count	61	6	7	6	6	4	1	3	4	1	1	0	6	2	2	0
mean	3.171	4.217	5.060	6.516	3.836	6.413	5.050	8.157	3.644	4.440	7.920		4.457	6.238	6.130	
stdev	0.844	0.744	0.801	0.967	0.972	2.423		1.608	0.464				0.627	3.115	0.170	
	Vehicle in position 3															
Trailing	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Leading	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
count	63	11	4	8	4	1	0	1	4	1	0	0	9	1	2	1
mean	2.512	3.482	4.344	5.212	2.875	3.605		5.830	2.923	3.800			4.158	3.510	4.835	3.900
stdev	0.785	0.546	1.410	0.765	0.714				0.768				0.924		2.029	
	Vehicle in position 4															
Trailing	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Leading	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
count	61	5	5	9	7	0	0	0	7	0	0	0	11	1	0	4
mean	2.360	2.922	3.097	4.670	2.880				4.120				3.842	3.360		5.739
stdev	0.631	0.240	0.619	0.955	0.637				1.298				1.054			0.349
	Vehicle in position 5															
Trailing	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Leading	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
count	58	5	5	13	6	0	1	0	5	1	1	0	11	1	0	3
mean	2.235	2.578	2.708	4.293	2.929		4.620		5.132	3.620	3.990		3.435	4.500		4.290
stdev	0.644	0.389	0.275	0.680	0.581				1.048				0.950			0.231

Table E-1. Continued

	Vehicle in position 6															
Trailing	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Leading	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
count	61	6	4	12	9	1	0	0	5	0	2	1	6	0	1	2
mean	2.297	3.548	2.833	3.973	3.147	3.165			2.876		5.195	4.240	5.331		4.000	5.345
stdev	0.509	0.673	0.383	0.747	1.135				0.975		1.520		1.653			0.714
	Vehicle in position 7															
Trailing	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Leading	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
count	57	7	6	6	8	1	2	1	8	1	0	0	10	1	0	2
mean	2.080	2.626	2.649	4.288	2.691	3.070	3.040	5.090	3.239	7.590			3.807	4.490		5.895
stdev	0.436	0.403	0.513	0.647	0.686		0.566		0.826				1.575			1.407
	Vehicle in position 8															
Trailing	1	1	1	1	2	2	2	2	3	3	3	3	4	4	4	4
Leading	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
count	56	11	6	7	5	1	1	2	3	0	0	0	12	0	2	4
mean	2.133	2.533	2.701	3.982	2.676	3.010	6.970	3.350	3.728				3.933		4.925	5.377
stdev	0.471	0.295	0.715	0.549	0.438			0.269	0.652				1.116		3.231	0.900

