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16. Abstract

This report is a compilation of research papers written by students participating in the 2002 Undergraduate Transportation Engineering Fellows Program. The ten-week summer program, now in its twelfth year, provides undergraduate students in Civil Engineering the opportunity to learn about transportation engineering through participation in sponsored transportation research projects. The program design allows students to interact directly with a Texas A&M University faculty member or TTI researcher in developing a research proposal, conducting valid research, and documenting the research results through oral presentations and research papers.

The papers in this compendium report on the following topics, respectively: 1) color recognition of retroreflective raised pavement markers; 2) impacts of localized congestion on mobile source emissions; 3) transportation improvement strategies for Houston's Texas Medical Center; 4) effects of access management on vehicle crashes; 5) computer micro-simulation analysis of intersections near highway-railroad grade crossings; and 6) the costs and benefits associated with electronic toll collection and variable toll rates.

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COMPENDIUM OF STUDENT PAPERS:

2002 UNDERGRADUATE TRANSPORTATION

ENGINEERING FELLOWS PROGRAM

Participating Students (left to right):
Jonathon Tydlacka, Jeffrey Miles, Justin Winn,
S. Matthew Feil, Kit Black, and Scott Abraham,

Program sponsored by

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PREFACE

The Southwest Region University Transportation Center (SWUTC), through the Advanced Institute Program, the Texas Transportation Institute (TTI) and the Civil Engineering Department at Texas A&M University, established the Undergraduate Transportation Engineering Fellows Program in 1990. The program design allows students to interact directly with a Texas A&M University faculty member or TTI researcher in developing a research proposal, conducting valid research, and documenting the research results through oral presentations and research papers. The intent of the program is to introduce transportation engineering to students who have demonstrated outstanding academic performance, thus developing a critical resource: capable and qualified future transportation leaders.

In the summer of 2002, the following six students and their faculty/staff mentors were:

STUDENTS	MENTORS
Mr. Scott Abraham Texas A&M University	Ms. Melisa Finley
Mr. Kit Black Texas A&M University	Mr. Stephen Farnsworth
Mr. S. Matthew Feil Texas A&M University	Mr. Brian Bochner
Mr. Jeffrey Miles Texas A&M University	Dr. William Eisele
Mr. Jonathon Tydlacka Texas A&M University	Dr. Laurence Rilett
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- Ms. Pam Rowe and Mr. Ivan Lorenz, who assisted in producing the final compendium manuscript; and
- Ms. Joan Stapp, who assisted with various program administrative matters.

The authors also recognize that support was provided by a grant from the U.S. Department of Transportation, University Transportation Centers Program to the Southwest Region University Transportation Center.

CONTENTS

DISCLAIMER

COLOR RECOGNITION OF RETROREFLECTIVE RAISED PAVEMENT MARKERS

By

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August 2002

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INTRODUCTION

Traffic control devices provide a vital link between the design and use of roadways. Traffic control devices are those cues to the driver that direct, regulate, or advise (*1*). Recently, new color schemes have been introduced in the area of signing (e.g., fluorescent colors). In order to increase the effectiveness of the new signing color schemes, a leading manufacturer of retroreflective raised pavement markers (RRPMs) has proposed three new fluorescent colors: orange, yellow-green, and yellow. However, before manufacturers and transportation agencies can begin utilizing the new RRPM colors, they must be proven safe and effective.

The primary concern of the new RRPMs is color recognition. In the first phase of a study conducted by the Texas Transportation Institute (TTI) on the color recognition of RRPMs, it was determined that approximately half the subjects misinterpreted the color of the fluorescent orange (FO) RRPMs in a simulated work zone environment at night (*2*). That is, the fluorescent orange markers were identified as a color other than orange (i.e., usually red). Based on these results, two alternative shades of the fluorescent orange RRPMs were developed. The experiment reported herein evaluates the color recognition of the two new shades of the fluorescent orange RRPMs (Phase 2). Secondary factors that were considered include motorist age, gender, and time of day (i.e., day versus night). The results will help determine what shade, if any, will be suitable for public use.

RESEARCH OBJECTIVE

The objective of this study was to determine the daytime and nighttime color recognition of two experimental shades of fluorescent orange RRPMs. Variables that were considered in the experiment included subject's age and gender, time of day, and shade of fluorescent orange RRPM. The experiment sought to evaluate the performance of two new fluorescent orange RRPMs, but did not theorize an optimal range of fluorescent orange shades.

Listed below are the goals the researchers believed would lead to useful results:

1. The subjects' perceptions of color are representative of the driving population at large.
2. Produce an accurate account of the frequency of correctly and/or incorrectly identified fluorescent orange markers.
3. The results could be used to determine the feasibility of the new pavement marker shades.

BACKGROUND

Previous research that has been reviewed includes studies and texts covering topics such as the properties of color, color perception, and the effects of aging on vision and driving performance. In addition, the experimental design and results from the Phase 1 RRPM color recognition study were reviewed.

Properties of Color

The color of any object is determined by the wavelengths of light that the object reflects rather than absorbs. The spectrum of visible colors, ranging from violet to red, is 400 to 700 nanometers (nm). Orange falls in the range of 580 to 630 nm and red falls in-between 630 and 700 nm (*3*). An orange object, therefore, absorbs all wavelengths of light in the range of 400 to 700 nm except those that lie in between 580 and 630 nm.

Some colors, as an intrinsic property of wavelength, can be more noticeable and distinctive than others. This property is known as intensity. Yellow-green has the highest intensity followed by orange and blue (*4*). Intensity aside, orange has a tendency to be confused with red and yellow. Red and yellow are primary colors that can be mixed together to form various shades of orange, a secondary color. The manufacturer's goal while developing the fluorescent orange markers is to minimize that confusion.

Fluorescent colors alter short wavelengths (ultraviolet) by making them longer. In the case of a fluorescent orange object, ultraviolet rays are transformed into visible orange light waves. The result is an increase in the amount of light waves reflected as compared to standard colors. In fact, fluorescent colors can be up to four times brighter than standard colors (*5*). The fluorescent RRPMs currently being developed are designed for daytime detection as well as nighttime detection. This is a departure from the current application of markers, which use standard colors and are designed for nighttime detection only.

Biological Explanation of Vision

Vision, the primary sensory mechanism in humans, encompasses both biological and psychological processes. The first step to vision is a light source, whether natural or artificial, that produces light waves. Once the light waves (an image) hit the human eye, the lens and cornea flip the image and focus it. After the image has been focused, it passes through an opening in the eye called the retina. The retina's job is to adjust the opening size according to how much light is in the environment. At midday, the retina is small because the light source is bright. At dusk, the retina expands because the light source has dimmed (*6*).

After the light passes through the retina, it enters the fovea, which is the part of the eye that contains millions of cones and rods. Ninety-five percent of the population contains three types of cones: red-green, blue-yellow, and black-white. Rods see only black, white, and shades of gray and are utilized in low-light environments. Appropriately stimulated cones and rods send a signal along the optical nerve to the visual cortex (a portion on the back of the brain responsible for vision) where the image is flipped right side up (*3*).

Healthy vision relies on the proper functioning of this process. Otherwise a host of vision problems may occur. For instance, color blindness results when one or more types of cones are missing in the eye's fovea (*3*). The most common type of color blindness is red-green defective vision. This type of color blindness occurs when the red-green cones are missing (*7*). Visual acuity also relies on the proper functioning of the visual system. Visual acuity is the ability to discriminate fine objects is expressed as a ratio (i.e., 20/30). Normal vision is considered to be 20/20, which means at a distance of 20 ft, an object "looks" like it is 20 ft away (*8*). In consideration of these deficiencies, two requirements for this experiment are that subjects have a visual acuity that allows them to have a Texas driver's license and that no form of color blindness exists.

Effects of Demographics on Driving Studies

An important consideration of any research project is the effect of demographics on the results. The two driver characteristics addressed in this research are age and gender. Previous studies on the performance of female drivers have found no significant differences between genders (*9*). In addition, in 2000 the United States had nearly as many females with drivers' licenses as males (*10*).

In contrast, previous studies have shown that age can be a great influence on driver ability. More specifically, a direct correlation between aging and degeneration of abilities such as visual perception and fine motor control has been found. However, many drivers adapt these fading abilities by using tactics such as driving at slower speeds, planning routes, or driving more user-friendly vehicles (*11*). It has also been determined that older drivers (55+) tend to miss cues in high stimulus areas such as work zones (*12*). In addition, some evidence supports the notion that older drivers may have a hard time distinguishing between some colors of pavement markings (*11*).

Effects of Aging on Vision

The lens of the eye, as discussed before, is responsible for the majority of the magnification within the eye. As a person ages, the lens loses its concavity and thus its focal power. This is the primary reason people's vision deteriorates with age. Completely clear at birth, the lens also develops a yellowish film that can distort color. The result of these conditions can be a loss of visual acuity and a decrease in color sensitivity (*9*).

Phase 1 RRPM Color Recognition Study

As noted earlier, the study conducted herein is the second phase of an ongoing study. The first phase, conducted in July of 2001 at the same location (Texas A&M Riverside Campus) included a similar color recognition experiment. Subjects were driven through a course consisting of 13 treatments. RRPM treatment colors consisted of red, white, yellow, blue, green, fluorescent orange, and fluorescent yellow-green. Subjects were instructed to identify the color of each treatment. A total of ten subjects (both young and old) volunteered to participate (*2*).

The nighttime data for the isolated fluorescent orange RRPM treatment had a 50 percent initial and an 80 percent final correct color response. The fluorescent orange work zone RRPM treatment had an initial correct color response of 40 percent and a final correct color response of 60 percent. The amount of incorrect responses to the fluorescent orange RRPMs led the researchers to conclude that the fluorescent orange markers tested should not be used at night in work zones (*2*). Additional product development by the manufacturer led to the two new shades of fluorescent orange markers that were studied in the research documented in this paper.

EXPERIMENTAL DESIGN

The purpose of the color recognition of retroreflective raised pavement marking study was to evaluate any color distinction between the fluorescent orange and red RRPMs in isolated and work zone conditions. Experimentation was carried out at the TTI proving ground facility located at the Texas A&M Riverside Campus near College Station, Texas. The TTI proving ground is a former WWII Air Force base consisting of runways and associated taxiways. A map of the runway system is shown in Figure 1. This site was ideally suited for this experiment due to the area's low ambient light and seclusion from the general public (insuring safety for the subjects).

As mentioned earlier, this study was the second phase of an ongoing research project. As such, the same manner of evaluating the RRPMs was used in Phase 2 as in Phase 1. The testing period for Phase 2 was from July 16, 2002 to July 25, 2002. Daytime and nighttime experiments were conducted between 9:00 a.m. and 4:00 p.m. and from 9:00 p.m. to midnight, respectively.

Subject Recruitment and Screening

TTI recruited ten subjects from the Bryan-College Station area to participate in the study. The general pool of possible subjects was a rural college centered community whose population is near 200,000. Potential subjects were screened using standard visual acuity (Snellen) and color blind (Dvorine) tests. In addition, all subjects were required to hold a current valid drivers license, although they did not drive in any part of the experiment.

Table 1 shows the age and gender distribution of the subjects. While the intention was for the total male and female ratios to be balanced, schedule and reservation conflicts forced the researchers to accept an older male subject in place of an older female subject.

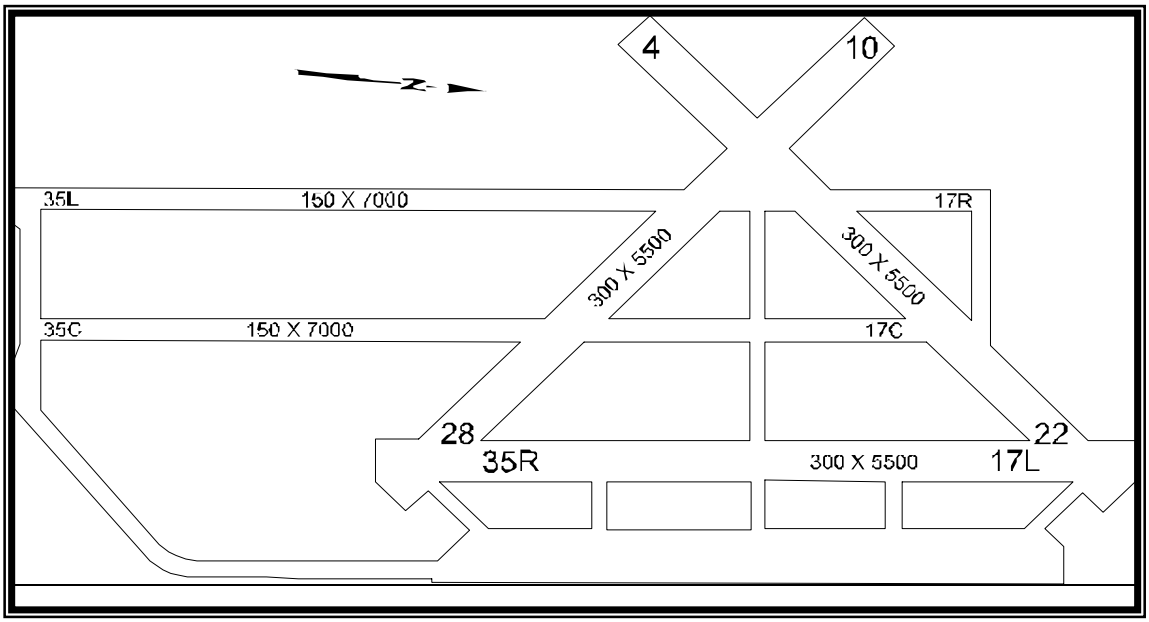


Figure 1: Riverside Campus Runway Layout

Table 1: Subject Demographics

Age Category	Males			Females			Overall		
	No. of Subjects	Avg. Age	Avg. VA	No. of Subjects	Avg. Age	Avg. VA	No. of Subjects	Avg. Age	Avg. VA
18-35	3	26	20/15	2	26	20/13	5	26	20/14
55+	3	63	20/20	2	65	20/15	5	64	20/18
Total	6	45	20/18	4	46	20/14	10	46	20/16

Treatments

As discussed previously, the research documented herein is part of a large-scale research project that contained 19 treatments and seven colors of RRPMs. Figure 2 shows the full diagram of treatments. The six treatments evaluated in this study have been circled. Table 2 describes the six treatments.

Each treatment contained a single color of marker. Each treatment was either categorized as isolation (only RRPMs) or work zones (contained signing, barrels, and RRPMs). Isolation treatments were 440 ft long and consisted of 23 RRPMs spaced 20 ft apart (Figure 3). Work zone treatments were 560 feet long, consisted of 29 RRPMs, and had the following characteristics (Figure 4):

- 180 foot taper length,
- 30 foot barrel spacing in the taper,

- 7 barrels in the taper,
- 300 foot tangent length,
- 60 foot barrel spacing in the tangent,
- 5 barrels in the tangent,
- 12 foot lane width, and
- straight alignment.

The materials for the treatments were procured from several sources. The signs were property of TTI and were stored on site. Barrels were rented from a local company for the duration of the study. A single manufacturer provided the various colors and shades of RRPMs. All treatments were left in place for the duration of the experiment.

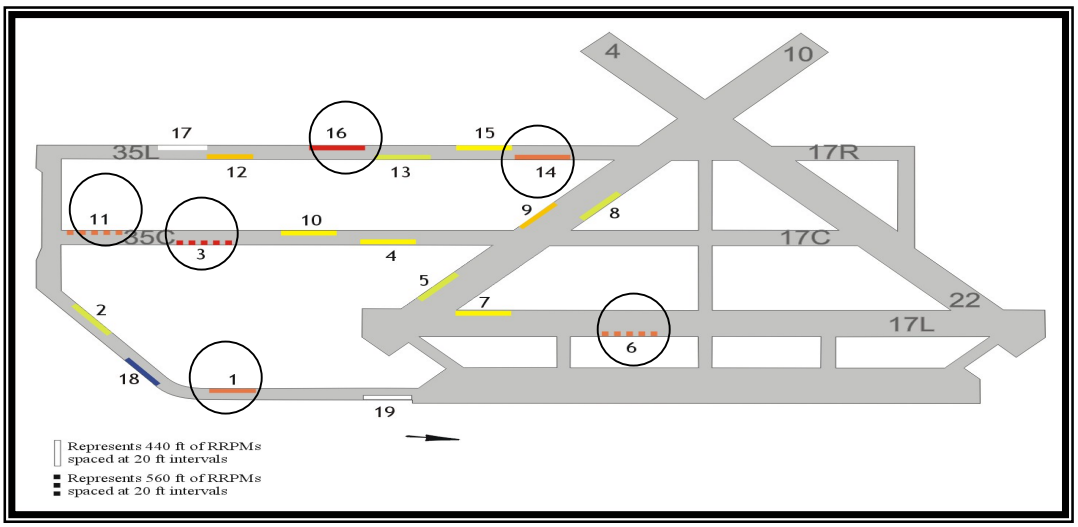


Figure 2: Research Treatment Layout

Study Protocol

Upon arrival of the subjects, researchers read a script explaining what the experiment would entail. If the subject was willing to continue with the experiment, they were then asked to read and sign an Informed Consent document. Before the start of the experiment, subjects were given standard static visual acuity and color blind screening tests.

Subjects completed a total of one daytime study and one nighttime study. In order to minimize any potential learning effects, the treatment order and time of day for the initial study was distributed as shown in Table 3. Six subjects completed the daytime study first, and four subjects completed the nighttime study first. Originally, it was intended that half the subjects would complete the daytime study first and half would complete the nighttime study first, but due to scheduling conflicts this did not occur. Table 4 describes the two different treatment orders. Gender and age were also divided by treatment order and time of day.

Table 2: Description of RRPM Treatments

Treatment	RRPM color	Description
1	Fluorescent Orange (shade 2)	Isolation
3	Red	Work Zone
6	Fluorescent Orange (shade 2)	Work Zone
11	Fluorescent Orange (shade 1)	Work Zone
14	Fluorescent Orange (shade 1)	Isolation
16	Red	Isolation

Figure 3: Daylight Isolation Treatment

Figure 4: Daylight Work Zone Treatment

Table 3: Randomization of Experiment Runs

	Daytime First	**Nighttime First**
Order 1 First	30%	20%
Order 2 First	30%	20%

Table 4: Treatment Orders

Order 1	**Order 2**
16	11
3	1
6	3
1	6
14	16
11	14

The test vehicle, a 1998 Chevrolet Lumina (sedan), was driven by a researcher. The subject was positioned in the passenger seat and provided verbal responses to the approaching RRPMs in the form of color descriptions. A second researcher, located in the backseat, recorded the color responses and their associated distances, including multiple descriptions of the same RRPM. For example, the researcher recorded a subject's initially incorrect response to the color of the RRPM and a later corrected response as the distance to the treatment decreased. The distance away from each treatment was recorded for each response using a distance-measuring instrument (DMI).

The experiment was designed as a course that contained a series of colored RRPM treatments. The vehicle approached the treatments at 30 mph. Daytime and nighttime procedures were identical.

Pilot Study

A representative of the manufacturing company as well as two employees of the Texas Transportation Institute evaluated the experimental design of the study during the day and at night. Some minor modifications to the experimental design were made (i.e., moving treatment locations). The experimental design as explained herein is the final version.

RESULTS

The two measurements of effectiveness used to evaluate the color recognition of the fluorescent orange RRPMs were initial color response and final color response. Originally, the color response distance from the treatment was also going to be utilized. However, the color response distance data was found to contain too many variables for the purpose of comparing the distances amongst the various treatments. The most notable problems were the variations in sight distance available before each treatment and pavement irregularities. These uncontrollable factors prevented cross the board comparisons among treatments.

The results are presented by daytime and nighttime, as well as by treatment description. Each color or shade of RRPM was tested in isolation and in a work zone. Table 5 below shows the method by which a subject's color response was recorded for analysis purposes.

Table 5: Color Classification

Subject's Color Response	Researcher's Interpreted Color
Yellow	Yellow
Red	Red
Orange	Orange
Light Orange	Orange
Dark Orange	Orange
Pale Orange	Orange

Daytime Results

The results of the daytime study show the high color recognition of the fluorescent orange RRPMs in the daylight (Figure 5). More specifically, initially the fluorescent orange shades performed near 100 percent correct. The final response of subjects demonstrates that every subject correctly identified the color of all fluorescent orange RRPMs. The red RRPMs had slightly lower initial and final daytime color recognition (85 and 95 percent correct, respectively).

As shown in Figure 5, some initial color responses from the subjects for both fluorescent orange and red RRPMs were incorrect. The work zone fluorescent orange shade 2 RRPM treatment had one incorrect initial response recorded as yellow. However, the subject made a final correct response of orange. Red RRPMs had a greater incidence of incorrect identification. The isolated red RRPM treatment was incorrectly identified once as orange, while the work zone red RRPM treatment had two initial incorrect identifications, one as orange and another as pink. In both cases, the orange responses were changed to red, but the pink response remained unchanged.

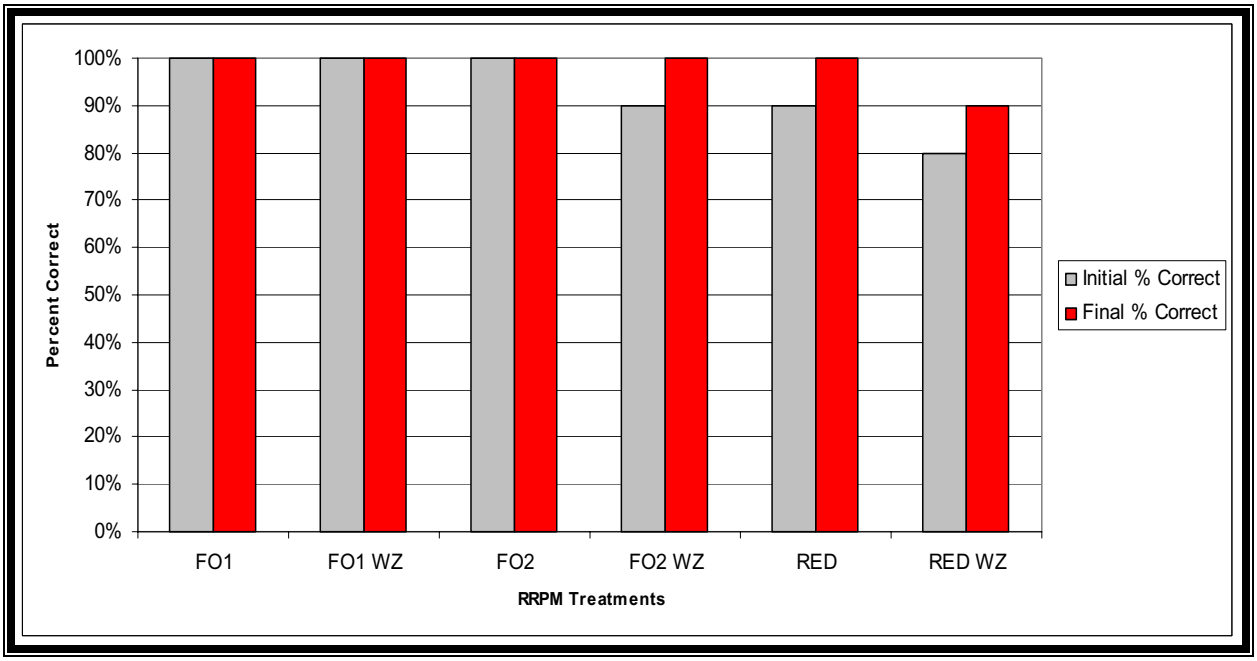


Figure 5: Initial and Final Daytime Color Responses

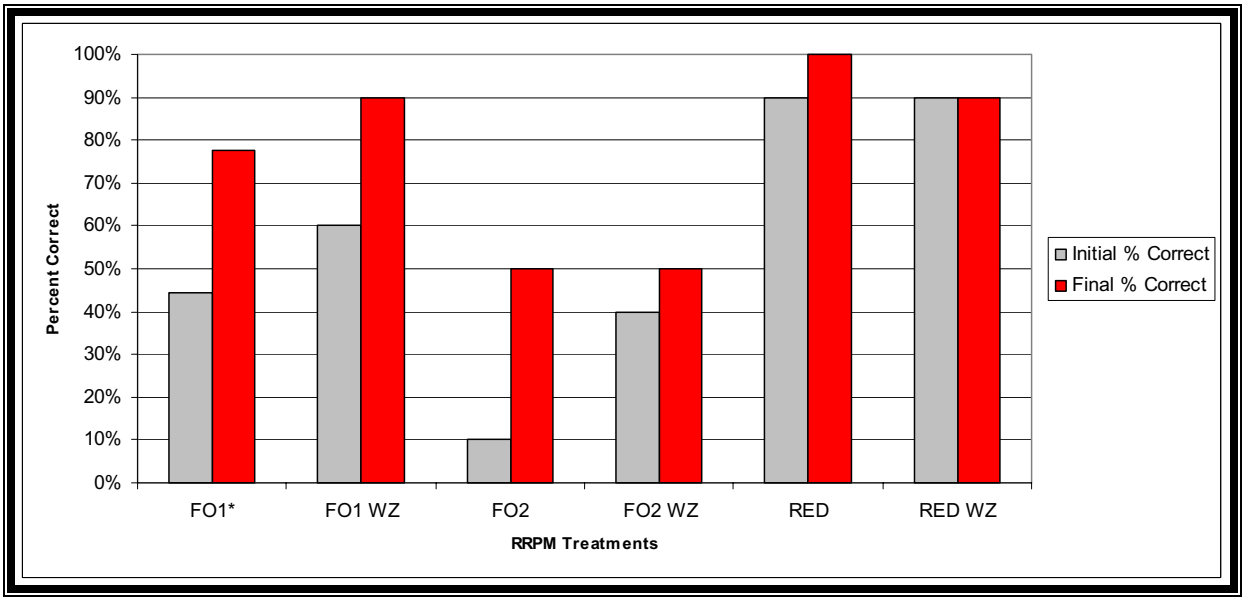
Table 6 summarizes the percentage of treatments correctly identified by age and gender rather than treatment type. Upon inspection, there were only minor differences between the two age groups and the two gender types.

Nighttime Results

The performance of the fluorescent orange RRPMs at night did not meet that of the daytime study (Figure 6). The fluorescent orange shade 1 RRPMs had an average initial correct response of approximately 50 percent over both treatments. Final correct responses improved to approximately 85 percent over both treatments (a 35 percent increase in correct color response). In both initial and final response categories, the work zone fluorescent orange shade 1 RRPM treatment had approximately 14 percent higher correct responses.

Table 6: Percentage of RRPMs Correctly Identified (Final Response)

Group	Percent Correct
Old	97%
Young	100%
Male	97%
Female	100%



*N = 9 instead of 10 because one subject's response was not recorded.

Figure 6: Initial and Final Nighttime Color Responses

All incorrect color responses for the fluorescent orange shade 1 RRPMs were red. In the case of the isolated fluorescent orange shade 1 RRPM treatment, four of the five subjects who gave incorrect initial responses were young. Concerning final responses, young subjects gave both of the incorrect responses. Three older subjects and one young subject gave the initial incorrect responses for the work zone fluorescent orange shade 1 RRPM treatment. All of these subjects, except for the one young subject, changed their color response to orange.

The performance of fluorescent orange shade 2 RRPMs was below that of the shade 1 RRPMs. The average initial correct response for the fluorescent orange 2 RRPMs was 25 percent over both treatments. The final correct response increased to an average of 50 percent (which is approximately 35 percent lower than fluorescent orange shade 1 RRPMs). While the final percent correct responses were equal for both shade 2 treatments, the initial responses for the work zone treatment were 30 percent more accurate than the isolated treatment.

As with fluorescent orange shade 1 RRPMs, all incorrect color responses for the fluorescent orange shade 2 RRPMs were red. Ninety percent of the subjects (a mixture of young and old) initially misinterpreted the color of the fluorescent orange shade 2 RRPMs in isolation. Only four subjects (two young and two old) changed their responses to orange. Similar to the isolated treatment, the incorrect color responses (both initially and finally) for the fluorescent orange shade 2 RRPMs in a work zone were provided by both young and old subjects (approximately 50/50 split).

The red RRPMs performed above either of the two fluorescent orange RRPMs. The average initial and final correct responses for both red RRPM treatments were 90 percent and 95 percent, respectively. Incorrect responses were all orange.

Table 7 summarizes the percentage of treatments correctly identified by age and gender rather than treatment type. As with the daytime data, there were only minor differences between the two age groups and the two gender types. Overall, age and gender differences in subjects did not seem to have considerable affect on detection in either the daytime or nighttime studies.

Table 7: Percentage of RRPMs Correctly Identified (Final Response)

Group	Percent Correct
Old	79%
Young	73%
Male	71%
Female	75%

Comparison of Phase 1 and Phase 2 Results

As was mentioned earlier, this experiment was the second phase of an ongoing study. Tables 8 and 9 summarize the results of the three shades of fluorescent orange RRPMs evaluated thus far. The notation "FO" is the fluorescent orange RRPM shade used in the Phase 1 study.

Table 8: Daytime Final Results

Treatment	Isolated	Work Zone	Overall
FO	90%	100%	95%
FO1	100%	100%	100%
FO2	100%	100%	100%

Table 9: Nighttime Final Results

Treatment	Isolated	Work Zone	Overall
FO	80%	60%	70%
FO1	78%	90%	84%
FO2	50%	50%	50%

The daytime color responses for all shades of fluorescent orange RRPMs are approximately equal, while the nighttime isolated treatments of fluorescent orange RRPMs show some variability. The fluorescent orange shade 1 RRPMs performed near the original fluorescent orange RRPMs in isolation (only a 2 percent difference in correct responses). In contrast, the performance of the fluorescent orange shade 2 RRPMs in isolation dropped by 30 percent from the original fluorescent orange RRPMs in isolation.

The effect the work zones had on the color response is also noteworthy. In Phase I of the experiment (FO), the work zone decreased color recognition by 20 percent. However, in Phase 2 the fluorescent orange shade 1 RRPMs showed a 12 percent improvement when included in a work zone. The fluorescent orange shade 2 RRPMs showed no difference in the color response from isolation to work zone.

SUMMARY AND CONCLUSIONS

Summary

During the day, both shades of the fluorescent orange RRPMs were identified correctly by all subjects. However, the color responses for the nighttime study varied more than the daytime color responses. The nighttime performance of the fluorescent orange shade 1 RRPMs (approximately 84 percent correct) was above that of shade 2 (50 percent correct), but both were exceeded by the red RRPMs (95 percent correct). The subjects that incorrectly identified the fluorescent orange RRPMs interpreted them to be red, and the subjects that incorrectly identified the red RRPMs thought they were orange.

Compared to the Phase 1 results, it is believed that the fluorescent orange shade 1 RRPMs had a closer color resemblance to the work zone barrels and sign, and thus were interpreted as “orange” by 90 percent of the subjects. In contrast, the fluorescent orange shade 2 RRPMs did not improve the color recognition.

Conclusions

Overall, the fluorescent orange shade 1 RRPMs improved the color recognition of the experimental orange markers compared to the original shade of fluorescent orange RRPMs and the fluorescent orange shade 2 RRPMs. Since some subjects misinterpreted the color of the fluorescent orange shade 1 RRPMs, more color recognition research is needed to ensure that there are no situations that may cause drivers to mistake the fluorescent orange RRPMs for red RRPMs. This research should include a larger subject pool, identical treatment locations (i.e., equal sight distance), and utilize color recognition distance data.

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SCOTT ABRAHAM

Scott Abraham will earn his Bachelor of Science degree in Civil Engineering from Texas A&M University in December 2002. He completed his core engineering curriculum at Georgia Institute of Technology before transferring to Texas A&M. He is a member of the Chi Epsilon Honor Society, as well as a student member of the American Society of Civil Engineers. His career interests include transportation technology and construction. Scott's hobbies include swimming, running, music, soccer and pilot training.

IMPACT OF LOCALIZED CONGESTION ON MOBILE SOURCE EMISSIONS

by

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August 2002

SUMMARY

Mobile source emissions have become a heavily scrutinized topic since the passing of the 1990 Clean Air Act Amendments (CAAA). As metropolitan areas in the southern and western United States become increasingly more suburbanized, annual vehicle miles of travel (VMT) increase which causes an increase in congestion. The impact areas of congestion have on mobile source emissions is not well known.

This study evaluated the impact that localized congestion had on mobile source emissions in the Dallas/Ft. Worth, TX metropolitan area using a newly developed emissions model, the Comprehensive Modal Emissions Model (CMEM). This model can be used at both the microscale and macroscale to estimate tailpipe emissions and fuel usage. Peak and off-peak data was collected on five (5) freeway sections that exhibited consistent, predictable congestion problems. The results show that congestion does cause emissions and fuel use, on a per mile basis, to increase.

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INTRODUCTION

With environmental protection interests becoming ever more important, air pollution is one of the major areas that is being scrutinized. Air pollution comes from many sources: stationary, mobile, and also natural occurring events. Stationary sources consist mainly of factories, power plants, smelters, and on a smaller scale dry cleaners and degreasing facilities. Cars, busses, air traffic, construction equipment, trains, and others make up the bulk of the mobile emissions sources. Naturally occurring sources include volcanic activity and windblown dust (1). Mobile and stationary sources produce many different types of emissions, which are regulated by local, state, and federal agencies.

The federal agency responsible for safeguarding the natural environment is the Environmental Protection Agency (EPA). The EPA has laws and regulations that are enforced by the state agencies, which in Texas is the Texas Natural Resources Conservation Commission (TNRCC). The TNRCC enforces many different types of regulations, some of which limit the amount of specific types of emissions such as carbon monoxide (CO), forms of nitrogen oxides (NO_x), and hydrocarbons (HC). These emissions are the major pollutants produced by mobile sources. Emissions are regulated and monitored throughout the state, but stricter policies are now in place for areas where the air quality fails to reach attainment status according to standards specified in the 1990 Clean Air Act Amendments (CAAA).

Currently, there are four non-attainment areas in Texas. Houston and Dallas/Ft. Worth are the only areas that require emissions testing for vehicle inspection. These two areas have the non-attainment status because of their high levels of ozone (O_3). These urban areas exhibit characteristics similar to other metropolitan areas in the southern and western United States. The heavy suburbanization of these areas has led to increased annual vehicle miles of travel (VMT) and a subsequent increase in the levels of congestion. El Paso and Beaumont/Port Arthur are the two other non-attainment areas in Texas, but they do not yet require emissions testing. Beaumont/Port Arthur has non-attainment levels of O_3, and El Paso is a non-attainment area due to high levels of O_3, CO, and particulate matter (PM-10). It is critical that the mobile emissions be reduced in order for these areas to reach attainment status so they can continue to receive federal transportation funds.

Problem Statement

Mobile emissions, particularly cars and small trucks, known as light duty vehicles (LDV), are a main contributor to the overall emissions produced. As traffic volumes on a roadway increase, the level of service (LOS) decreases and localized congestion occurs. The impact these areas of localized congestion and bottlenecks have on emissions is not well known. There is a need to better define what impact localized congestion, bottlenecks, and other problem areas have on mobile source emissions.

Currently, agencies at the local, state, and federal levels use the mobile source emission-factor model MOBILE (developed by the U.S. Environmental Protection Agency) to develop emission factors. MOBILE can be used to develop emission inventories on a regional level but is not well suited for evaluating improvements that are more "microscopic" in nature. A newly developed emissions model, the Comprehensive Modal Emissions Model (CMEM), considers, "…at a more fundamental level the *modal* operation of a vehicle, i.e., emissions that are directly related to vehicle operation modes such as idle, steady-state cruise, various levels of acceleration/deceleration, etc (2)."

If the impact of localized congestion on mobile source emissions can be better understood, then further steps can be taken to help reduce emissions.

Research Objective

The overall objective of this research is to develop a better understanding of the impact localized congestion has on mobile source emissions. The research made use of emissions modeling methodology

and newly developed emissions modeling software, which has previously not been widely used. A determination will be made of how emissions would change if the peak period LOS were improved to the same LOS that is present during off-peak periods. That is, if the peak-period volume of traffic were to flow at a higher LOS because a bottleneck was removed, what would be the resulting change in emission?

Scope

This study evaluated the impact that localized congestion had on mobile source emissions in the Dallas/Ft. Worth, TX metropolitan area. Since there were numerous sites in the area that qualified for evaluation, the study was limited to five sites that had consistent, predictable congestion problems. Data was collected at each site using seven to eight replicates for both peak and off-peak periods. The morning and afternoon peak periods which were used for this study were 6:00-9:00 a.m. and 4:00-7:00 p.m., respectively. All of the locations were limited to freeway sections.

Literature Review

Prior to the beginning of this research, the Comprehensive Modal Emissions Model (CMEM) User's Guide (2) and several articles were reviewed in order to get a better understanding of the CMEM emission model and the ways the model has been used in the past. This model is relatively new and has not yet been widely used. The following is a result of the literature review search.

- Barth, Scora, and Younglove (3) compared the CMEM model to the EPA's MOBILE (version 6) and its new driving cycles. This article estimates emissions and fuel consumption for various levels of freeway congestion. This study found that the, "…mild acceleration perturbations at high speeds can lead to significantly higher emissions compared with the steady-state values. Because of this, the new high-speed freeway driving cycles (representing higher levels of service) in many cases have (modeled) emissions higher than those for the cycles that represent lower levels of service. Fuel consumption by speed does not change drastically in the comparison (3)."
- Barth, Malcolm, Younglove, and Hill (4) describe the latest validation work that compares results using CMEM to independent emission testing results, and the latest versions of EMFAC and MOBILE. In general, CMEM predicts well when compared with these other models. It was found that, "…CMEM is consistent with MOBILE and EMFAC at low to medium speeds. Greater deviations were found at very high speeds and very low speeds. At high speeds, CMEM tends to predict higher hydrocarbon (HC) emissions and lower oxides of nitrogen (NO_x) emissions. At the very low speeds, CMEM tends to predict lower than EMFAC and MOBILE for all emissions (4)."
- An, Barth, Scora, and Ross (5) present a modal emissions model for vehicles that are operated under incremental soak-time conditions. The CMEM model has previously been describe for vehicles operation under hot-stabilized conditions. "Vehicle incremental soak-time emissions refer to vehicle emissions after intermediate variable soak times of between 10 min and 24 h. Recent research shows that most on-road vehicles experience soak times of between 10 min and 24 h during daily driving; thus, there is strong desire to model vehicle emissions under such circumstances (5)." The modeling is a fuel-based modal emissions approach that models "…vehicles' fuel use, engine-out emissions, air/fuel equivalence ratio, catalyst efficiencies, and tailpipe emissions…individually as a function of variable soak time (5)." The model is capable of predicting vehicle emissions for variable soak time and any given test cycle, and it can predict emissions under different starting test cycles.
- Bose and Ioannou (6) evaluate the Intelligent Cruise Control (ICC) system using the CMEM model. The environmental evaluation of ICC vehicles shows that they have a beneficial effect on vehicle emissions and fuel consumption. The CMEM model was used for this emissions evaluation because of its modal aspects that relates vehicle emissions to vehicle operating modes such as idle, steady state cruise, various levels of acceleration/deceleration, etc.

- Barth, Malcolm, and Scora (7) perform an examination on the key interface issues between the comprehensive modal emissions and energy consumption (CME/EC) model and other Intelligent Transportation Systems (ITS) simulation models. "Methodologies for integrating various ITS transportation models/data sets with the CME/EC model were established…. Much of the work focused on integrating the CME/EC model with PARAMICS (7)." Two case studies were carried out using this PARAMICS/CME/EC tool. These case studies serve as an example of how to apply this new PARAMICS/CME/EC tool to create microscale emission inventories.
- An, Barth, Scora, and Ross (8) present a modal emissions model for vehicles operated under enleanment conditions. This is the same CMEM model that has been described previously for vehicles operating under stoichiometric and enrichment conditions. Lean-burn hydrocarbon emissions (HC_{lean}) account for 10 to 20 percent of the overall HC emissions based on the over 200 vehicles that were tested and modeled. The model estimates are compared with emission measurements, with encouraging results.

Most of the articles that are briefly summarized above were written during the development stages of the CMEM model. Only one article was obtained that described the use of CMEM for an evaluation purpose. In this article (6), the emissions model was used to evaluate the advantages of an Intelligent Cruise Control, but this was a limited evaluation.

STUDY METHODOLGY

CMEM Modeling Approach

CMEM uses a physical, power-demand modal modeling approach. In this type of physical model, the emissions process is broken down into different components associated with vehicle operation and emissions production. Each of the components is then modeled on various parameters that are characteristic of the component selected. These parameters are specific to different vehicle categories and are based on vehicle specifications (e.g. engine size, vehicle mass, number of gears, etc.) (2).

There are several key factors why the physical, deterministic modeling approach was chosen during the model development. Various components model the different vehicle processes related to emissions. Factors of the operation of the vehicle such as vehicle technology, operating modes, maintenance, road grade, and accessory use are taken into consideration. A heterogeneous vehicle population can be integrated and correctly weighted to create an entire emission inventory. This model can be used at both the microscale and macroscale level. Additionally, it can be validated and calibrated easily, it is not limited to pure steady-state emission events as is an emissions map approach, and the model is transparent making the results easy to evaluate.

Model Structure

The model does not predict emissions for specific makes and models of vehicles, but it estimates emissions based on different vehicle categories. Each of the composite vehicle categories were derived by grouping vehicles with similar operating characteristics. Table 1 provides the vehicle/technology categories that are utilized by CMEM.

Vehicle tailpipe emissions, on a second-by-second scale, are comprised of fuel rate (FR), engine-out emission indices ($g_{emission}/g_{fuel}$), and time-dependent catalyst pass fraction (CPF). The formula is described as:

$$\textit{Tailpipe emissions} = FR * (g_{emission}/g_{fuel}) * CPF$$

The fuel usage rate is measured in grams, engine-out emission index is shown in grams of engine-out emissions per gram of fuel consumed, and CPF is the ratio of tailpipe to engine-out emissions.

Table 1. CMEM Vehicle Categories

Category #	Vehicle Technology Category
	Normal Emitting Cars
1	**No Catalyst**
2	**2-way Catalyst**
3	**3-way Catalyst, Carbureted**
4	**3-way Catalyst, FI, >50K miles, low power/weight**
5	**3-way Catalyst, FI, >50K miles, high power/weight**
6	**3-way Catalyst, FI, <50K miles, low power/weight**
7	**3-way Catalyst, FI, <50K miles, high power/weight**
8	**Tier 1, >50K miles, low power/weight**
9	**Tier 1, >50K miles, high power/weight**
10	**Tier 1, <50K miles, low power/weight**
11	**Tier 1, <50K miles, high power/weight**
24	**Tier 1, >100K miles**
	Normal Emitting Trucks
12	**Pre-1979 (<=8500 GVW)**
13	**1979 to 1983 (<=8500 GVW)**
14	**1984 to 1987 (<=8500 GVW)**
15	**1988 to 1993, <=3750 LVW**
16	**1988 to 1993, >3750 LVW**
17	**Tier 1 LDT2/3 (3751-5750 LVW or Alt. LVW)**
18	**Tier 1 LDT4 (6001-8500 GVW, >5750 Alt. LVW)**
25	**Gasoline-powered, LDT (>8500 GVW)**
40	**Diesel-powered, LDT (>8500 GVW)**
	High Emitting Vehicles
19	**Runs Lean**
20	**Runs Rich**
21	**Misfire**
22	**Bad catalyst**
23	**Runs very rich**

CMEM is composed of six modules which are: 1) engine power demand; 2) engine speed; 3) fuel/air ratio; 4) fuel-rate; 5) engine-out emissions; and 6) catalyst pass fraction. The model requires two groups of inputs: A) input operation variables; and B) model parameters. With this information, the model produces tailpipe emissions and fuel consumption as outputs. A schematic diagram of the model as shown in the CMEM User's Guide (2) is reproduced in Figure 1. There are four operating conditions in the model, which are shown in Figure 1 as ovals. These operating conditions are: a) variable soak time start; b) stoichiometric operation; c) enrichment; and d) enleanment. The soak time has to be input by the user but the model determines which condition the vehicle is operating at for any given moment (i.e. stoichiometric, enrichment, or enleanment). It does this by comparing the vehicle power demand with two power demand thresholds.

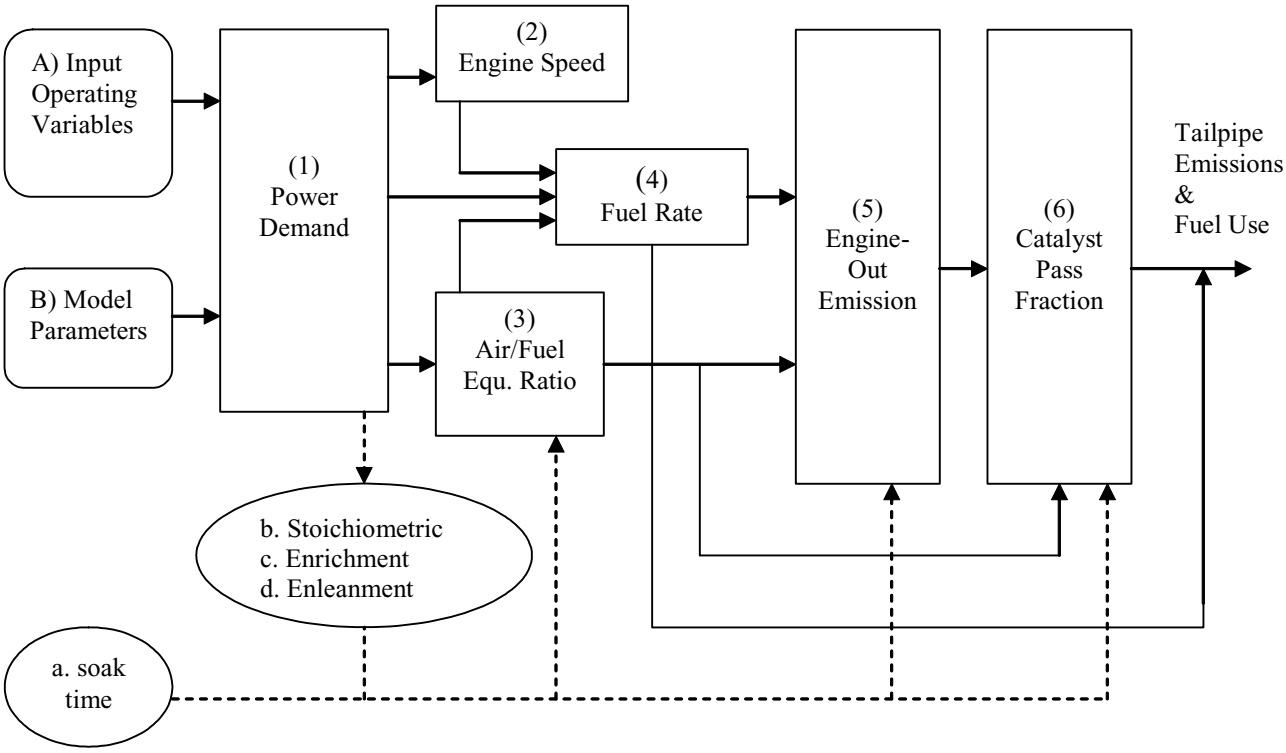

Figure 1. CMEM Structure

Data Collection

Data had to be collected that supplied the specific information needed by the CMEM software. The data collection makes use of three components: 1) a vehicle, 2) a Global Positioning Satellite (GPS) antenna, and 3) a laptop computer. There are several different ways in which the second-by-second speed data can be collected, but the only one described here was the method that was utilized to collect data in Dallas/Ft. Worth TX, area in June and July 2002. In this study, a GPS antenna used in conjunction with a laptop computer was utilized. The antenna that was used to collect the data was a GPS 35/36 TracPak™ manufactured by the GARMIN Corporation. The second-by-second data streams that are collected by this particular GPS package are as follows:

- GPS Fix Data (GPGGA) – includes Universal Time Coordinated (UTC) time of position fix, latitude and longitude data, GPS reception quality, and antenna elevation.
- GPS DOP and Active Satellites (GPGSA) – includes number of satellites used in solution and horizontal, vertical, and position dilution values.
- GPS Satellites in View (GPGSV) – includes number of sentences to be transmitted, number of satellites in view, and satellite information.
- Recommended Minimum Specific GPS/Transmit Data (GPRMC) – includes UTC time of position fix, latitude and longitude data, speed course, and magnetic variation.