APPENDIX F
CALIBRATION RESULTS AND MSE CALCULATIONS

Table F-1. Simulation model calibration results

	Vehicle composition								Field		Simulation		MSE calculation	
Case	1	2	3	4	5	6	7	8	2-8	5-8	2-8	5-8	2-8	5-8
1	1	1	1	1	1	1	1	1	2.36	2.18	2.39	2.05	0.001	0.018
2	1	1	1	1	1	1	1	2	2.46	2.64	2.50	2.25	0.002	0.155
3	1	1	1	1	1	1	1	3	2.66	2.59	2.60	2.41	0.003	0.031
4	1	1	1	1	1	1	1	4	2.80	2.69	2.75	2.67	0.002	0.000
5	1	1	1	1	1	1	2	1	2.61	2.56	2.57	2.38	0.001	0.034
6	1	1	1	1	1	1	3	1	2.97	2.68	2.74	2.66	0.050	0.000
7	1	1	1	1	1	1	4	1	2.92	3.19	3.01	3.14	0.009	0.003
8	1	1	1	1	1	2	1	1	2.47	2.44	2.58	2.40	0.012	0.002
9	1	1	1	1	1	3	1	1	2.75	2.94	2.76	2.70	0.000	0.061
10	1	1	1	1	1	4	1	1	3.13	3.33	3.03	3.18	0.009	0.022
11	1	1	1	1	2	1	1	1	2.57	2.44	2.60	2.41	0.000	0.001
12	1	1	1	1	4	1	1	1	2.81	3.04	3.06	3.22	0.059	0.034
13	1	1	1	2	1	1	1	1	2.42	2.29	2.60	2.24	0.033	0.003
14	1	1	1	4	1	1	1	1	2.94	2.86	3.09	2.82	0.023	0.002
15	1	1	2	1	1	1	1	1	2.77	2.40	2.62	2.09	0.022	0.092
16	1	1	3	1	1	1	1	1	2.52	2.09	2.81	2.13	0.085	0.002
17	1	1	4	1	1	1	1	1	3.03	2.39	3.12	2.20	0.007	0.034
18	1	2	1	1	1	1	1	1	2.70	2.38	2.62	2.06	0.007	0.100
19	1	3	1	1	1	1	1	1	2.71	2.24	2.85	2.15	0.020	0.007
20	1	4	1	1	1	1	1	1	2.89	2.11	3.15	2.21	0.071	0.010
21	2	1	1	1	1	1	1	1	2.50	2.18	2.64	2.10	0.019	0.007
22	3	1	1	1	1	1	1	1	2.70	2.28	2.87	2.15	0.030	0.016
23	4	1	1	1	1	1	1	1	2.81	2.18	3.21	2.20	0.159	0.000
24	1	1	1	1	1	1	2	2	2.69	2.70	2.69	2.57	0.000	0.016
25	1	1	1	1	1	1	3	2	3.21	3.33	2.85	2.87	0.129	0.206
26	1	1	1	1	1	1	3	4	2.89	3.42	3.03	3.18	0.021	0.057
27	1	1	1	1	1	1	4	4	3.03	3.42	3.24	3.54	0.042	0.013
28	1	1	1	1	1	2	3	1	3.35	3.97	2.91	2.96	0.196	1.024
29	1	1	1	1	1	2	4	1	2.97	3.57	3.15	3.39	0.032	0.033
30	1	1	1	1	1	2	1	4	3.31	3.51	2.91	2.97	0.156	0.288
31	1	1	1	1	1	3	2	1	2.75	2.66	2.95	3.04	0.040	0.142
32	1	1	1	1	1	4	2	1	3.36	4.30	3.22	3.51	0.019	0.612
33	1	1	1	1	2	1	2	1	3.00	2.96	2.78	2.73	0.050	0.052
34	1	1	1	1	2	1	1	4	2.93	3.19	2.91	2.96	0.001	0.049
35	1	1	1	1	2	2	1	1	2.89	3.02	2.77	2.72	0.015	0.092
36	1	1	1	1	3	1	2	1	3.05	3.05	2.96	3.04	0.008	0.000
37	1	1	1	1	3	1	4	1	3.26	3.83	3.29	3.63	0.001	0.038
38	1	1	1	1	3	3	1	1	2.97	3.50	3.07	3.24	0.010	0.070
39	1	1	1	1	4	1	2	1	2.93	3.10	3.25	3.57	0.100	0.215
40	1	1	1	1	4	1	1	2	2.86	3.08	3.17	3.42	0.094	0.120
41	1	1	1	2	1	1	2	1	3.30	2.77	2.79	2.57	0.255	0.042
42	1	1	1	2	1	1	4	1	3.12	2.91	3.17	3.24	0.002	0.111