The primary data used by the CMEM software is the GPRMC data stream. This raw data that is collected has to be manipulated before it can be input into the model. The data reduction process will be described later.

A laptop computer was used for the collection of this data because of its compactness and ease of use which is needed while operating a vehicle. The laptop was equipped with specific pieces of software to

effectively collect the data. Three different software applications were used and are described in the order they were used to collect the data.

- NMEA Time – is software that allows the computer's internal clock to be set with the atomic standard time. A signal is received through the GPS antenna that updates the time.
- Maptitude – is a Geographic Information System (GIS) software program that makes it possible to compile a GPS playback file. The GPS playback file records the GPS data that is received from the antenna. This recorded data is saved as a comma-delimited text file.
- Runstart – is a FORTRAN program that was written by researchers at the Texas Transportation Institute (TTI). This program marks the data file to identify where the true data collection begins and ends. This type of program is needed because the Maptitude GPS playback file begins recording second-by-second data as soon as the file is created. The Runstart program allows the user to give it a file name, mark where the actual data collection begins and ends, and the user can mark intermediate points of reference anywhere along the route. When data is entered into the Runstart program, the data file time stamps it. This time stamp matches with the corresponding time in the GPS playback file.

Data Reduction Process

The GPS playback file that is created during the data collection step includes more information than is needed for input in the CMEM model software. Therefore, it is necessary to reduce the raw data into a format that can be used by CMEM. There are two issues that have to be resolved before this can occur: remove the inapplicable data and get the reduced data into a form that is usable by the CMEM model. The following provides the procedure steps, in order, that were used to reduce the raw data.

- The first step in the data reduction is to remove the extra data collected by the GPS antenna. The only data that will be needed by CMEM is the speed data; all of the other data lines need to be deleted.
- Second, the Runstart program that corresponds to the data file was opened to find the times when the true data collection began and ended. These times were matched to the beginning and ending times, respectively, of the reduced GPS playback file. All of the data in the playback file that was recorded before the beginning time or after the ending time was deleted. What was left was a series of speed data lines that showed only the true data collection.
- The single column of speed data that is extracted from the playback file has units of knots. This was converted to miles per hour (MPH) by using the conversion of 1 knot = 1.150779 mph. It is possible to use kilometers per hour (KPH) for input into the CMEM model but it was decided to use MPH for this study.
- The last step in the data reduction process is to assign sequential numbering to the column of speed data beginning with 1 and continuing to the last speed data entry. This file was then saved as a comma-delimited ascii text file.

Data Processing

CMEM was designed so it could process data by two different methods: the core model and the batch model. The core model was used for the purposes of this study. Also, the model can be run in a DOS or a Microsoft Windows operating system. The DOS version of the software was used for this evaluation. To process the two required input files a command line argument had to be entered. A core model command line argument would look similar to:

cmemcore example-ctr example.csv

This command line argument provides three pieces of information. It identifies the type of model; in this case it was the core model (cmemcore). Second, it provides the names of the two input files needed. In the example shown above they are "example-ctr" and "example.csv". The first input file is the control file. It contains specific vehicle information as to the vehicle category. The second control file is the vehicle activity file, which is a column-oriented data set. The minimum required data for this input file is the time, in seconds, and the vehicle speed, in mph or kph depending upon how the control file is setup. After pressing the "enter" key, the CMEM software will produce its two output files with extensions *.sbs and *.sum. The *.sbs file gives the second-by-second emissions output and the *.sum file provides summary information such as fuel use and emissions in units of grams-per-mile (grams/mile). An example of the two CMEM output files is located in Appendix B. Figure 2 shows a graphical representation of the of the core model.

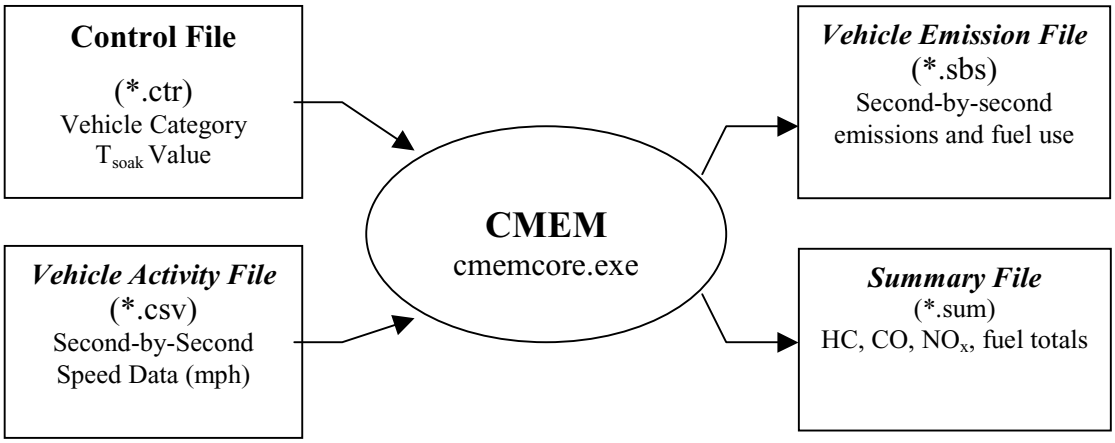

Figure 2. CMEM Core Model Input/Output

RESULTS

TTI collected data in the Dallas/Ft. Worth metropolitan area over a span of six weeks in June and July of 2002. Over thirty (30) hours of speed data was collected during this time period and approximately eight hundred (800) miles were covered. This data was collected during both peak and off-peak time periods for each of the study locations. A description of the findings will be discussed in the following section.

It is important to note that the results that are output from CMEM are the emissions and fuel usage for one vehicle. All of the following results depict emissions modeled as a category 10 vehicle, a tier 1 vehicle with less than 50,000 miles and a low power to weight ratio. A soak time (T_{soak}) of zero (0) minutes was input into the model to develop these results. Using CMEM, it is possible to expand the data to estimate emissions and fuel usage for the total diverse traffic volume of a roadway, but this was not carried out in this study.

Freeway Routes in the Peak and Off-peak Periods

Data was collected on five (5) freeway sections in an effort to better understand the emissions produced on congested freeways. This data was collected in the same direction during the peak and off-peak travel periods. In an effort to assure the validity of the data, seven to eight replicates of data collection was

utilized for each freeway section for both the peak and off-peak periods. Figure 3 has the freeway sections highlighted. The freeway sections studied are as follows:

- **Route 1** – This section consisted of a portion of Loop 12 starting at State Road 356 and ending where Loop 12 merged with I-35E. Data was collected in the northbound direction during the morning peak period. The off-peak data was collected in the northbound direction also, but only after the peak period had expired. The one-way length of this section was approximately 4.3 miles.
- **Route 2** – For this section data was collected on the ramp that connected eastbound I-30 to northbound I-35E, which is the morning peak direction. The section started approximately 1.5 miles west of the ramp on I-30. The end of the section was on I-35E at Oak Lawn. This section was approximately 4.6 miles in length.
- **Route 3** – Data was collected on southbound State Highway 360 approaching I-30, which is the afternoon peak direction. The section started on SH 360 at Midway, which is just north of SH 183, and it ended at SH 360 and Abram St, which is south of I-30. The length of this section was approximately 6.4 miles.
- **Route 4** – This section consisted of a stretch of northbound Highway 75 approaching I-635, which is the afternoon peak direction. The starting point was Park Lane and the section ended north of I-635 at Spring Valley Rd. The approximate length of this section was 4.3 miles.
- **Route 5** – This route consisted of a section of westbound I-635 approaching I-35E. This is the afternoon peak direction. The route started on I-635 at Marsh Lane (Exit 24) and ended at Luna Rd. (Exit 29). The approximate length of this route was 2.7 miles.

For all of these locations traffic flow during the off-peak period was free-flow. Some of the locations, such as Route 4 and Route 5, had high volumes of traffic during the off-peak periods but the speeds did not drop considerably. There were a few times during the off-peak period when speeds were reduced lower than the posted speed limits because of debris in the roadway, or an abandoned or broken-down automobile. If there was something unusual that drastically slowed traffic, such as a collision, then that data was discarded and not used to develop these results. All of these locations had heavy traffic and average speeds that were well below the posted speed limits during their respective peak periods. Often traffic was stop-and-go during the peak period.

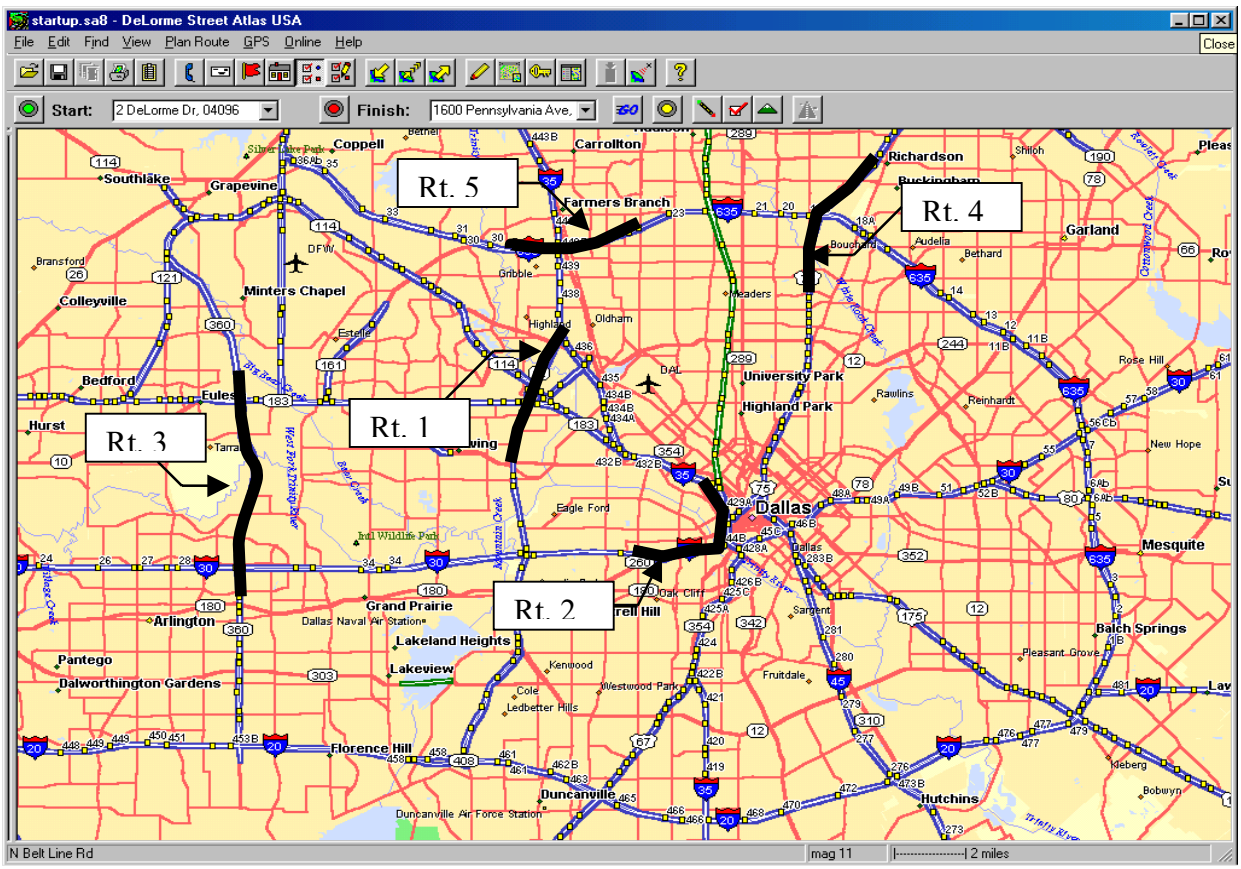

Figure 3. Map of Freeway Sections Studied

Emission Analysis

All five (5) routes showed an average speed reduction from the off-peak to peak period with Route 4 having the largest speed reduction of more than sixty-four (64) percent. The average speed reduction of all the routes was forty-two (42) percent from the off-peak period to the peak period. All the routes also showed an increase in fuel use from the off-peak to peak period. Route 1 had only a minimal increase of fuel use of three (3) percent but Route 4 had a more substantial increase of fuel use of over forty-three (43) percent. On average, fuel use increased twenty-four (24) percent from the off-peak to the peak period. Table 2 shows that the emissions and fuel usage levels associated with each location are significantly more for the peak period than for the off-peak period.

The difference in emissions output between peak and off-peak periods for the five freeway routes is relatively consistent. On average, HC emissions increased thirty-seven (37) percent from the off-peak period to the peak period. CO emissions increased ninety-three (93) percent, and NO_x emissions increased seven (7) percent on average. Route 1 showed increases in emission levels for the peak period over the off-peak period for HC and CO, but not for NO_x. For Route 1, HC emissions increased fifty (50) percent and CO increased one hundred thirty-four (134) percent. Route 2 had increases in emissions for the peak period over the off-peak period of thirty-seven (37) percent for HC, sixty-six (66) percent increase in CO, and eleven (11) percent for NO_x. For Route 3 CO, HC, and NO_x emissions increased thirty (30), seventy-seven (77), and seven (7) percent, respectively for the peak period over the off-peak period. Route 4 showed emissions increases of twenty-nine (29) percent for HC, seventy-two (72) percent for CO, and seven (7) percent for NO_x. For Route 5, HC emissions increased forty-two (42)

percent, CO emissions increased one hundred fifty-six (156) percent, and NO_x emissions increased a moderate seven (7) percent. Figure 4 depicts the peak to off-peak period comparison of HC emission for the five (5) freeway routes. Figure 5 and Figure 6 show the peak to off-peak period comparisons of CO and NO_x emissions, respectively, for the five (5) freeway routes. All emissions are shown in units of grams/mile. Appendix A contains graphs that compare speed and emissions, at the second-by-second level, for all the freeway routes.

Table 2. Freeway Section Summary Information

Route	Section Length (miles)	Direction	Avg. Speed (mph)	Emissions (grams/mile)			Fuel Usage (grams/mile)
				HC	CO	NO_x	
1	4.3	Peak	49	0.054	4.33	0.16	82.4
		Off-Peak	61	0.036	1.85	0.16	79.9
2	4.6	Peak	30	0.074	6.54	0.20	106.4
		Off-Peak	51	0.054	3.93	0.18	82.0
3	6.4	Peak	26	0.035	1.54	0.14	98.8
		Off-Peak	61	0.027	0.87	0.13	75.2
4	4.3	Peak	21	0.040	2.13	0.15	110.1
		Off-Peak	59	0.031	1.24	0.14	76.7
5	2.7	Peak	42	0.043	2.66	0.15	85.5
		Off-Peak	59	0.031	1.04	0.14	76.7

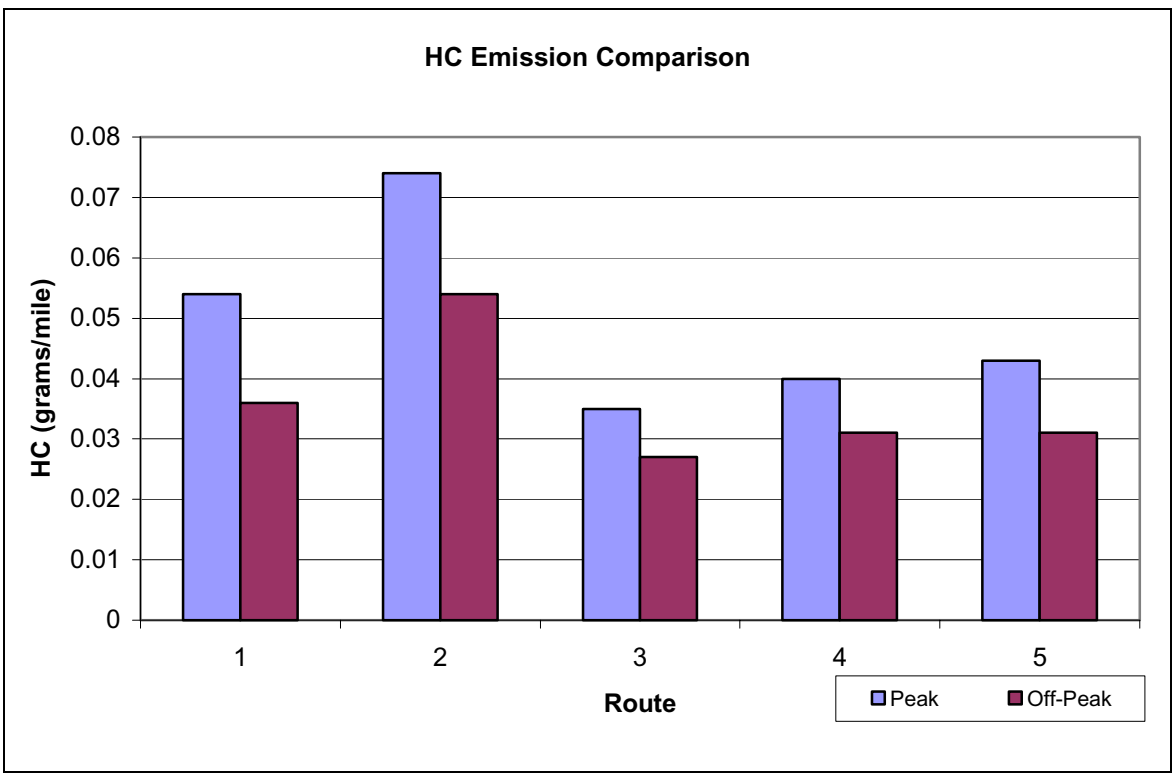

Figure 4. HC Emissions Comparison of Peak/Off-Peak Period

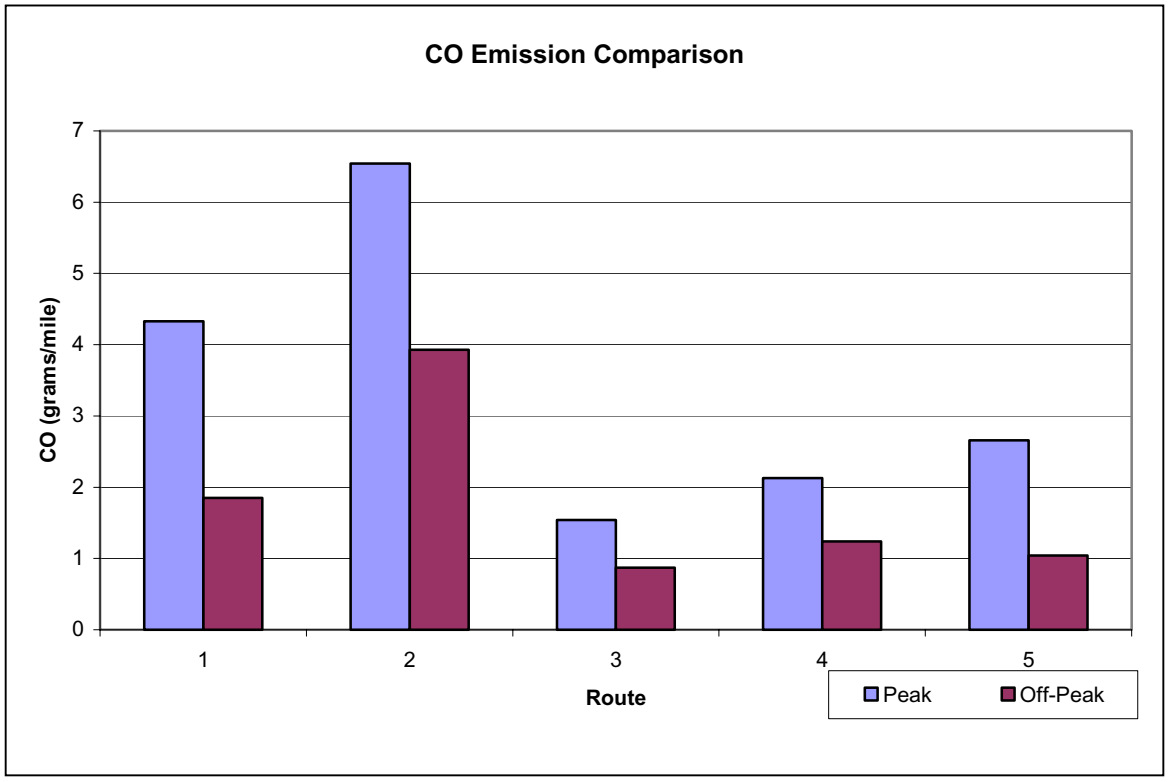

Figure 5. CO Emissions Comparison of Peak/Off-Peak Period

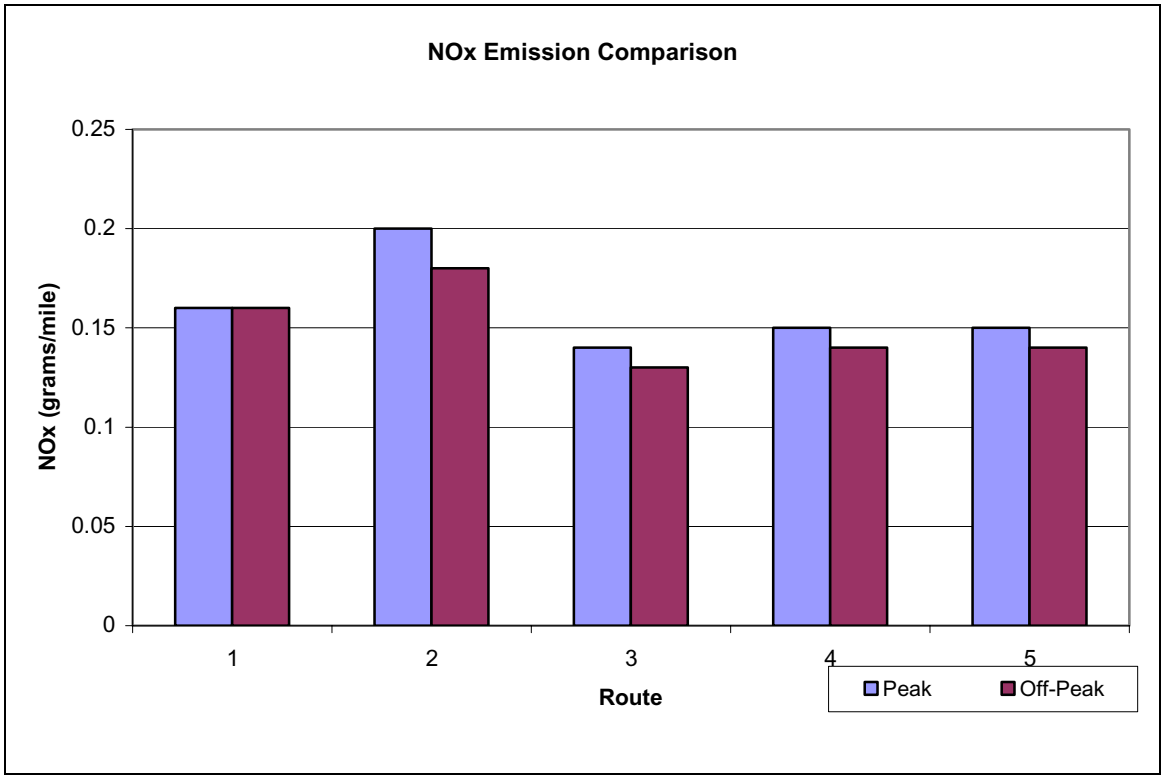

Figure 6. NO_x Emissions Comparison of Peak/Off-Peak Period

CONCLUSIONS

The CMEM software was developed out of a need for a useful tool that would estimate second-by-second tailpipe emissions and fuel usage of LDV's. The software was used to estimate the emissions and fuel usage for only one vehicle in this study; however, it is possible to expand the data to approximate the emissions of a larger population of vehicles. It was shown in the previous sections by using the CMEM software to compare peak and off-peak periods that congestion does cause an increase in emissions output and also increases fuel use. On average, for the five (5) freeway sites researched in this study, peak period emissions were greater than off-peak period emissions. HC emissions increased thirty-seven (37) percent, CO emissions increased ninety-three (93) percent, NO_x emissions increased seven (7) percent, and fuel use was increased by twenty-four (24) percent.