Table F-1. Continued

	Vehicle composition								Field		Simulation		MSE calculation	
Case	1	2	3	4	5	6	7	8	2-8	5-8	2-8	5-8	2-8	5-8
43	1	1	1	2	1	1	1	4	2.72	2.50	2.93	2.81	0.044	0.097
44	1	1	1	2	3	1	1	1	2.84	2.54	2.93	2.82	0.010	0.084
45	1	1	1	4	1	1	2	1	2.69	2.81	3.26	3.12	0.324	0.096
46	1	1	1	4	1	1	3	1	3.85	3.70	3.40	3.36	0.201	0.114
47	1	1	1	4	1	1	4	1	3.41	2.96	3.55	3.60	0.020	0.410
48	1	1	1	4	1	1	1	3	3.43	3.44	3.25	3.11	0.032	0.108
49	1	1	1	4	1	4	1	1	3.32	3.66	3.54	3.61	0.051	0.003
50	1	1	2	1	1	1	3	1	2.49	2.44	2.93	2.64	0.190	0.039
51	1	1	2	4	1	1	1	1	2.86	2.66	3.24	2.73	0.141	0.004
52	1	1	4	1	1	1	1	2	2.97	2.65	3.24	2.42	0.075	0.053
53	1	1	4	1	1	3	1	1	3.67	2.63	3.42	2.74	0.059	0.012
54	1	1	4	1	4	1	1	1	3.59	3.08	3.58	3.00	0.000	0.005
55	1	1	4	4	1	1	1	1	3.65	2.83	3.57	2.62	0.006	0.044
56	1	2	1	1	1	1	3	1	2.93	3.23	2.95	2.63	0.000	0.363
57	1	2	2	1	1	1	1	1	2.81	2.28	2.81	2.08	0.000	0.042
58	1	2	4	1	3	1	1	1	3.53	3.45	3.57	2.70	0.002	0.576
59	1	3	1	3	1	1	1	1	2.89	2.29	3.15	2.39	0.067	0.010
60	1	4	1	1	1	1	1	4	3.09	2.25	3.37	2.59	0.080	0.116
61	1	4	1	1	1	3	1	1	2.96	2.66	3.47	2.74	0.260	0.008
62	2	1	1	1	1	1	4	1	3.40	3.79	3.21	3.06	0.036	0.525
63	2	1	1	1	1	1	1	4	2.91	2.74	2.95	2.63	0.002	0.012
64	2	1	1	1	2	1	1	1	2.40	2.48	2.83	2.41	0.182	0.006
65	2	1	4	1	1	1	1	1	2.96	2.32	3.27	2.16	0.093	0.027
66	2	2	1	1	1	1	1	1	2.61	1.89	2.83	2.08	0.047	0.037
67	2	3	1	1	1	1	1	1	3.08	2.09	3.00	2.10	0.006	0.000
68	2	4	1	1	1	1	1	1	3.05	2.37	3.28	2.16	0.054	0.044
69	3	1	1	1	1	2	1	1	2.74	2.88	3.06	2.48	0.103	0.162
70	3	1	1	2	1	1	1	1	2.86	1.92	3.06	2.27	0.039	0.123
71	3	1	1	3	1	1	1	1	2.65	1.92	3.17	2.39	0.280	0.223
72	3	2	1	1	1	1	1	1	2.82	2.18	3.07	2.13	0.064	0.002
73	3	3	1	1	1	1	1	1	3.42	2.35	3.17	2.09	0.062	0.069
74	4	1	1	1	1	1	1	4	3.50	3.51	3.44	2.60	0.004	0.824
75	4	1	1	1	4	1	1	1	3.08	3.19	3.68	3.02	0.363	0.032
76	4	1	3	1	1	1	1	1	2.93	1.69	3.51	2.13	0.335	0.195
77	4	2	1	1	1	1	1	1	3.16	2.13	3.39	2.16	0.053	0.001
78	1	1	1	1	1	4	4	4	4.36	5.62	3.73	4.41	0.403	1.462
79	1	1	1	1	3	3	1	2	3.76	4.06	3.19	3.46	0.316	0.363
80	1	1	1	2	4	1	1	4	3.40	4.01	3.43	3.71	0.001	0.092
81	1	1	1	3	1	3	2	1	2.97	2.72	3.28	3.31	0.097	0.347
82	1	1	1	4	1	1	4	2	3.09	3.00	3.65	3.81	0.319	0.647
83	1	1	1	4	4	1	1	4	3.21	3.55	3.78	4.02	0.327	0.216
84	1	1	2	1	2	1	2	1	3.28	3.03	2.98	2.72	0.091	0.092
85	1	1	3	1	4	1	1	4	2.81	2.80	3.57	3.45	0.574	0.434
86	1	1	3	1	4	3	1	1	3.00	2.97	3.63	3.58	0.399	0.372
87	1	1	4	4	4	1	1	1	3.43	2.94	4.03	3.42	0.364	0.227