Implementing CMEM

The CMEM software is still relatively new and not widely used, but there are many situations that could gain an advantage by using this model. It could be used in before and after studies to estimate how much an improvement measure, such as a grade separation or adding additional lanes to a facility, reduced emissions. This will be the best method to use the model to determine the measure's effectiveness in terms of emissions output and fuel consumed. The model could also be used to predict if a future project will have an impact on emissions. This can be done by collecting data on the current facility and comparing it to a separate facility that is similar to what the completed project would be. This type of implementation of the model may be less accurate than a before and after study. This comparison between sites could be less accurate because there are often site-specific elements (e.g. geometric layout, signal timing, high/low traffic generating businesses, etc.) that would not allow for a true comparison.

Recommendations

To get results that are accurate as possible, it is necessary to expand the data when using the CMEM software to approximate the entire vehicle volume of a facility. This will give an output of total emissions and fuel usage for the overall diverse vehicle volume of a facility. Traffic counts and vehicle distribution types, or fleet mix, can be used to approximate the number of vehicles in each CMEM vehicle category. Then the speed data, and vehicle category, can be input into the model for all of the vehicle categories. The CMEM output files' information can be used along with the known approximate number of vehicles in each class to produce an emissions inventory for the entire facility. It is also recommended that data be collected using multiple replicates for each facility due to the microscopic scale of the model and the varying speeds of the test facility. Small changes in speed or acceleration rates can be averaged over multiple replicates to depict a more accurate representation of the conditions on the facility.

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APPENDIX A

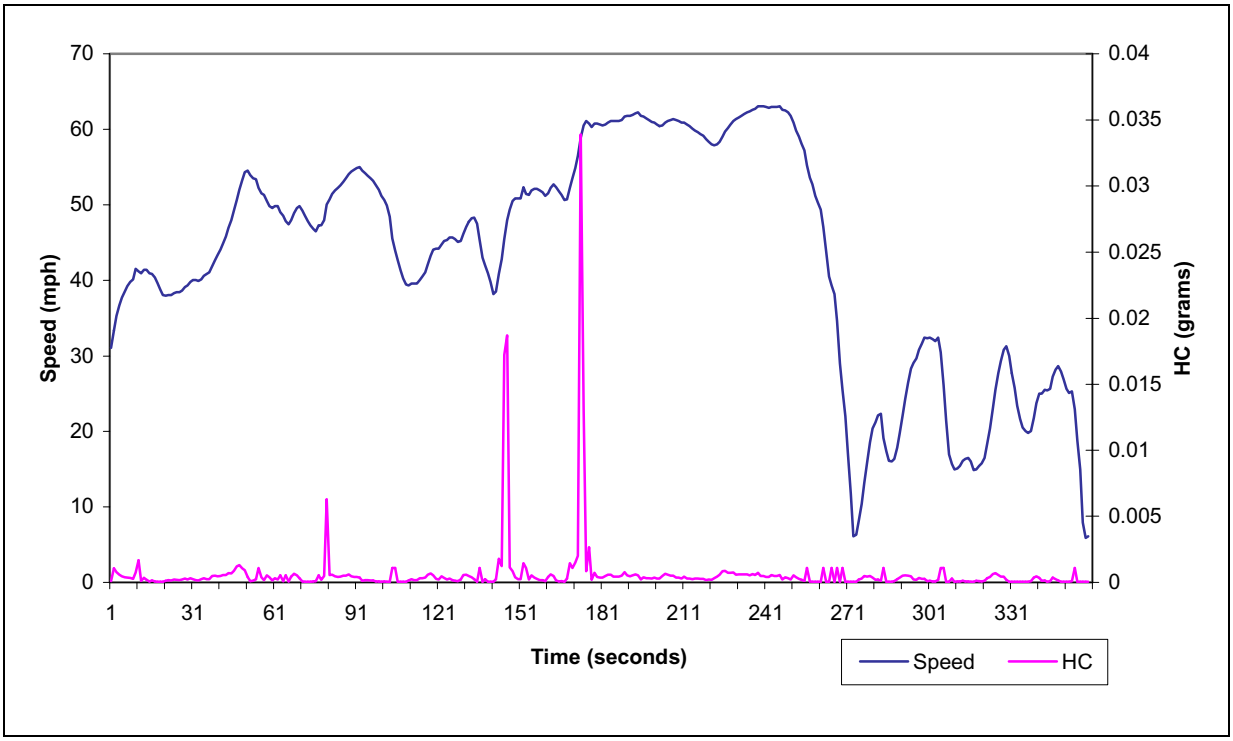

Figure 1a. Route 1 Speed-HC Comparison (Peak Period)

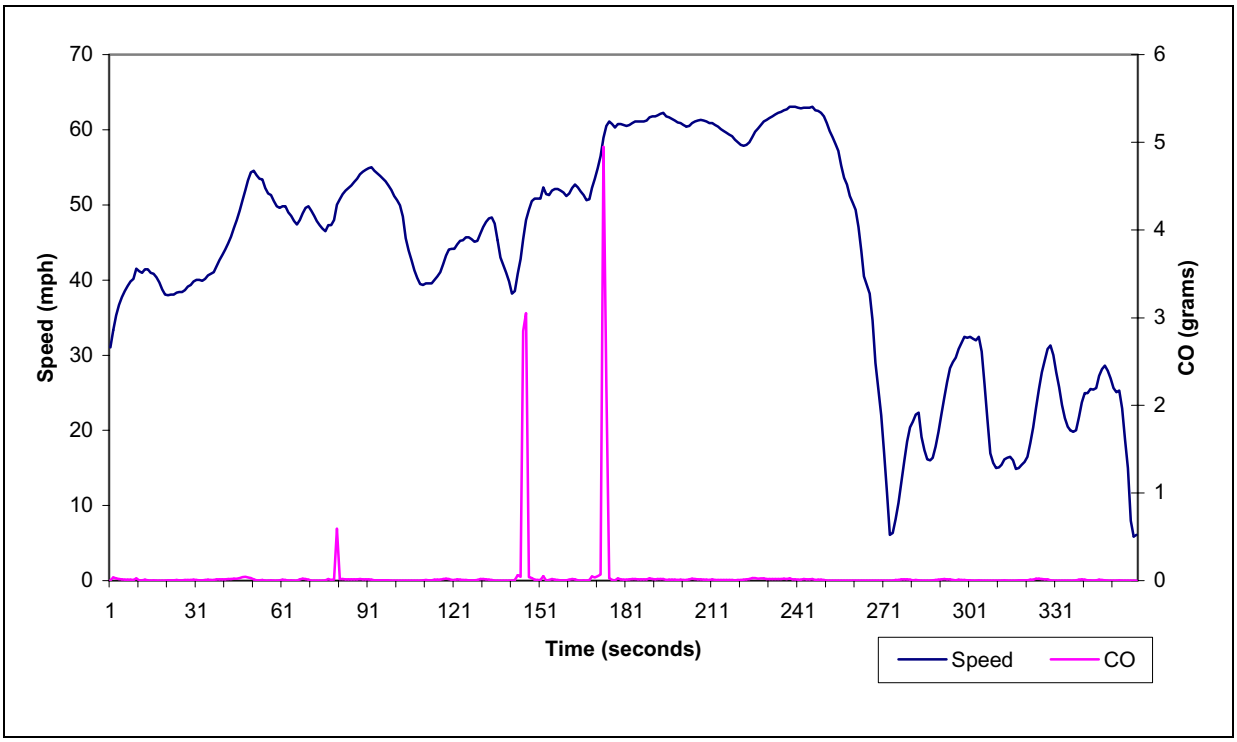

Figure 2a. Route 1 Speed-CO Comparison (Peak Period)

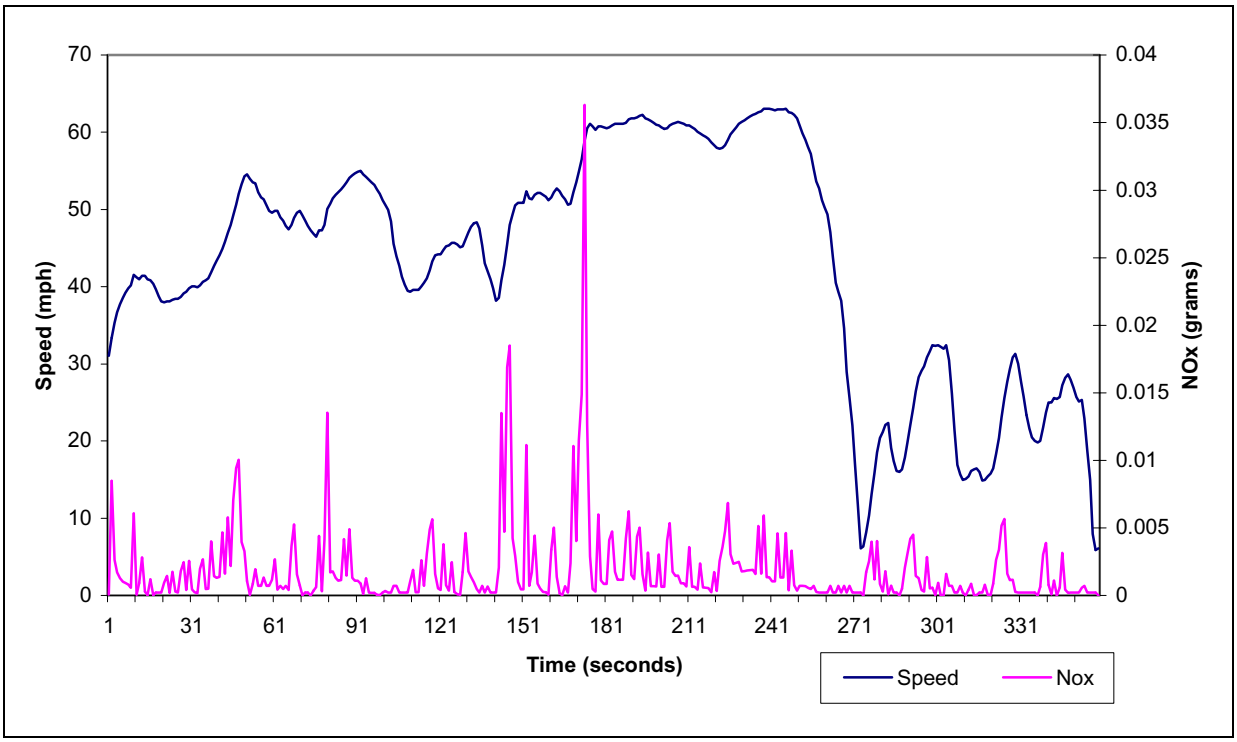

Figure 3a. Route 1 Speed-NO_x Comparison (Peak Period)

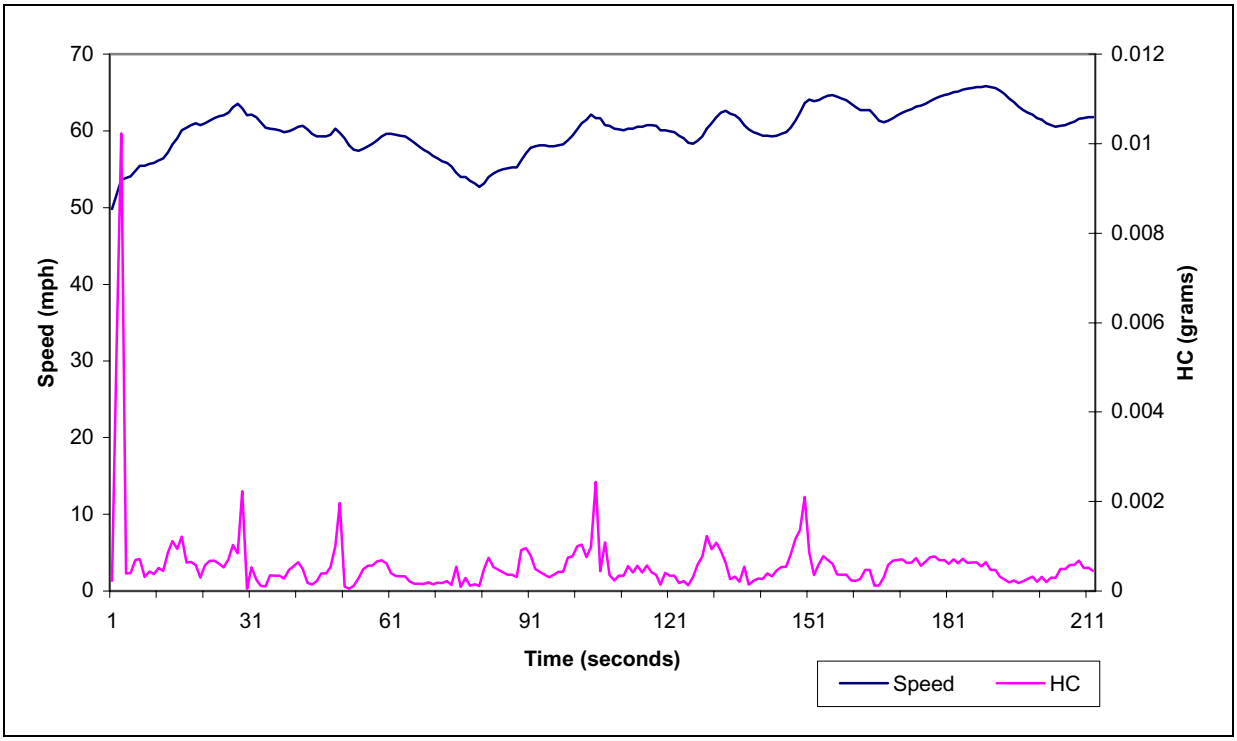

Figure 4a. Route 1 Speed –HC Comparison (Off-Peak Period)

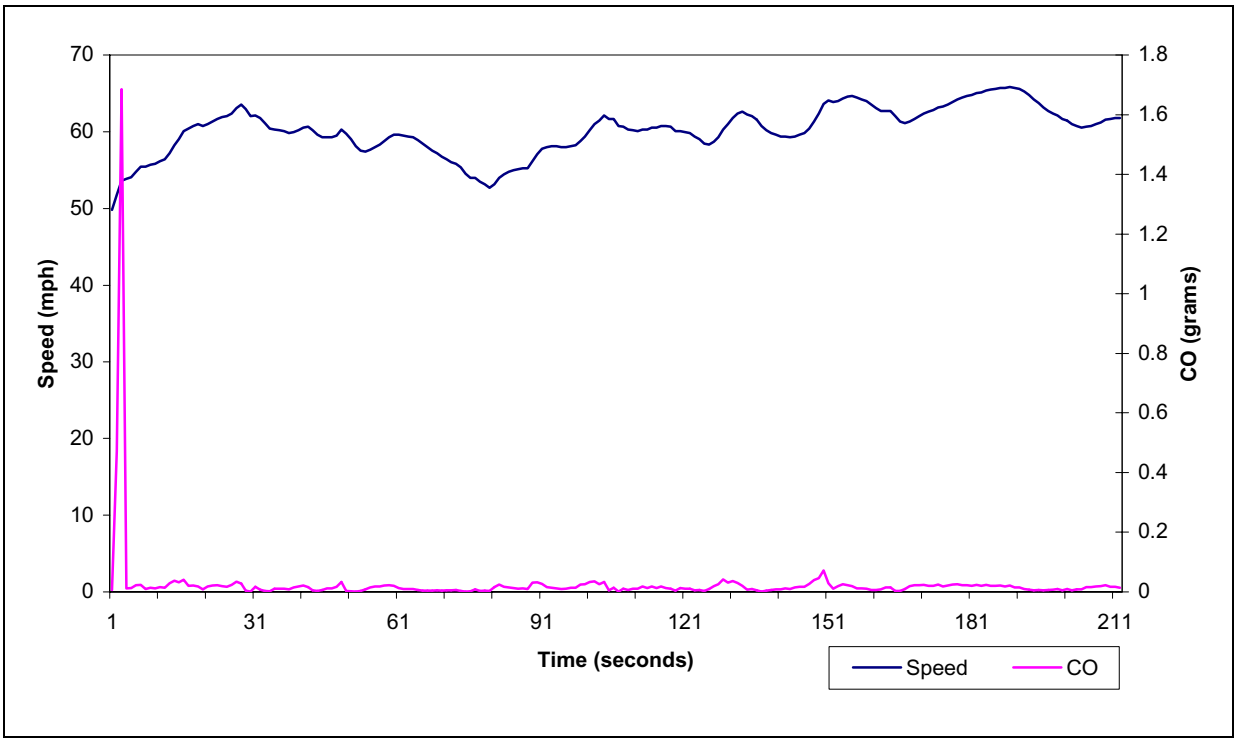

Figure 5a. Route 1 Speed-CO Comparison (Off-Peak Period)

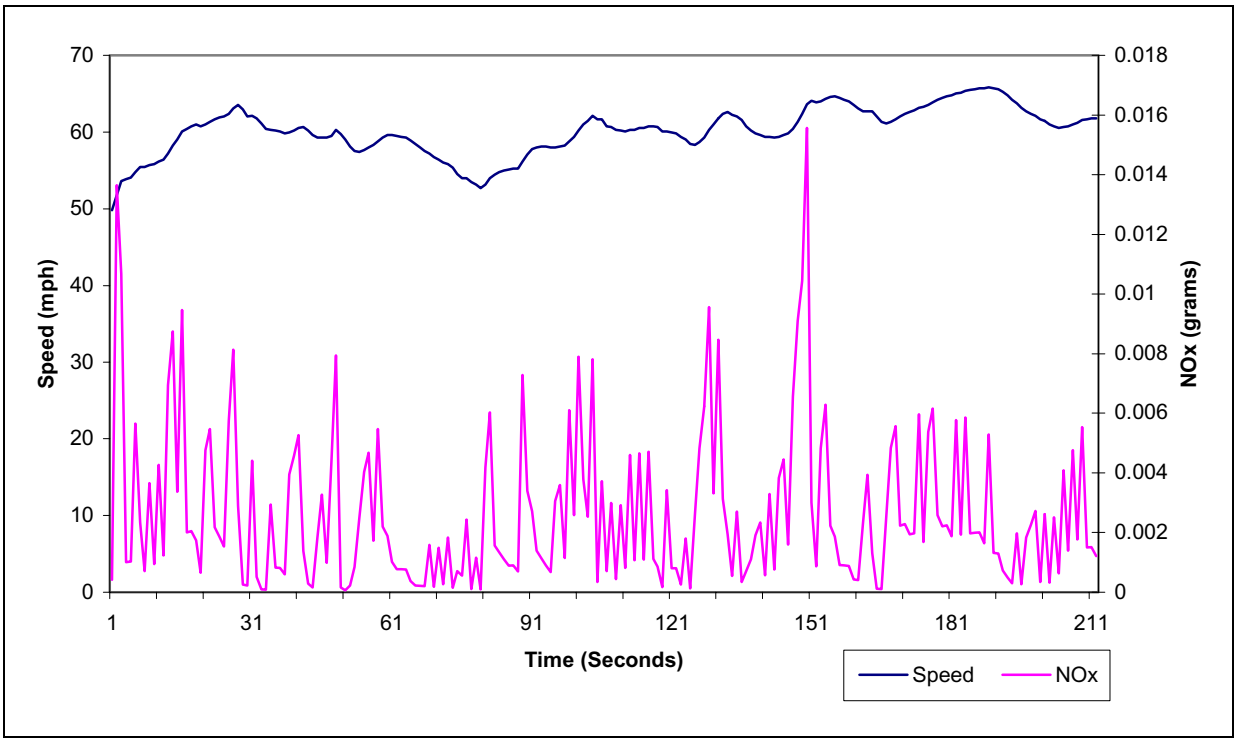

Figure 6a. Route 1 Speed-NO_x Comparison (Off-Peak Period)

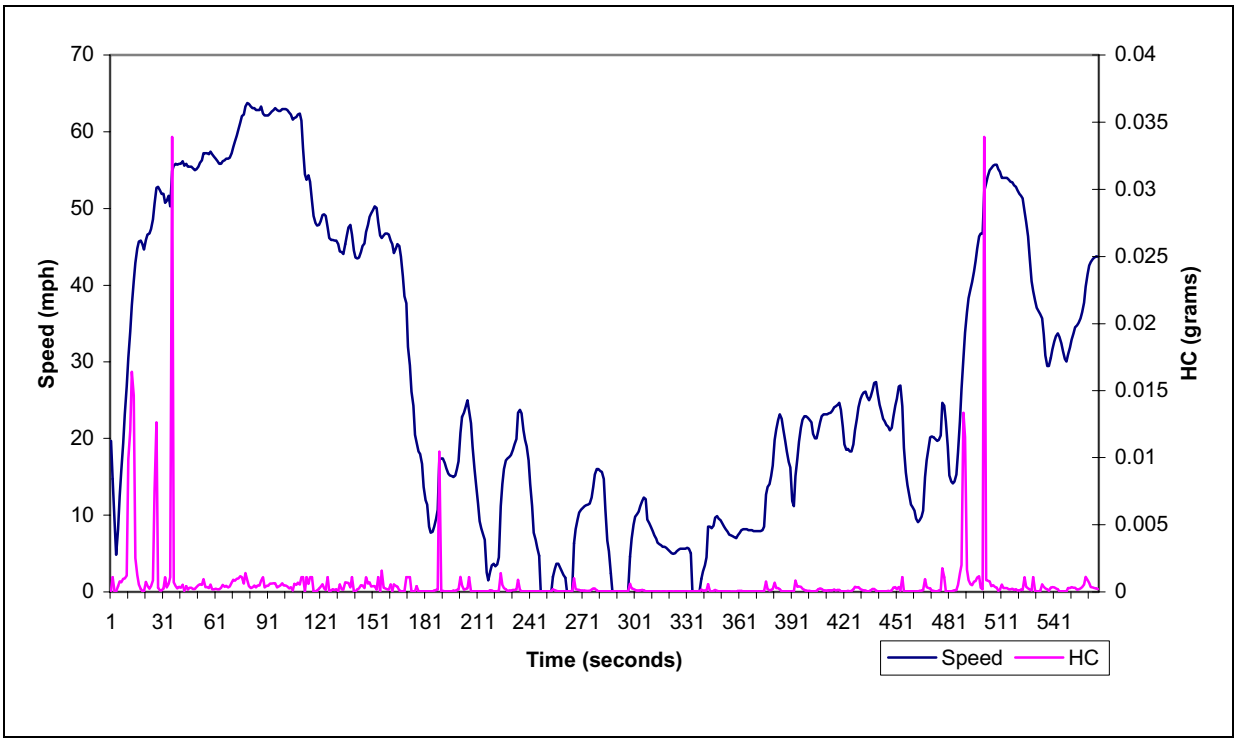

Figure 7a. Route 2 Speed-HC Comparison (Peak Period)

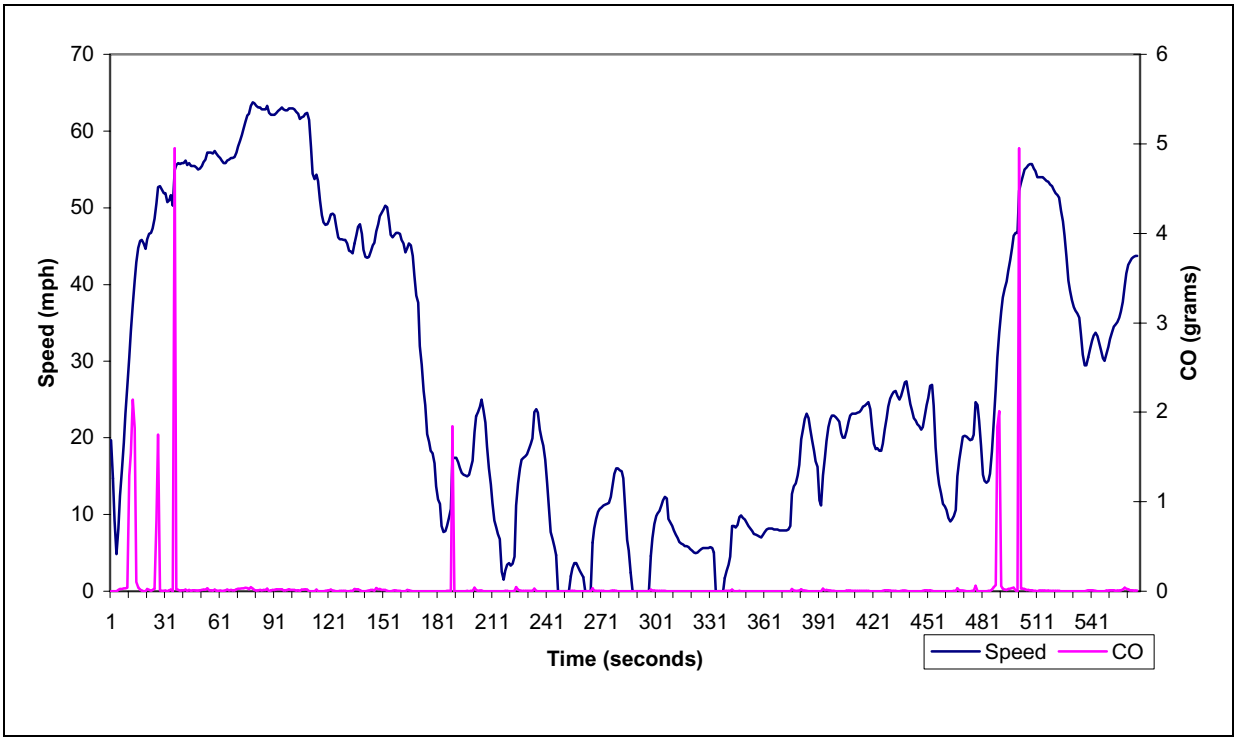

Figure 8a. Route 2 Speed-CO Comparison (Peak Period)

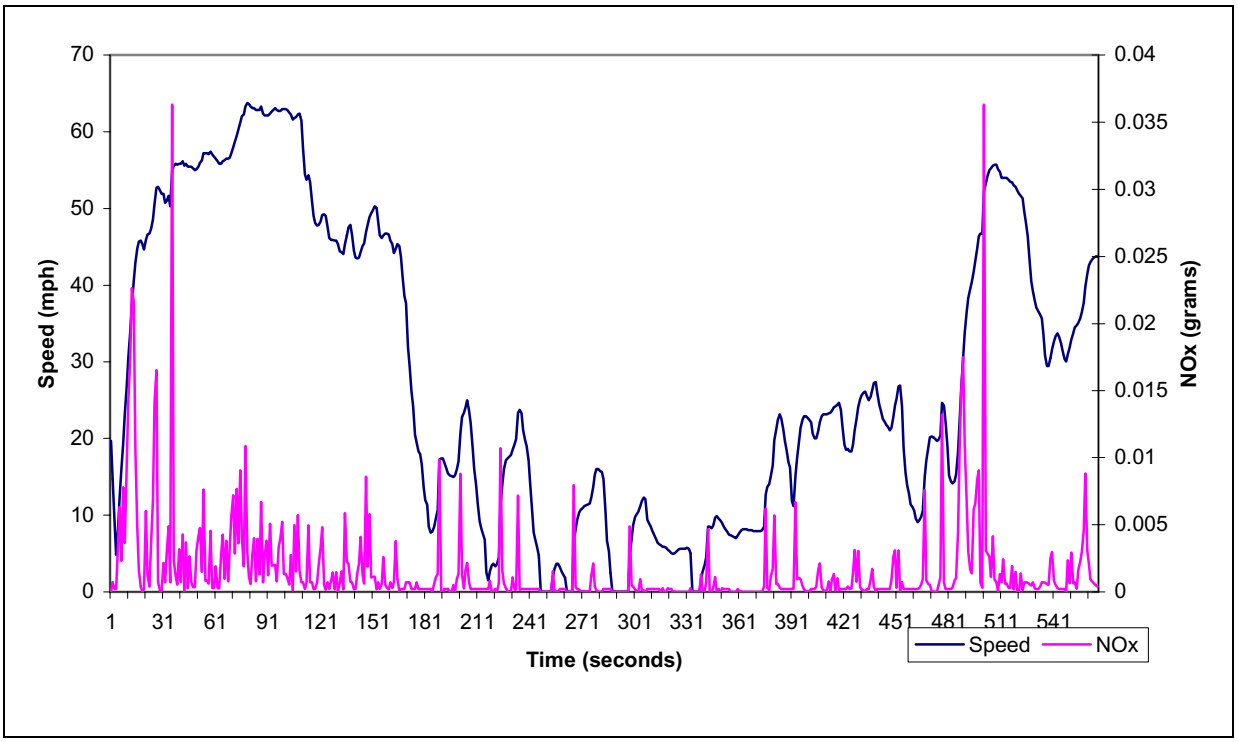

Figure 9a. Route 2 Speed-NO_x Comparison (Peak Period)

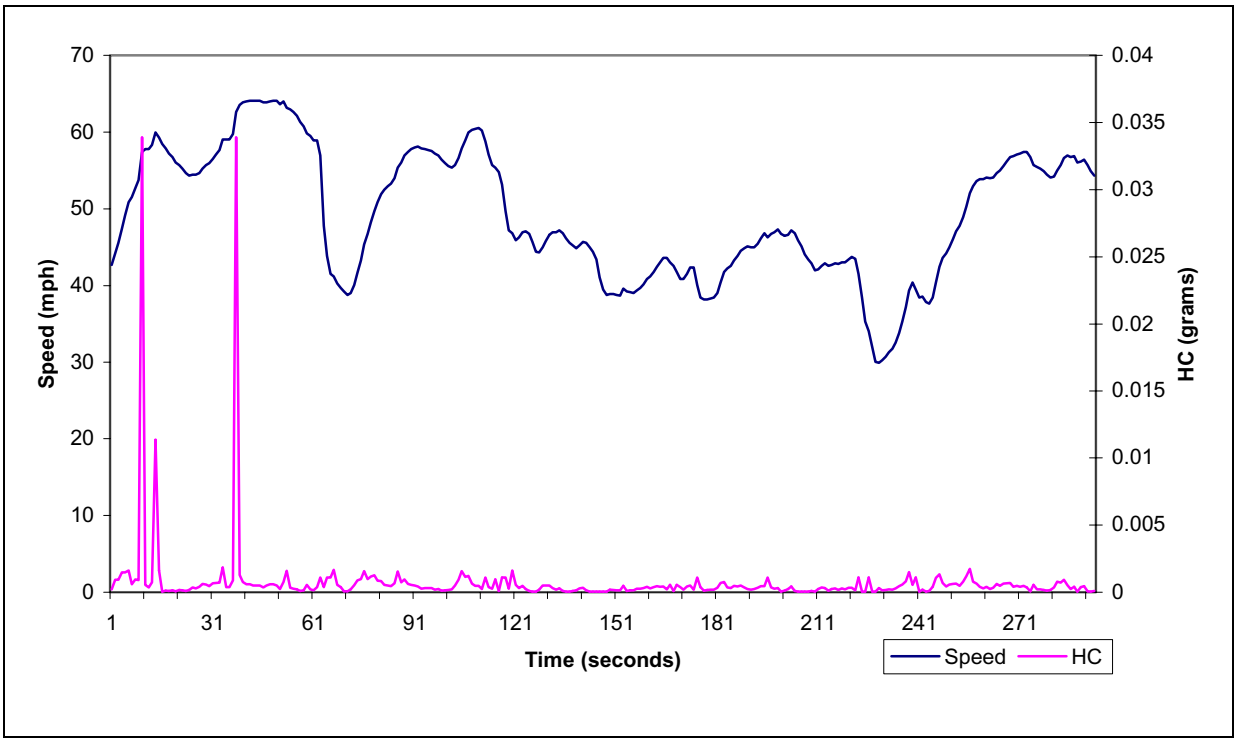

Figure 10a. Route 2 Speed-HC Comparison (Off-Peak Period)

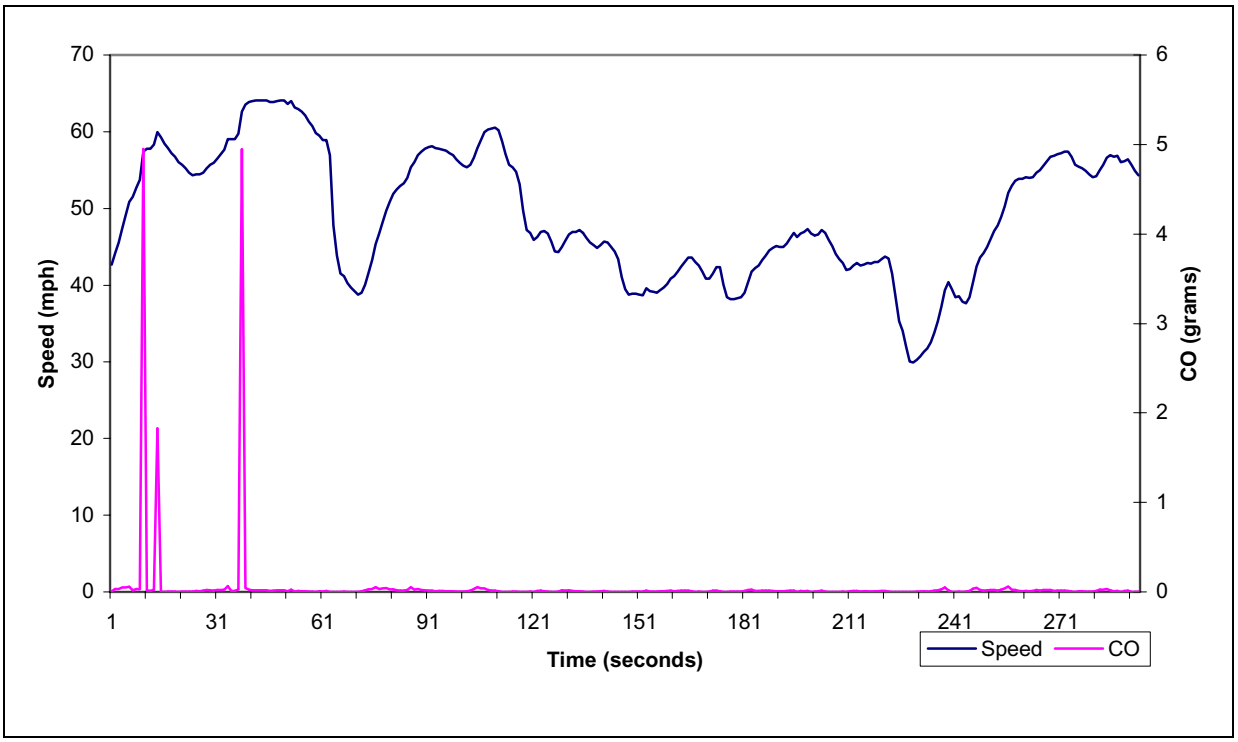

Figure 11a. Route 2 Speed-CO Comparison (Off-Peak Period)

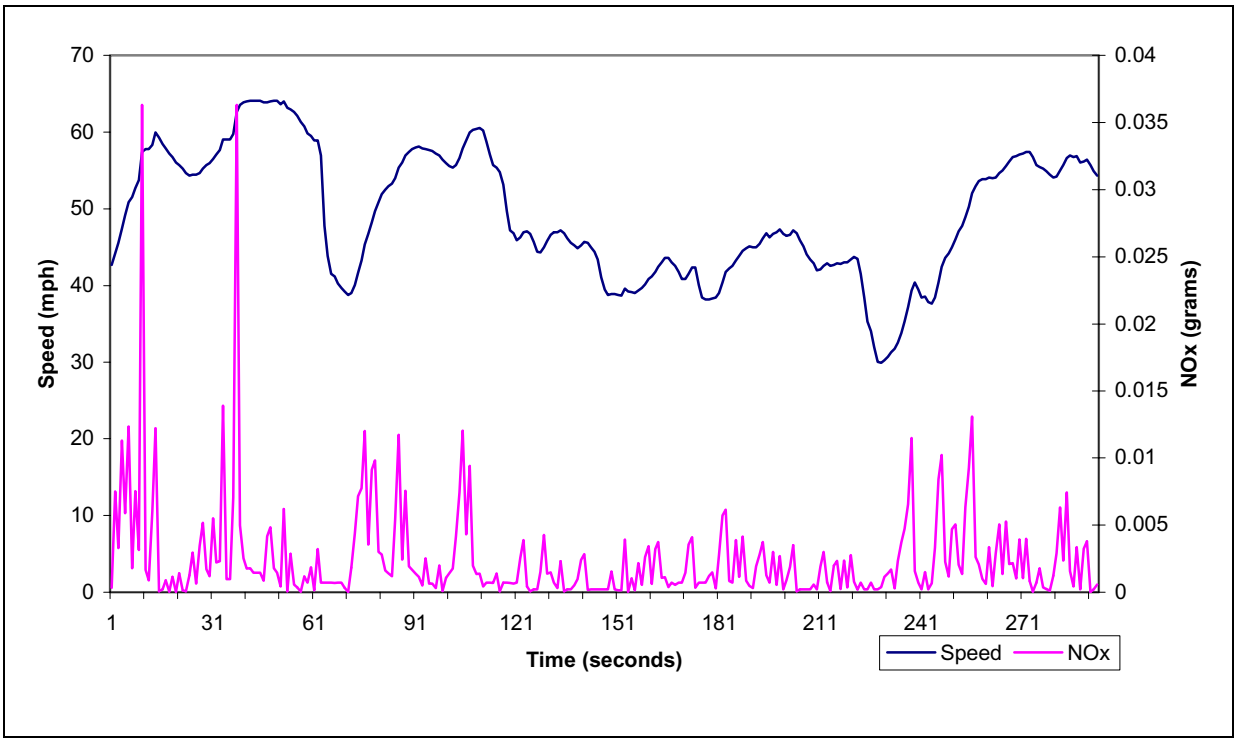

Figure 12a. Route 2 Speed-NO_x Comparison (Off-Peak Period)

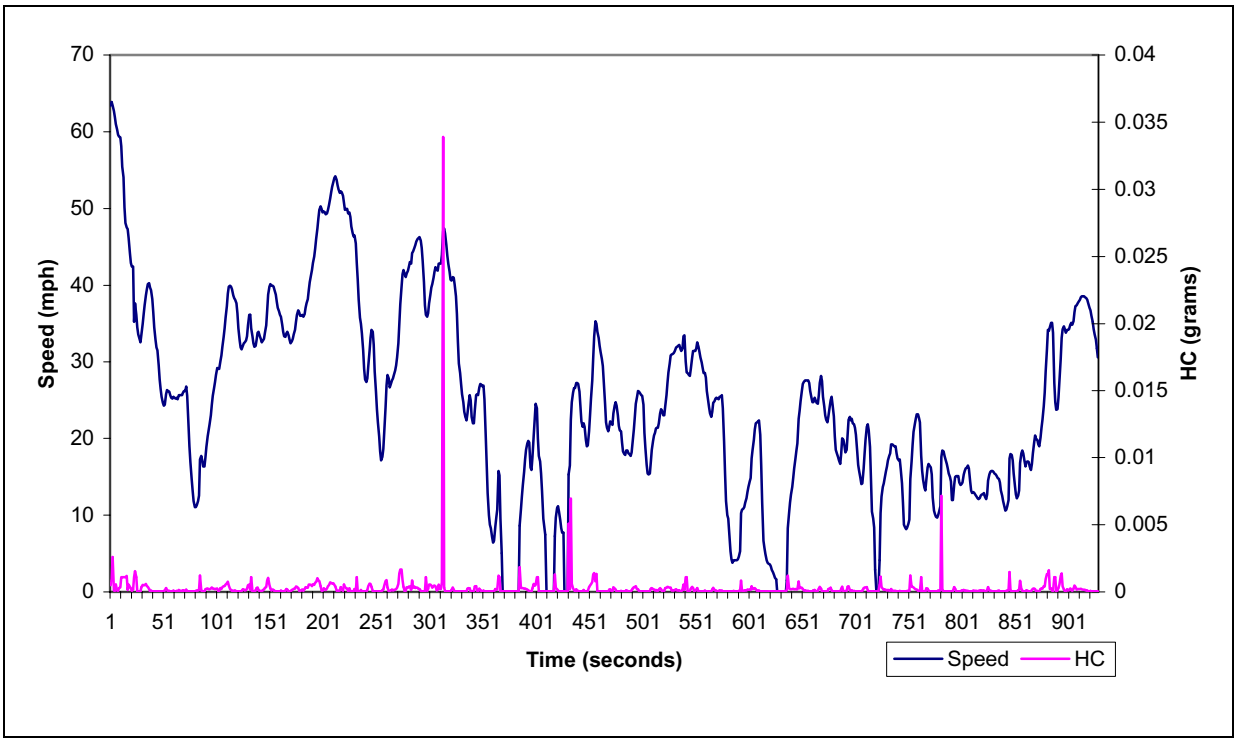

Figure 13a. Route 3 Speed-HC Comparison (Peak Period)

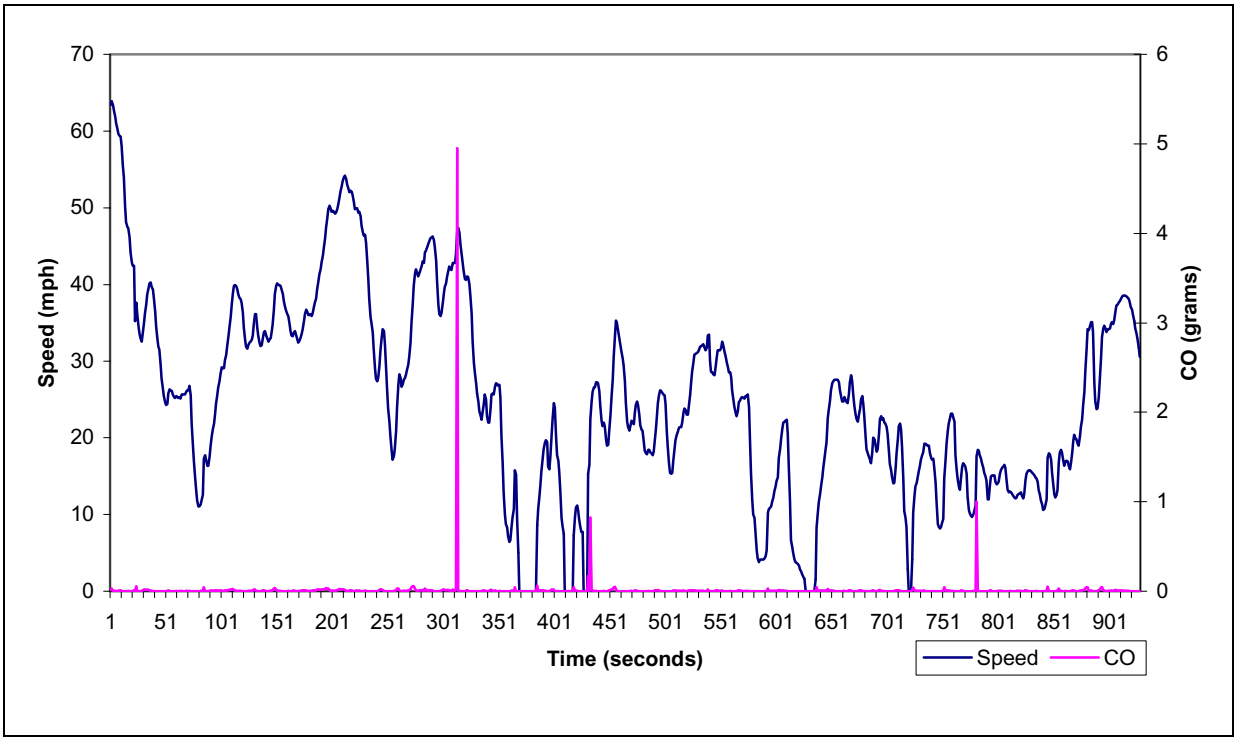

Figure 14a. Route 3 Speed-CO Comparison (Peak Period)