Table F-1. Continued

	Vehicle composition								Field		Simulation		MSE calculation	
Case	1	2	3	4	5	6	7	8	2-8	5-8	2-8	5-8	2-8	5-8
88	1	2	1	3	1	1	1	3	2.84	2.44	3.13	2.70	0.088	0.067
89	1	2	1	4	1	2	1	1	3.09	2.49	3.44	3.06	0.120	0.330
90	1	4	1	1	1	1	4	2	3.56	2.92	3.72	3.19	0.027	0.075
91	1	4	2	1	1	1	3	1	3.30	2.57	3.68	2.75	0.141	0.033
92	1	4	4	1	4	1	1	1	3.06	2.46	4.06	2.90	0.995	0.190
93	2	4	1	1	1	4	1	1	4.37	3.39	3.73	2.96	0.415	0.182
94	2	2	1	1	1	2	1	1	3.38	2.10	3.00	2.36	0.145	0.066
95	2	2	1	3	1	1	1	1	3.83	2.46	3.16	2.37	0.451	0.008
96	2	2	3	1	1	1	1	1	2.78	2.32	3.15	2.08	0.140	0.059
97	3	1	1	1	1	2	2	1	2.88	2.57	3.25	2.81	0.139	0.057
98	3	1	1	1	4	4	1	1	3.65	3.61	3.82	3.81	0.030	0.043
99	3	1	1	3	2	1	1	1	3.38	2.97	3.35	2.71	0.001	0.071
100	3	4	1	1	1	1	1	4	3.21	2.34	3.65	2.55	0.195	0.044
101	3	4	1	1	1	2	1	1	3.19	2.43	3.60	2.48	0.171	0.003
102	4	1	1	1	1	1	4	4	3.59	3.42	3.91	3.45	0.106	0.001
103	4	2	1	4	1	1	1	1	3.95	2.83	3.86	2.59	0.007	0.055
104	1	1	1	1	4	4	4	4	4.07	4.82	4.21	5.24	0.019	0.172
105	1	1	4	4	1	4	1	4	3.66	3.28	4.27	3.83	0.375	0.307
106	1	3	4	1	4	1	1	2	4.03	3.04	3.96	3.15	0.006	0.012
107	1	3	4	4	4	1	1	1	3.58	3.04	4.27	3.29	0.471	0.059
108	2	1	1	3	3	4	1	1	3.76	3.80	3.78	3.80	0.000	0.000
109	4	1	4	1	1	1	3	4	3.30	2.12	4.23	3.10	0.868	0.972
110	4	2	1	4	1	1	4	1	4.58	3.87	4.30	3.37	0.074	0.247

LIST OF REFERENCES

1. Highway Capacity Manual. Transportation Research Board (TRB), National Research Council, Washington, D.C., 2000.

2. Washburn, S.S., Courage, K.G., and MacKenzie, S. Development of a Red Light Violation Data Collection Tool. Southeastern Transportation Center, Final Report. 2001. http://stc.utk.edu/htm/pdf%20files/RLR_TRB1.pdf

3. Highway Capacity Manual. Highway Research Board, Special Report 87, National Research Council, Publication 1328, Washington D.C., 1965.

4. Molina, CJ, Jr. Development of Passenger Car Equivalencies for Large Trucks at Signalized Intersections. Institute of Transportation Engineers, Journal vol. 57 Issue 11, pp. 33-37. November 1987.

5. Benekohal, R and Zhao, W. Delay-Based truck Equivalencies at Signalized Intersections: Results and Field Data. Third International Symposium on Highway Capacity, Copenhagen, Denmark. June 1998.