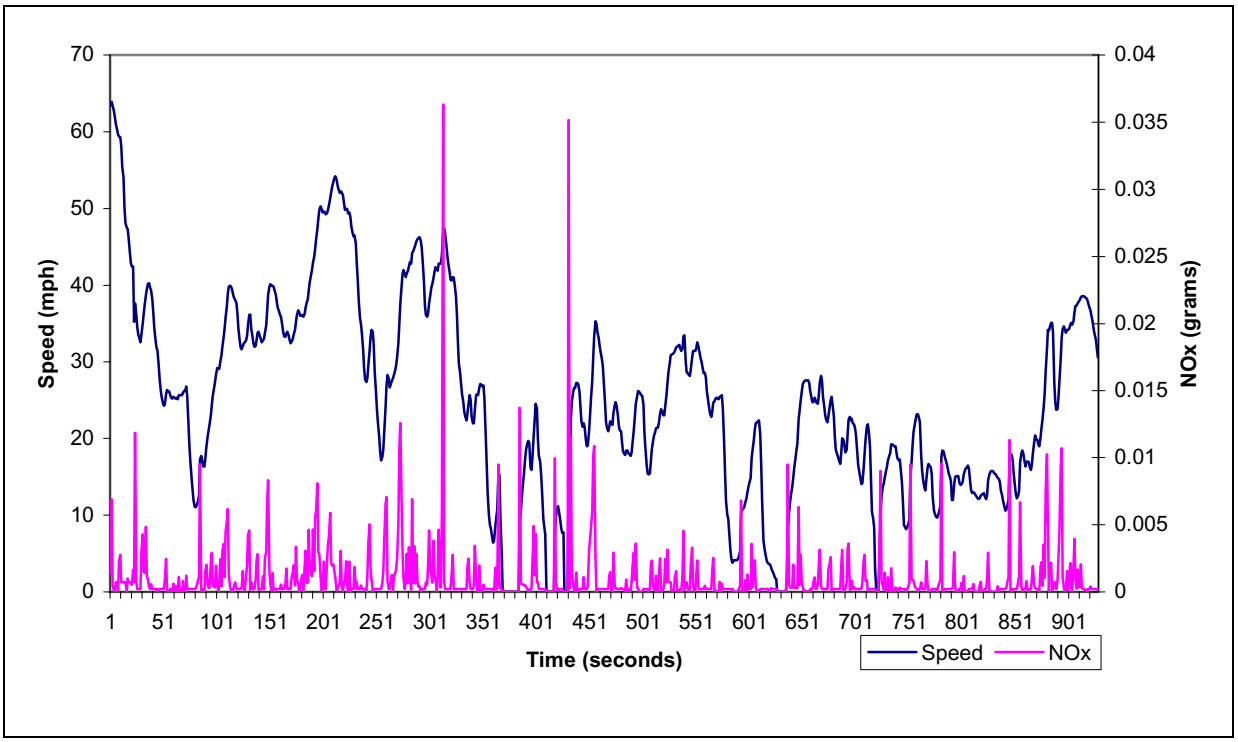

Figure 15a. Route 3 Speed-NO_x Comparison (Peak Period)

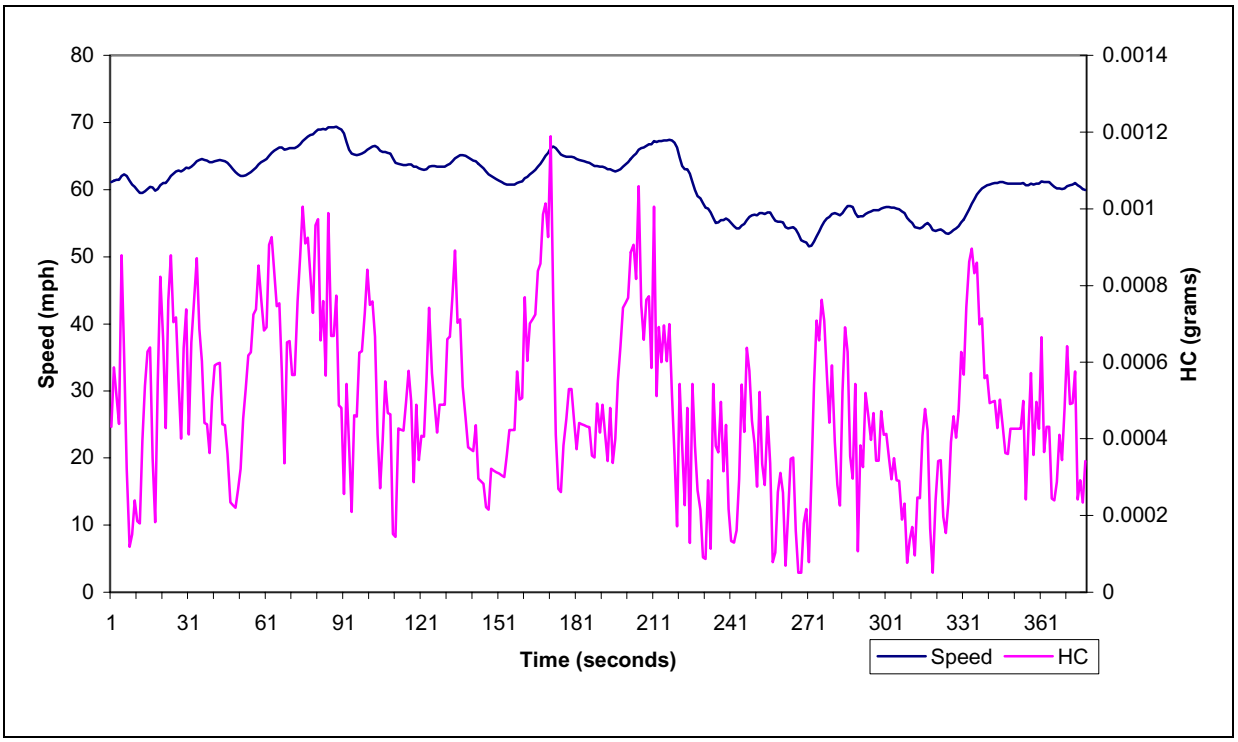

Figure 16a. Route 3 Speed-HC Comparison (Off-Peak Period)

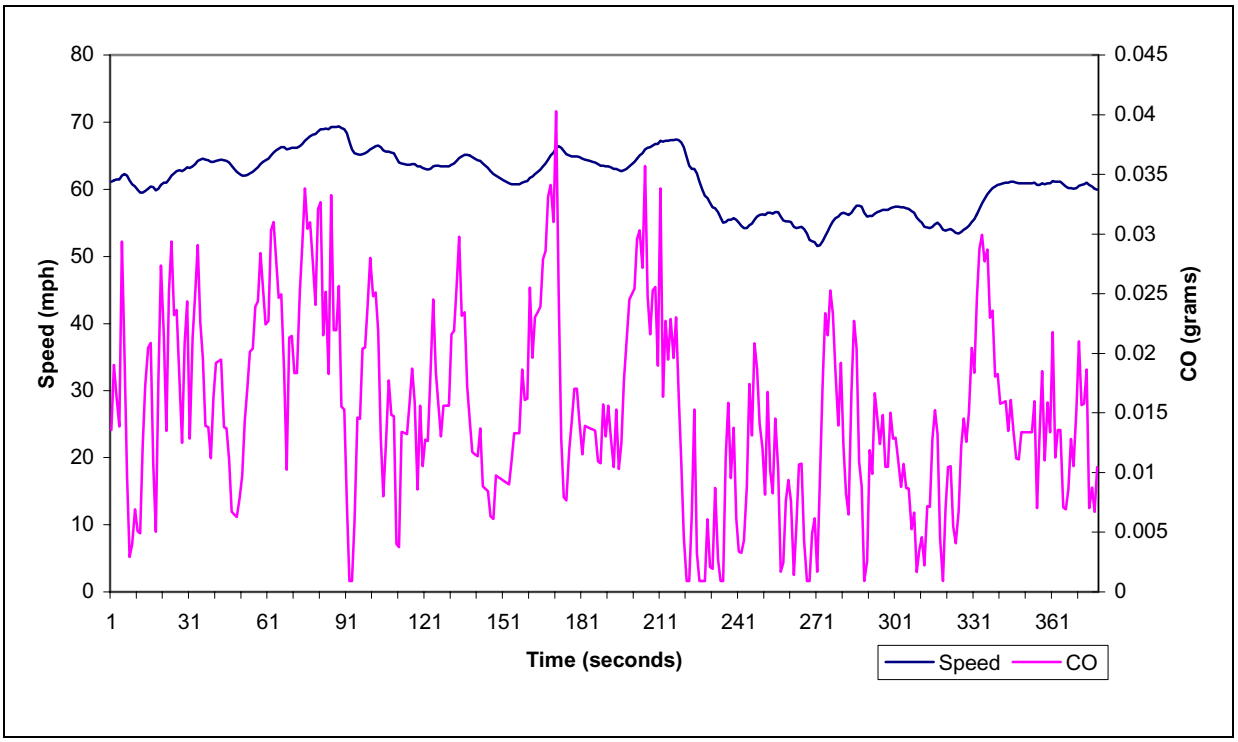

Figure 17a. Route 3 Speed-CO Comparison (Off-Peak Period)

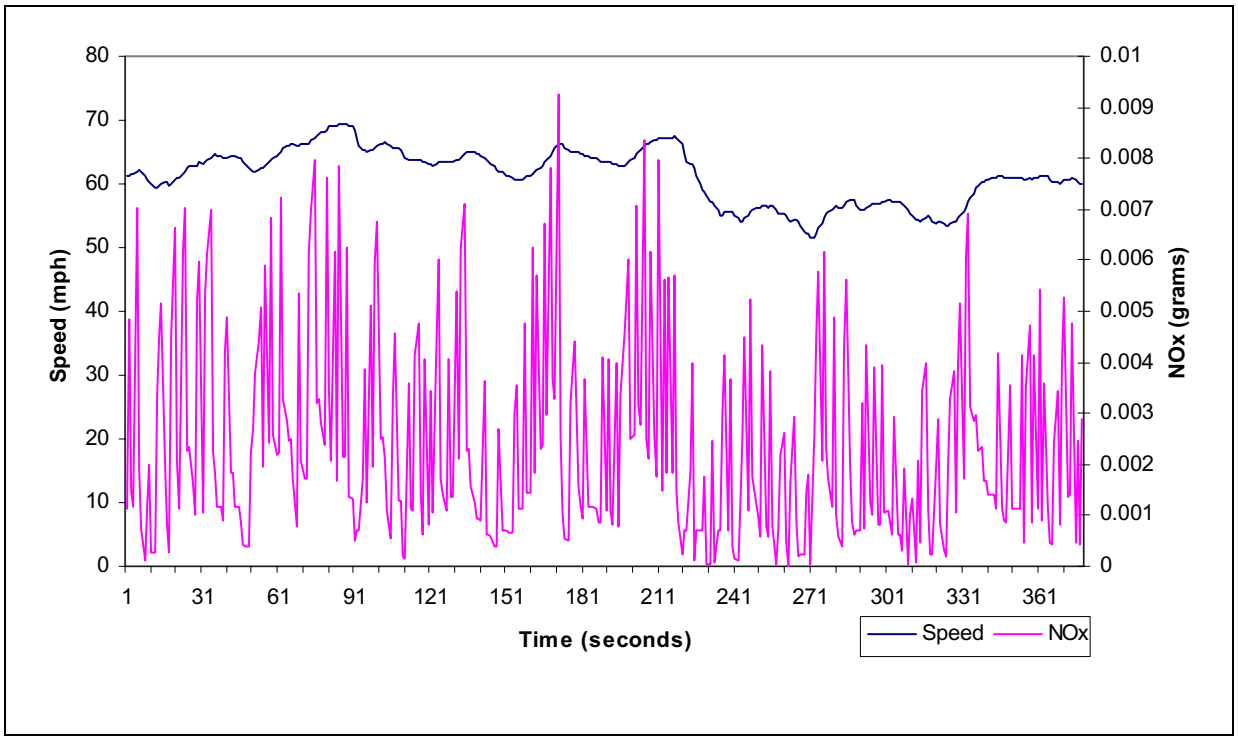

Figure 18a. Route 3 Speed-NO_x Comparison (Off-Peak Period)

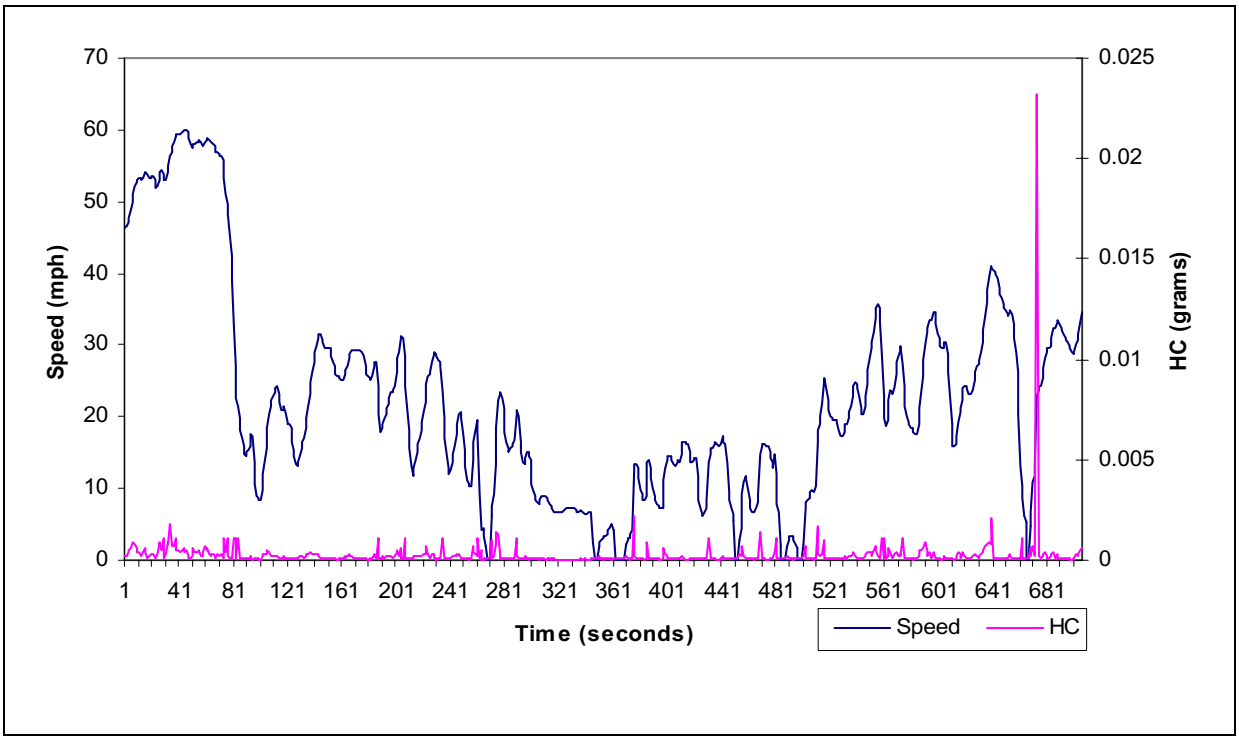

Figure 19a. Route 4 Speed-HC Comparison (Peak Period)

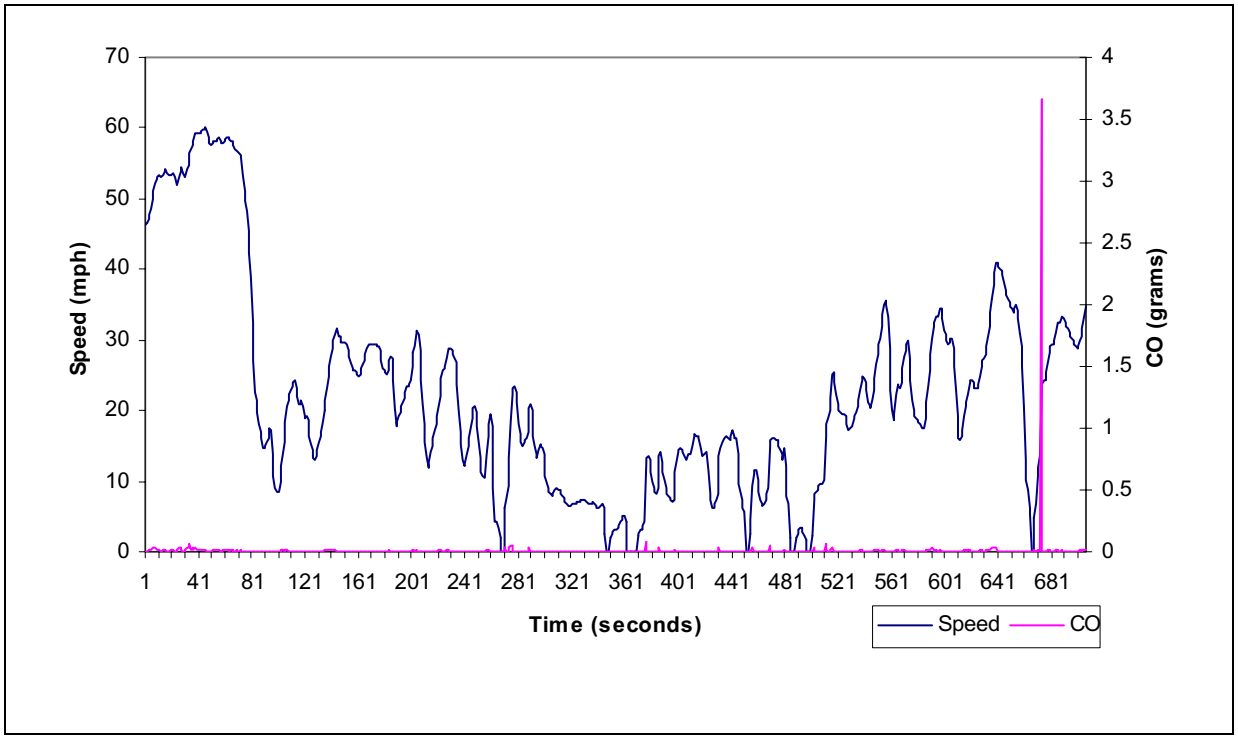

Figure 20a. Route 4 Speed-CO Comparison (Peak Period)

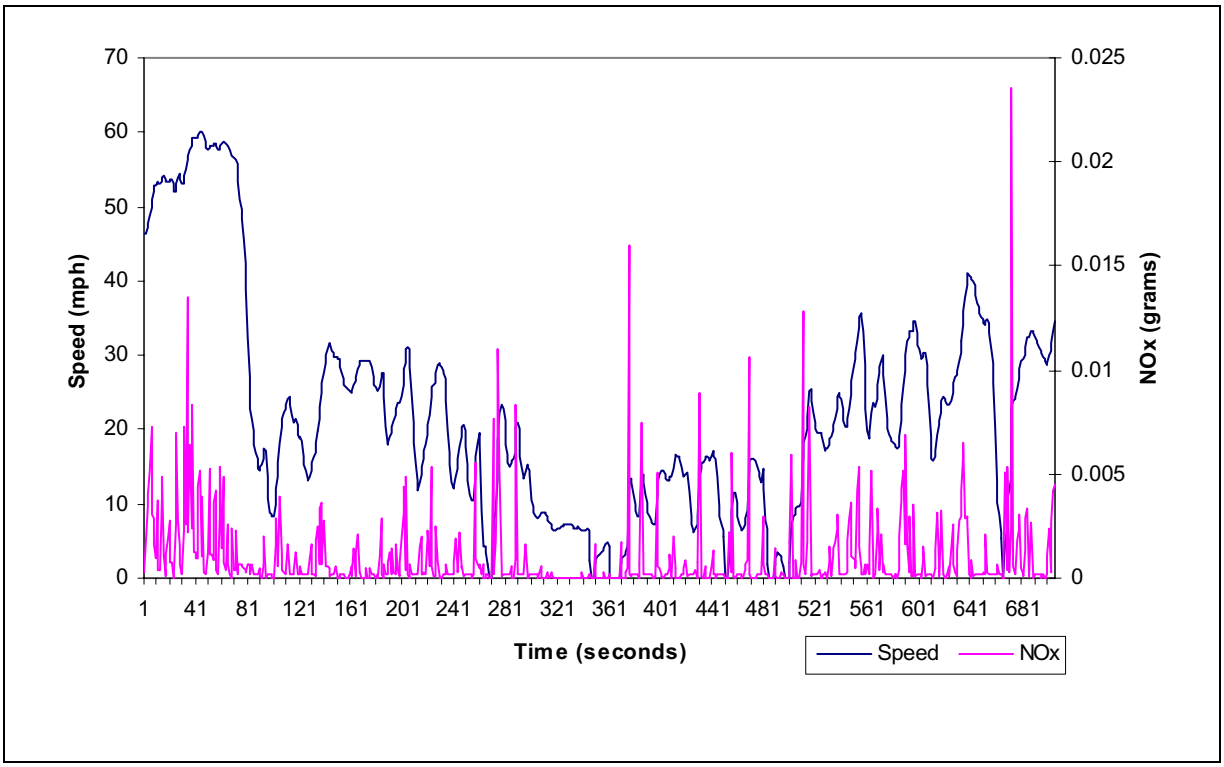

Figure 21a. Route 4 Speed-NO_x Comparison (Peak Period)

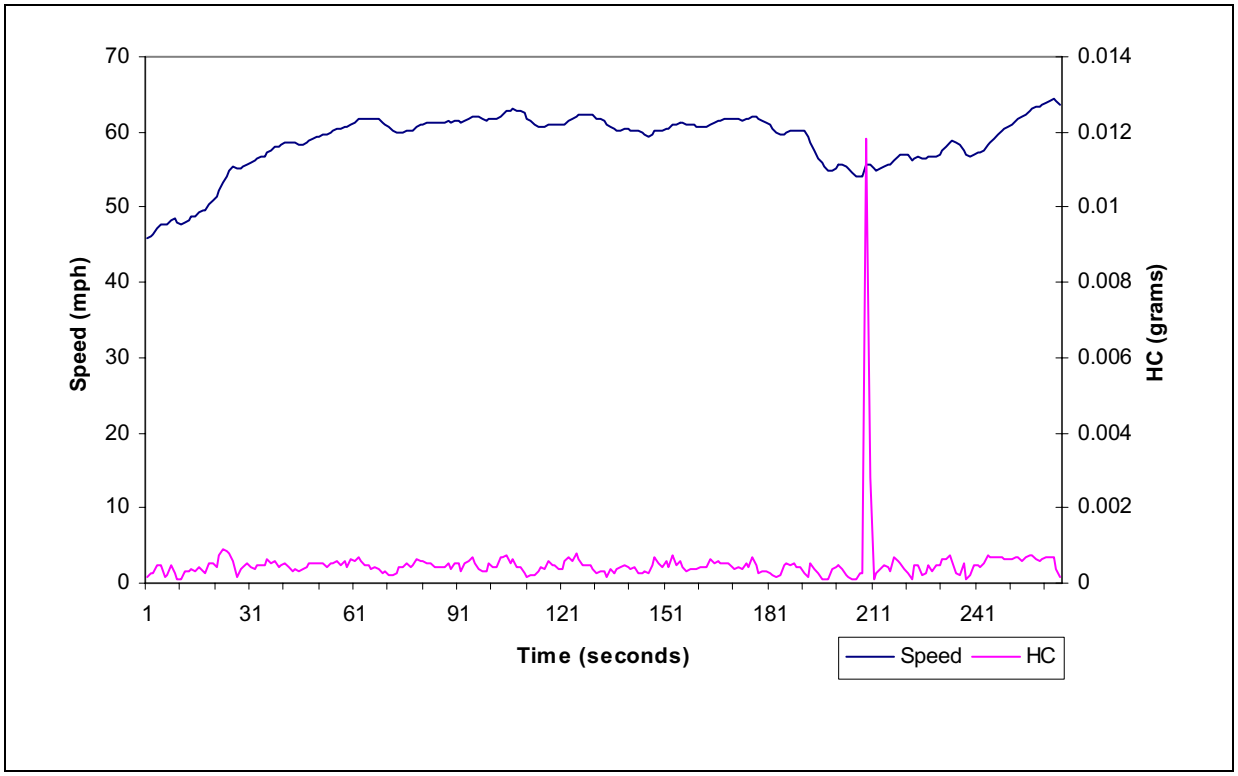

Figure 22a. Route 4 Speed-HC Comparison (Off-Peak Period)

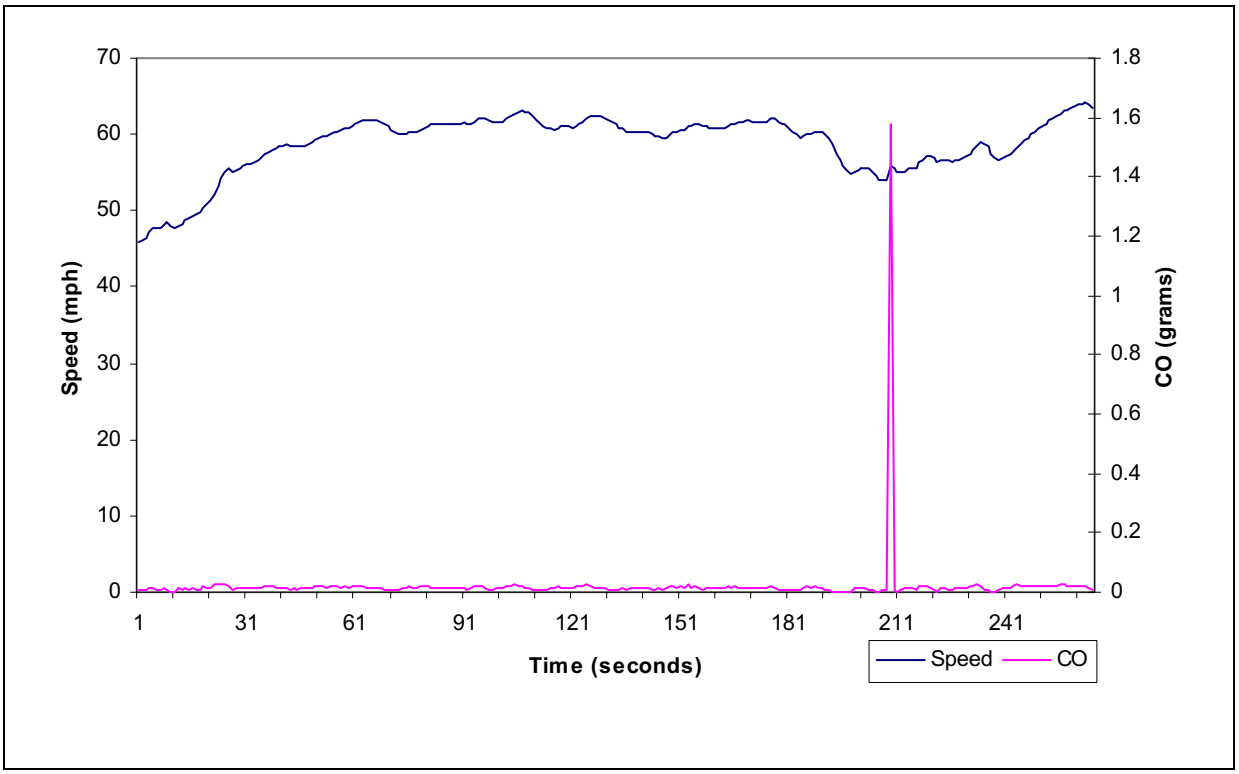

Figure 23a. Route 4 Speed-CO Comparison (Off-Peak Period)

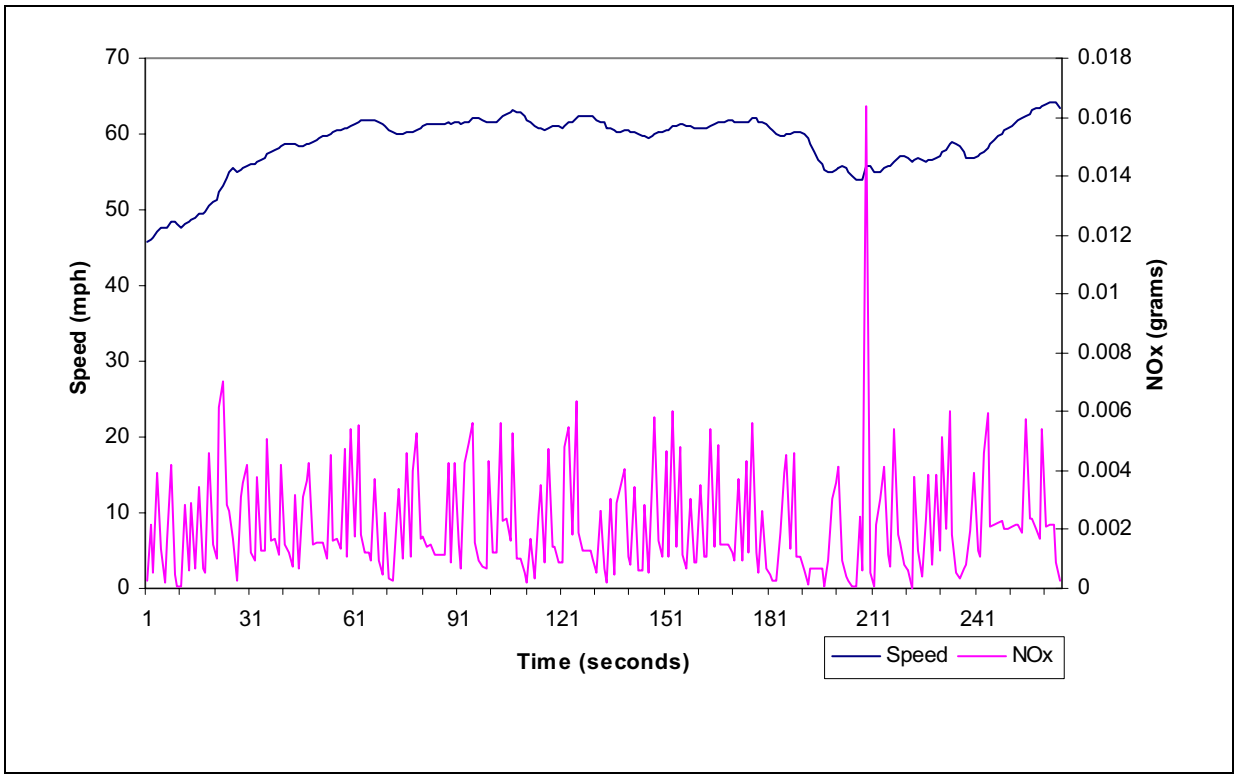

Figure 24a. Route 4 Speed-NO_x Comparison (Off-Peak Period)

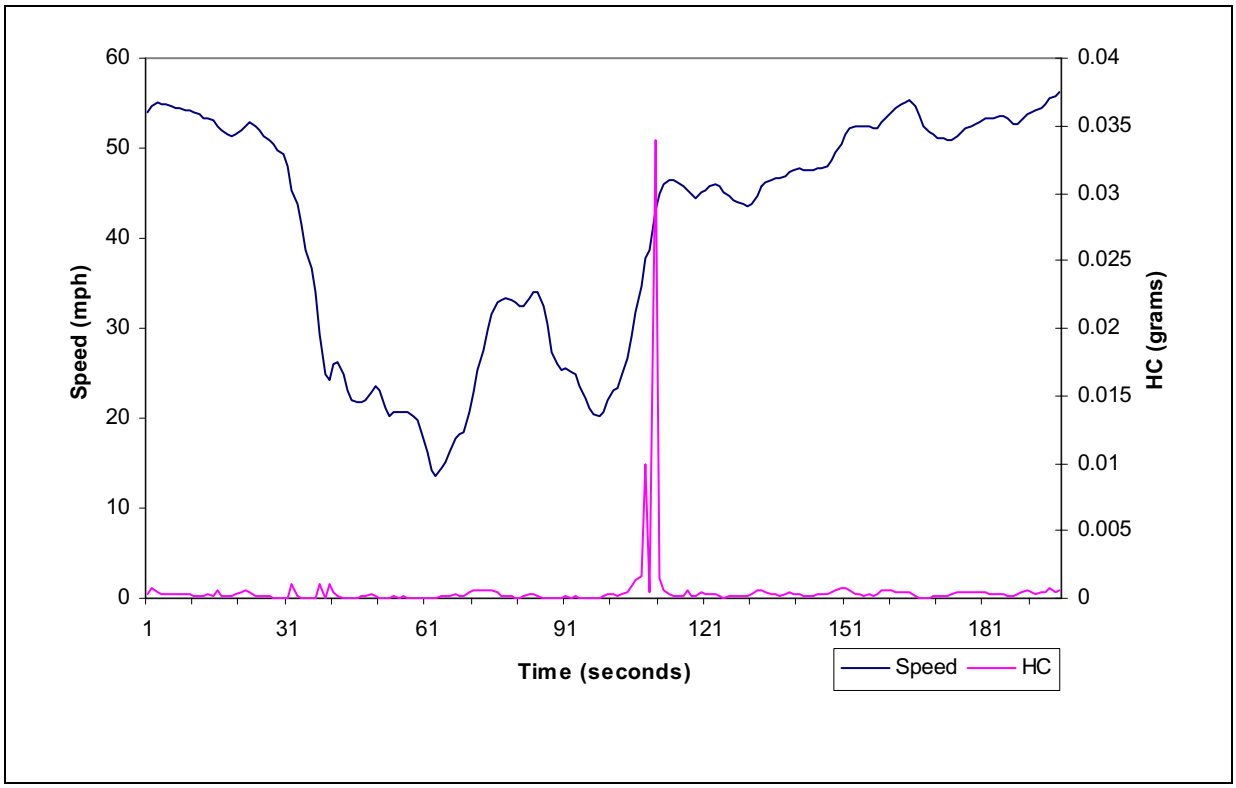

Figure 25a. Route 5 Speed-HC Comparison (Peak Period)

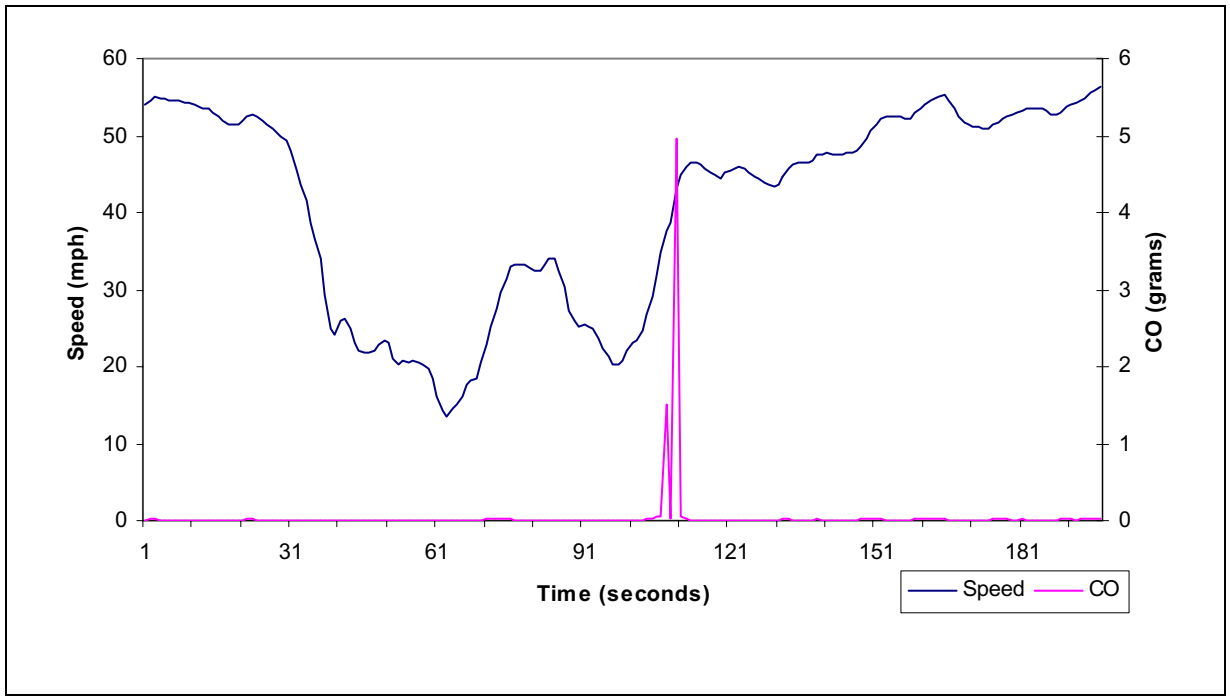

Figure 26a. Route 5 Speed-CO Comparison (Peak Period)

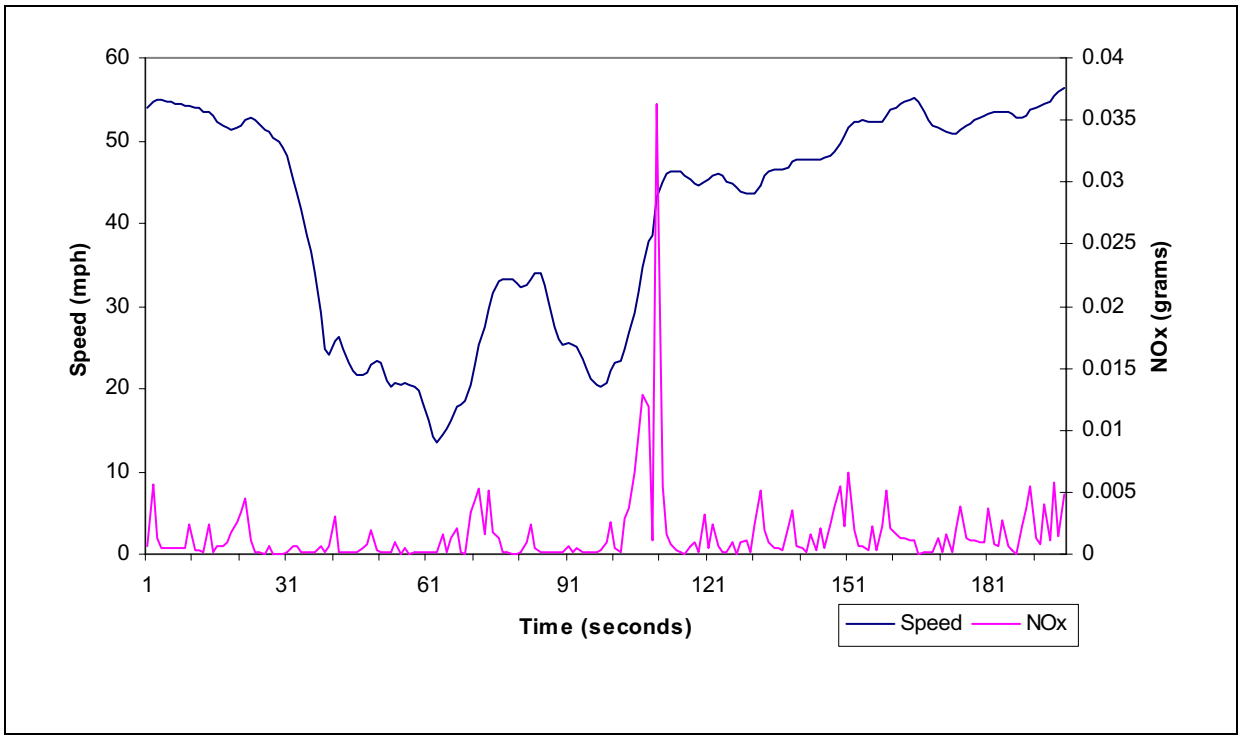

Figure 27a. Route 5 Speed-NO_x Comparison (Peak Period)

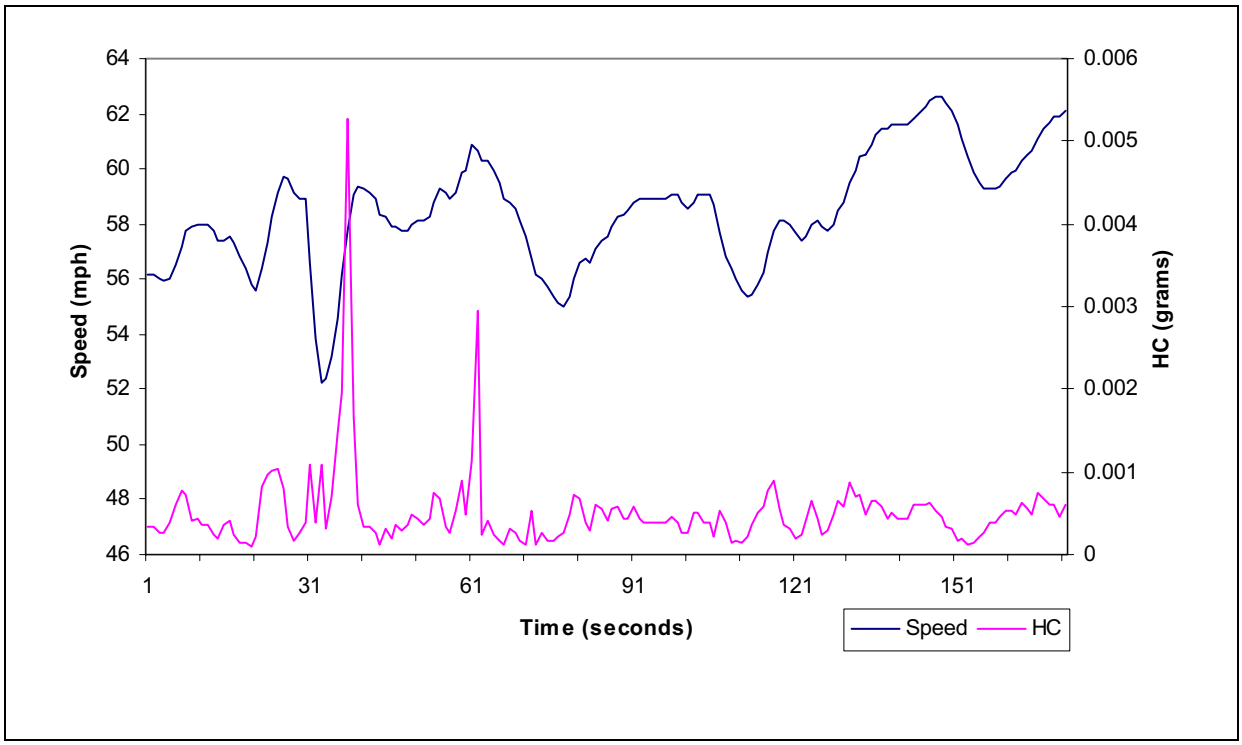

Figure 28a. Route 5 Speed-HC Comparison (Off-Peak Period)

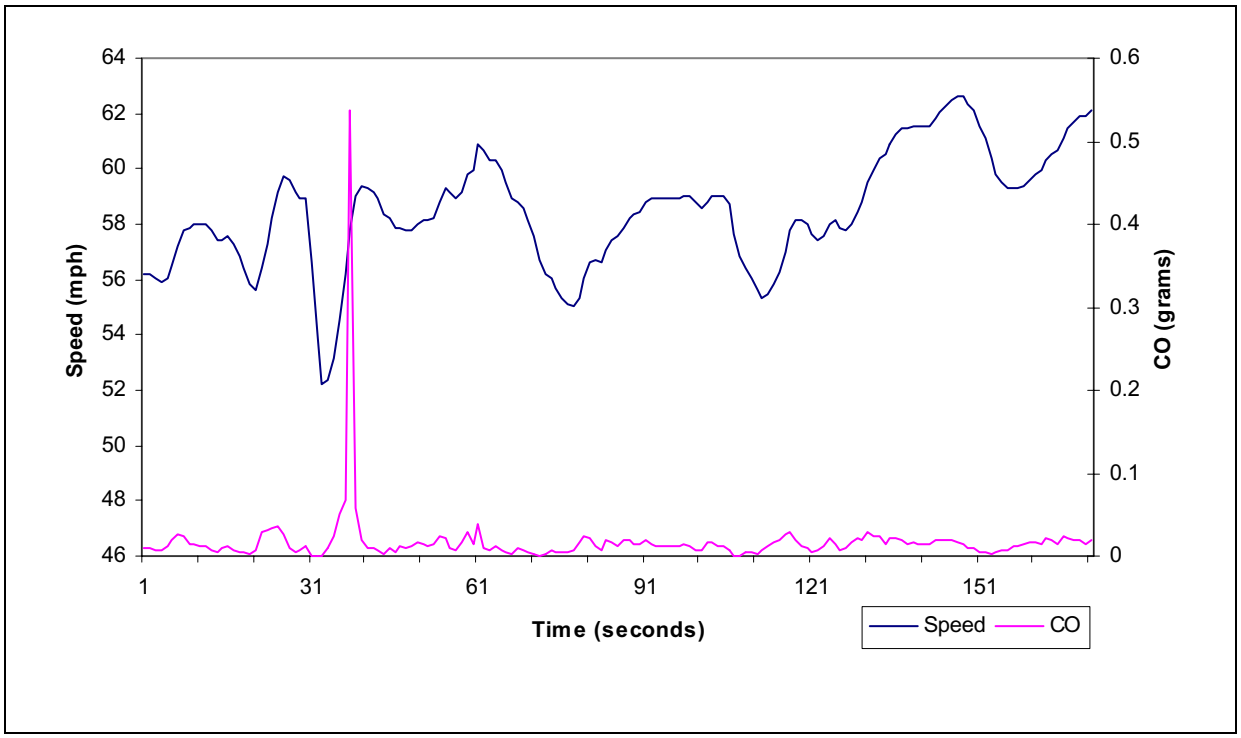

Figure 29a. Route 5 Speed-CO Comparison (Off-Peak Period)

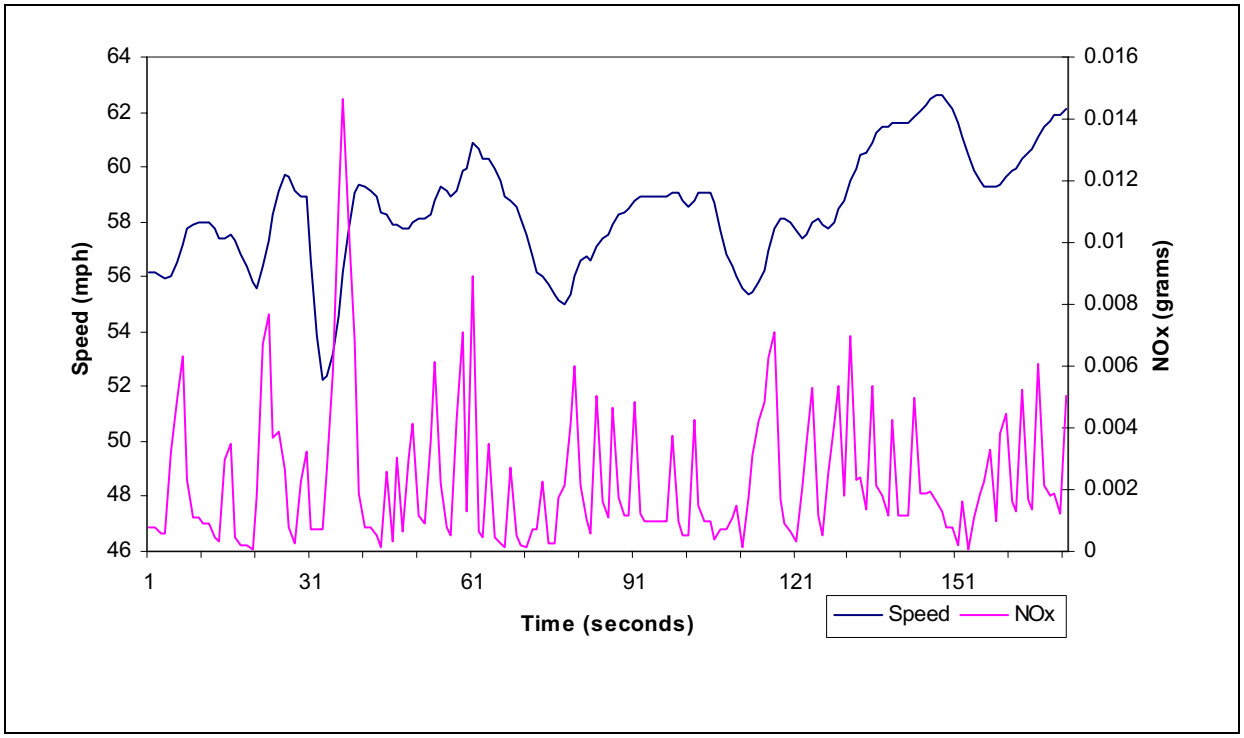

Figure 30a. Route 5 Speed-NO_x Comparison (Off-Peak Period)