6. Kockelman, K and Shabih, R. Effect of Light-Duty Trucks on the Capacity of Signalized Intersections. Journal of Transportation Engineering: American Society of Civil Engineers, Volume: 126 Issue: 6 pp. 506-512. November 2000.

7. Bonneson, JA and Messer, CJ. Phase Capacity Characteristics from Signalized Interchange and Intersection Approaches. Transportation Research Record: Journal Transportation Research Board. No. 1646. TRB, National Research Council. Washington D.C., pp. 96-105. 1998.

8. Bonneson, JA, Nevers, B., Zegeer, J., Nguyen, T. and Fong, T. Guidelines for Quantifying the Influence of Area Type and Other Factors on Saturation Flow Rate. Final Report #DO2319. Florida Department of Transportation. Tallahassee, FL. June 2005.

9. Perez-Cartagena, R. and Tarko, A. Calibration of Capacity Parameters for Signalized Intersections in Indiana. 84th Annual Meeting of the Transportation Research Board, January 9-13, 2005, Washington D.C.

10. Li, H. and Prevedouros, P. Detailed Observations of Saturation Headway and Start-Up Lost Times. Journal of the Transportation Research Board, TRR 1802, Washington, D.C., 2002.

11. Cohen, S. L. Application of Car-Following Systems to Queue Discharge Problem at Signalized Intersections. Transportation Research Record: Journal of the Transportation Research Board, TRR 1802, Washington, D.C., 2002.

12. Greenshields, B. D. Distance and Time Required to Overtake and Pass Vehicles. Highway Research Board Proceedings, Vol. 15, 1935, pp.332-342.

13. Bonneson, James A. Modeling Queued Driver Behavior at Signalized Junctions. Transportation Research Record: Journal Transportation Research Board. No. 1365. TRB, National Research Council. Washington D.C., pp. 99-107. 1992.

14. Briggs T. Time Headways on Crossing the Stop Line after Queueing at Traffic Lights. Traffic Engineering, July & Control, May 1977, pp. 264-265.

15. Messer C. J. and Fambro D. B. Effects of Signal Phasing and Length of Left-Turn Bay on Capacity. In Transportation Research Record 644, TRB National Research Council, Washington, D.C., 1977, pp. 95-101.

16. Buhr J.H., Whiston R.H., Brewer K.A., and Drew D.R. Traffic Characteristics for Implementation and Calibration of Freeway Merging Control Systems. In Highway Research Record 279, HRB, National Research Council, Washington, D.C., 1969, pp. 87-106.

17. Evans L. and Rothery R.W. Influence of Vehicle Size and Performance on Intersection Saturation Flow. Proc., 8th International Symposium on Transportation and Traffic Theory, University of Toronto Press, Toronto, Ontario, Canada, 1981, pp. 193-222.

18. George E.T. and Heroy F.M. Starting Response of Traffic at Signalized Intersections. Traffic Engineering, July 1966, pp. 39-43.

19. Akçelik R., Besley M. and Roper R. Fundamentals Relationships for Traffic Flows at Signalised Intersections. Research Report ARR 340. ARRB Transport Research Ltd, Vermont South, Australia. 1999

20. Pipes L.A. An Operational Analysis of Traffic Dynamics, Journal of Applied Physics, Vol. 24, NO. 3, 1953, pp. 274-287.

21. May, A. Traffic Flow Fundamentals. Published by Prentice-Hall, Inc., 1990, Chapter 6, pp. 160-191.

22. Long, Gary. Intervehicle Spacing and Queue Characteristics. Transportation Research Record: Journal Transportation Research Board. No. 1796. TRB, National Research Council. Washington D.C., pp. 86-96. 2002.

23. Florida Department of Transportation. ARTPLAN Level Of Service analysis software. http://www.dot.state.fl.us/planning/systems/sm/los/los_sw2.htm

24. Cohen, S. L. Application of Car-Following Systems in Microscopic Time-Scan Simulation Models. Journal of the Transportation Research Board, TRR 1802, Washington, D.C., 2002.

25 StatSoft, Inc. (2006). STATISTICA (data analysis software system), version 7.1 www.statsoft.com.