APPENDIX B

```
 ****************************************************************************
*                                                                            *
*                                                                            *
*            CCCCCCC    MMM      MMM   EEEEEEEEEE   MMM      MMM            *
*           CCCCCCCCC   MMMM    MMMM   EEEEEEEEEE   MMMM    MMMM            *
*          CCC     CC   MMMMM  MMMMM   EEE          MMMMM  MMMMM            *
*          CCC          MMMMMMMMMMMM   EEEEEEEEEE   MMMMMMMMMMMM            *
*          CCC          MMM MMMM MMM   EEEEEEEEEE   MMM MMMM MMM            *
*          CCC     CC   MMM  MM  MMM   EEE          MMM  MM  MMM            *
*           CCCCCCCCC   MMM      MMM   EEEEEEEEEE   MMM      MMM            *
*            CCCCCCC    MMM      MMM   EEEEEEEEEE   MMM      MMM            *
*                                                                            *
*                  Comprehensive Modal Emissions Model                       *
*                                                                            *
*                      Version 2.01   July     2000                          *
*                                                                            *
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*                                                                            *
*                                                                            *
 ****************************************************************************

Input Files
Control File:  rep7-ctr
Activity File: pnb757.csv

Using calculated acceleration.

VEHICLE_CATEGORY = 10

FuelType not specified
  Defaulting to 'Gasoline' based on vehicle category.

Condition Parameters
Tsoak    =         0
SH       =     75.00

Vehicle Parameters
Ed       =      2.14
Masslb   =   2955.36
Trlhp    =     11.51
S        =     38.15
Nm       =      3218
Qm       =    125.42
```

```
Zmax     =    108.21
Np       =      5011
Idle     =    871.00
ng       =         4
Sload    =      3.42

Calibrated Parameters
K_0      =    0.2364
Edt3     =    0.1000
C0       =    3.6929
aCO      =    0.0908
aHC      =    0.0095
rHC      =    0.0028
aNO1     =    0.0363
aNO2     =    0.0325
FRNO1    =   -0.3061
FRNO2    =    0.1849
MAXCO    =   99.9749
MAXHC    =   99.8272
MAXNO    =   99.8460
bCO      =    0.0958
cCO      =    0.9564
bHC      =    0.0213
cHC      =    0.3568
bNO      =    0.8650
cNO      =    1.4750
Lamb_0   =    1.2348
lam_m    =    0.1228
Pscale   =    1.3120
maxhc    =    0.0631
hc_jk    =    4.8037
r_R      =    0.3006
spd_th   =  106.3888
rO2      =   50.4484
COB1     =    9.3302
HCB1     =   10.4121
NOB1     =    6.2056
lam_cold =    1.1454
csHC     =    4.5515
csNO     =    4.8408
Tlamb    =   50.2254
id       =    0.0716
Csoak_co =  280.6832
Csoak_hc =   15.1177
Csoak_no =  399.9964
Bcat_co  =  224.8712
Bcat_hc  =  239.9966
Bcat_no  =    0.2637
Edt1     =    0.9178

Distance Traveled
4.33 miles

Tail Out Emissions
EHC  =      1.952 (grams/mile)
ECO  =      10.71 (grams/mile)
ENOx =       2.11 (grams/mile)
HC   =      0.039 (grams/mile)
CO   =       1.70 (grams/mile)
NOx  =       0.17 (grams/mile)
Fuel =      107.6 (grams/mile)
```

Figure 1b. CMEM Summary Output File

1	46.38	0.000187	0.005176	0.000287	0.784642
2	46.49	0.000225	0.006442	0.001860	0.877461
3	47.07	0.000415	0.012988	0.003479	1.257667
4	47.76	0.000485	0.015464	0.004047	1.376295
5	48.68	0.000634	0.020719	0.005208	1.601683
6	49.94	0.000911	0.030486	0.007271	1.959399
7	51.09	0.000874	0.029181	0.003036	1.914995
8	52.13	0.000832	0.027714	0.002849	1.864006
9	52.71	0.000555	0.017927	0.001650	1.485738
10	52.94	0.000377	0.011659	0.000938	1.189613
11	53.28	0.000442	0.013954	0.003703	1.305124
12	53.17	0.000231	0.006636	0.000422	0.890872
13	53.05	0.000229	0.006583	0.000417	0.887252
14	53.40	0.000445	0.014048	0.003724	1.309653
15	53.97	0.000592	0.019221	0.004882	1.540434
16	54.20	0.000403	0.012582	0.001039	1.237236
17	53.74	0.000122	0.003041	0.000114	0.598511
18	53.40	0.000153	0.004050	0.001182	0.692391
19	53.28	0.000232	0.006689	0.001926	0.894503
20	53.40	0.000331	0.010081	0.002787	1.103899
21	53.51	0.000333	0.010154	0.000777	1.107995
22	53.63	0.000335	0.010228	0.000784	1.112103
23	53.05	0.000087	0.001956	0.000043	0.478405
24	51.90	0.000051	0.000915	0.000223	0.325637
25	52.02	0.000307	0.009240	0.002580	1.055596
26	53.05	0.000870	0.029061	0.006975	1.910852
27	53.97	0.000823	0.027385	0.002808	1.852390
28	54.43	0.000536	0.017236	0.001569	1.455784
29	53.74	0.001089	0.000915	0.000709	0.325637
30	53.05	0.000051	0.000915	0.000223	0.325637
31	53.05	0.000275	0.008146	0.002305	0.989464
32	54.09	0.000915	0.030640	0.007303	1.964563
33	54.89	0.000776	0.025718	0.002598	1.792577
34	56.39	0.001790	0.061264	0.013459	2.834788
35	56.96	0.000687	0.022584	0.002210	1.675139
36	57.65	0.000796	0.026432	0.006426	1.818420
37	58.23	0.000731	0.024145	0.002402	1.734501
38	59.15	0.001052	0.035478	0.008297	2.121805
39	59.27	0.000455	0.014419	0.001244	1.327413
40	59.38	0.000458	0.014518	0.001255	1.332104
41	59.38	0.000392	0.012199	0.000997	1.217698
42	59.38	0.000392	0.012199	0.000997	1.217698
43	59.61	0.000536	0.017250	0.004447	1.456372
44	59.96	0.000624	0.020349	0.005128	1.586772
45	59.96	0.000405	0.012641	0.001045	1.240227
46	60.07	0.000476	0.015123	0.003970	1.360507
47	59.61	0.000181	0.004954	0.000268	0.767274
48	58.80	0.000051	0.000915	0.000223	0.325637
49	58.00	0.000051	0.000915	0.000223	0.325637
50	57.54	0.000157	0.004187	0.001223	0.704214
51	58.00	0.000642	0.020973	0.005263	1.611855
52	58.11	0.000428	0.013462	0.001137	1.281175

Figure 2b. CMEM Second-By-Second Output File

KIT R. BLACK

Kit R. Black will earn his Bachelor of Science degree in Civil Engineering from Texas A&M University in December 2002. He interned for the Amarillo Area Office of the Texas Department of Transportation (TxDOT) in the summer of 2001 as a field inspector. In the summer of 2002, he worked for The Texas Transportation Institute (TTI).

Kit is currently working part-time for TTI where he is involved in emissions modeling research. He is a member of Tau Beta Pi Engineering Honor Society and the Texas A&M University Student Chapter of the Institute of Transportation Engineers (ITE). Kit will begin pursuing his Master of Engineering degree in Civil Engineering in January 2003 at Texas A&M University. His career interests include traffic control devices, highway design and safety.

TRANSPORTATION IMPROVEMENT STRATEGIES FOR THE TEXAS MEDICAL CENTER (TMC)

by

S. Matthew Feil

Professional Mentor
Brian S. Bochner, P.E., P.T.O.E.
Senior Research Engineer
Texas Transportation Institute

Program Coordinators
Conrad L. Dudek, PhD., P.E.
Shawn Turner, P.E.

Prepared for
Texas A&M University
Undergraduate Fellows Program

Texas Transportation Institute
and the
Department of Civil Engineering
Texas A&M University
College Station, TX

August 2002

SUMMARY

The Texas Medical Center (TMC) area currently has significant on-going problems with traffic congestion and parking problems due to the innovative advances continually occurring at the many institutions housed within the TMC. This research was conducted to assist in the development of a successful solution to the continual problem TMC is facing. This research was to comparatively evaluate three distinct transportation improvement strategies that included a study area consisting of TMC and the surrounding activity centers. The three strategies that were evaluated were as follows: network improvements, system connectivity, and operational improvements and redevelopment.

These strategies were chosen based on characteristics such as the future need for commercial and residential development, the growing need for accessibility to the institutions housed within TMC, the realized need for controlling the flooding in and around TMC, and the traffic-related needs associated with the growth and popularity of TMC and its institutions (*2*).

A complete traffic volume analysis of the study area was conducted for each of the transportation improvement strategies. The total delays were calculated for both vehicles and persons for each of the strategies. All three strategies provided a significant reduction in delay, but Strategy 2 did not produce as much delay reduction as Strategies 1 and 3. Based on the findings of this evaluation, combining the most effective improvements from all three of the previous strategies created a new strategy; the resulting strategy generated about 12% more reduction than the greatest delay reduction resulting from the previous strategies. Some of the major transportation improvements included in this new strategy are the improvement of Holcombe, either by widening or making it a 1-way pair, and the improvement and extension of Cambridge. Further research and evaluation will be performed to determine the most viable and conducive options for these major projects.

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INTRODUCTION

The Texas Medical Center (TMC) area currently has significant on-going problems with traffic congestion, parking problems, and emergency vehicle access due to the innovative advances continually occurring at the many institutions housed within the TMC. This research was conducted to assist in the development of a successful solution to the current problem TMC is facing. The study area consists of a primary study area and a secondary study area. The primary study area is composed of a northern boundary of Bissonnet-Binz, an eastern border of State Highway (SH) 288, a southern border of Interstate Highway (IH)-610, and a western border of Main Street. The secondary study area extends the northern boundary US 59, and also extends the western boundary to Kirby.

Texas Transportation Institute (TTI) is currently overseeing a project involving TMC and the surrounding activity centers. TTI is directing the evaluation and study of the area and receiving information obtained by companies that include Walter P. Moore and Associates, Kimley-Horn and Associates, Inc., AECOM, and LKC Consulting Services. Each of these companies has the responsibility of completing specific studies based on their knowledge and their areas of expertise. TTI will then assemble the given information, evaluate the transportation improvement strategies, and endorse the best possible strategy (according to its performance under the set criteria).

Literature Review

Before beginning this research, several technical memorandums were written regarding the Transportation Master Plan for the Greater Texas Medical Center Area. These memorandums were read to grasp an understanding of the current conditions, the traffic-related needs of the study area, and the goals and objectives that need to be addressed in this study. Several studies have been conducted to study the impacts that closing a section of roadway may have on the affected area, obtain traffic counts, study existing travel patterns, review existing traffic operations, and study the impact of proposed parking lots (*1*).

Some of the traffic-related needs that have been voiced by stakeholders of the study area have been categorized into accessibility, flood control, future developments, and neighborhood considerations (*2*). These needs have been considered during the process to formulate the criteria requirements of the strategies. The relevant transportation-related goals and objectives of the authorities that have jurisdiction over the study area have provided a list of common goals and objectives for TMC's master plan (*3*). Many other activity centers throughout the nation have had success with the strategies imposed to satisfy their transportation access and mobility needs, and interviews with representatives from these centers helped to provide some characteristics and concepts that may be applicable to TMC (*3*).

Prior to and throughout this research, a number of different resources were read to gain an understanding of the process of comparative evaluation, the assessment process used during the evaluation of the transportation improvement strategies. A comparative evaluation refers to assessing the environmental and economic impacts of the transportation improvement strategies, including the adherence of each strategy to the necessary criterion, and comparing the results of each strategy against the base case as well as against each other. This form of evaluation will enable the best possible improvement strategy to be selected for implementing the most conducive transportation improvements.

An understanding of level of service (LOS) was also acquired from the information read and studied prior to this research. A LOS analysis is a technique used to determine how well intersections and transportation networks are functioning (*2*). The LOS is established by evaluating the peak periods of traffic at an intersection or in a network. An intersection level of service is based on the average delay that each vehicle experiences at an intersection. The rankings of LOS range from LOS A being the best

to LOS F being the worst. Most intersections are considered acceptable either at or above LOS C, but in high activity areas like the study area a LOS D is deemed acceptable (*2*).

Criteria Requirements

The strategy that will best satisfy the problem in and around the TMC must also satisfy certain criteria that was established to aid in the decision to determine this strategy. The criterion reflects the wants and needs of the shareholders involved and is an effective tool to evaluate the transportation improvements associated with each strategy. The strategy to adhere to these guidelines most effectively yields the best possible transportation improvement strategy to combat the consistent growth and expansion of TMC and the surrounding activity centers. The criteria that must be strictly utilized for evaluation are the following:

Table 1. Transportation Objective, Criteria, and Measure

Objective	Criteria	Measure
1.Improve Access	Accessibility	Peak hour intersection delay reduced
2. Improve activity area circulation	Circulation	Number of additional blocks less than 7,000 feet in length
3. Increase parking	Parking	% Parking demand provided
4. Balance transportation system	Transportation modes	% Transit entering at LOS D or better
	Transportation modes	% Bike route miles not on major thoroughfares
5. Enhance Neighborhood character	Neighborhood	VMT in residential and park frontage
6. Emergency access during flooding	Emergency vehicles	Miles of key routes above 100-yr flood plain
7. Use transportation resources effectively	Resources	Improvement cost per estimated person hour of delay reduced

EVALUATION

Traffic volume maps, generated from an existing 2025 travel forecast for the adopted Metropolitan Transportation Plan (MTP) covering the study area, were created for each of the transportation improvement strategies. Each map shows the improvements to be made that are specific to that individual strategy, and includes daily traffic volume projections. These traffic volume maps for the three transportation improvement strategies are available in Appendix B.

Based on the improvements related to each strategy, traffic volume counts obtained from the Houston-Galveston Area Council (HGAC) and the METRO Mobility 2025 Plan were assigned to the roadways and distributed to demonstrate the traffic conditions associated with the individual strategy (*1*). A vehicle

occupancy factor of 1.2 was used to calculate the person delay-hours associated with personal vehicles, a bus occupancy factor of 30 was used to calculate the person delay-hours associated with buses, and a transit occupancy factor of 50 was used to calculate the person-delay hours associated with Light Rail Transit (LRT).

The calculations using the traffic volume measurements were based on the number of vehicles approaching the major or problematic intersections included in the study. An automated spreadsheet was developed to aid in the quantifying analysis and evaluation of each of the three proposed transportation improvement strategies. This spreadsheet established the Level of Service (LOS) for the intersection and the directional approaches, the average delay corresponding to the LOS, the weighted delay of each intersection as a whole, and the overall delay to reflect the success of the strategy. The LOS/delay analysis was a planning method developed by TTI project staff for this project by combining features from capacity analysis techniques contained in the latest *Highway Capacity Manual* and TRB Transportation Circular 212.

Strategy 1: Network Improvements

The elements indicative of Strategy 1 include improving intersections along major thoroughfares, changing Holcombe-Galen/Pressler area into a 1-way pair, extending and improving Cambridge, promoting the use of Braeswood west of Greenbriar and connect to Old Spanish Trail via Stadium Drive, extending the frontage roads through the interchange of IH-610 and SH 288, and connecting the future HOV lanes on IH-610 or SH 288 to major thoroughfare systems within the study area. Figure 1 shows the modal delays associated with the Base Case (do nothing) and Strategy 1, and Figure 2 shows a map depicting the network improvements.. The overall person delay would decrease by 3,244 hours by implementing Strategy 1.

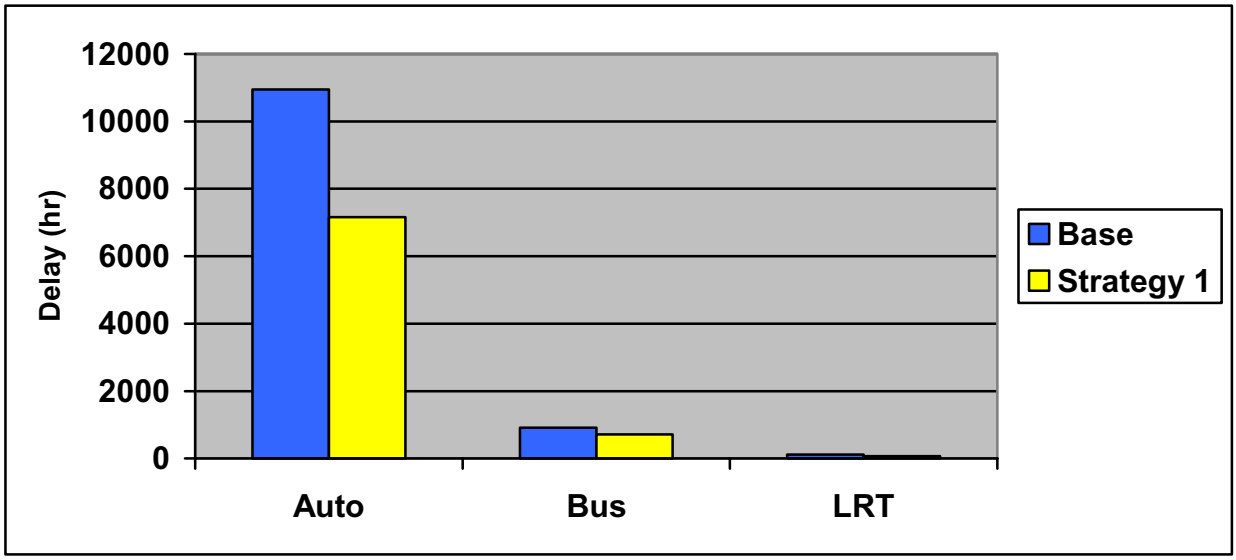

Figure 1. Modal Delay Comparison Base vs. Strategy 1

Strategy 2: System Connectivity

The characteristic elements of Strategy 2 are the following: extend Bertner and create a Knight-Bertner through movement, improving Holcombe intersections between Kirby and SH 288, complete Bellfort between Stella Link and Buffalo Speedway, add lanes to Greenbriar between Rice Blvd. and Main, create MacGregor-Dixie-Almeda access for TMC with IH-610 access to/from the east, and completing other

discontinuities. Some of the other discontinuities include portions of La Concha, several sections of Grand, and Dixie. Figure 3 is a map showing the network connectivity improvements, and Figure 4 shows the total person delays associated with each mode of transportation and compares the Base Case delays with Strategy 2 delays. The overall person delays for Strategy 2 would decrease by an amount of 2,424 hours.

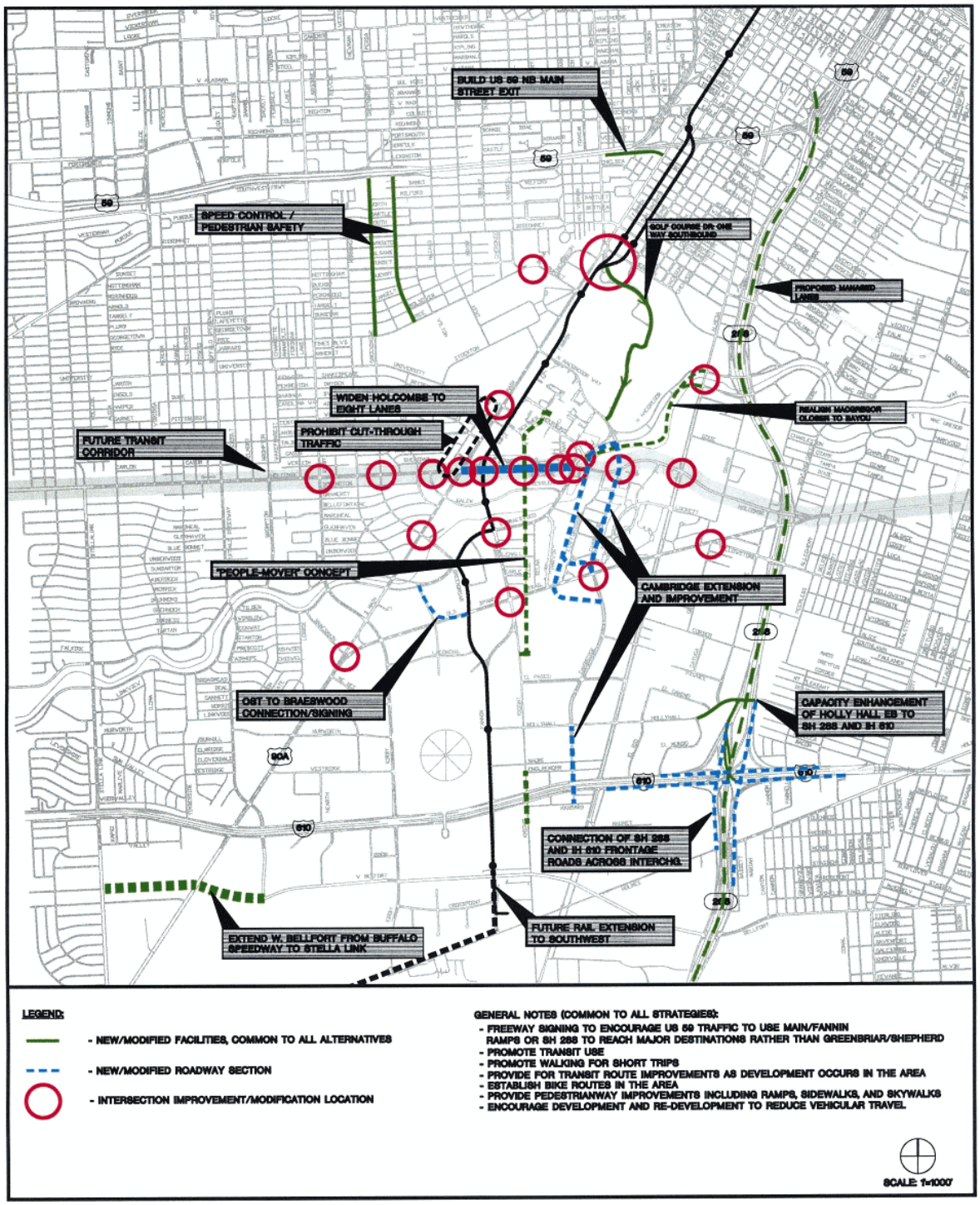

Figure 2. Network Improvement Map

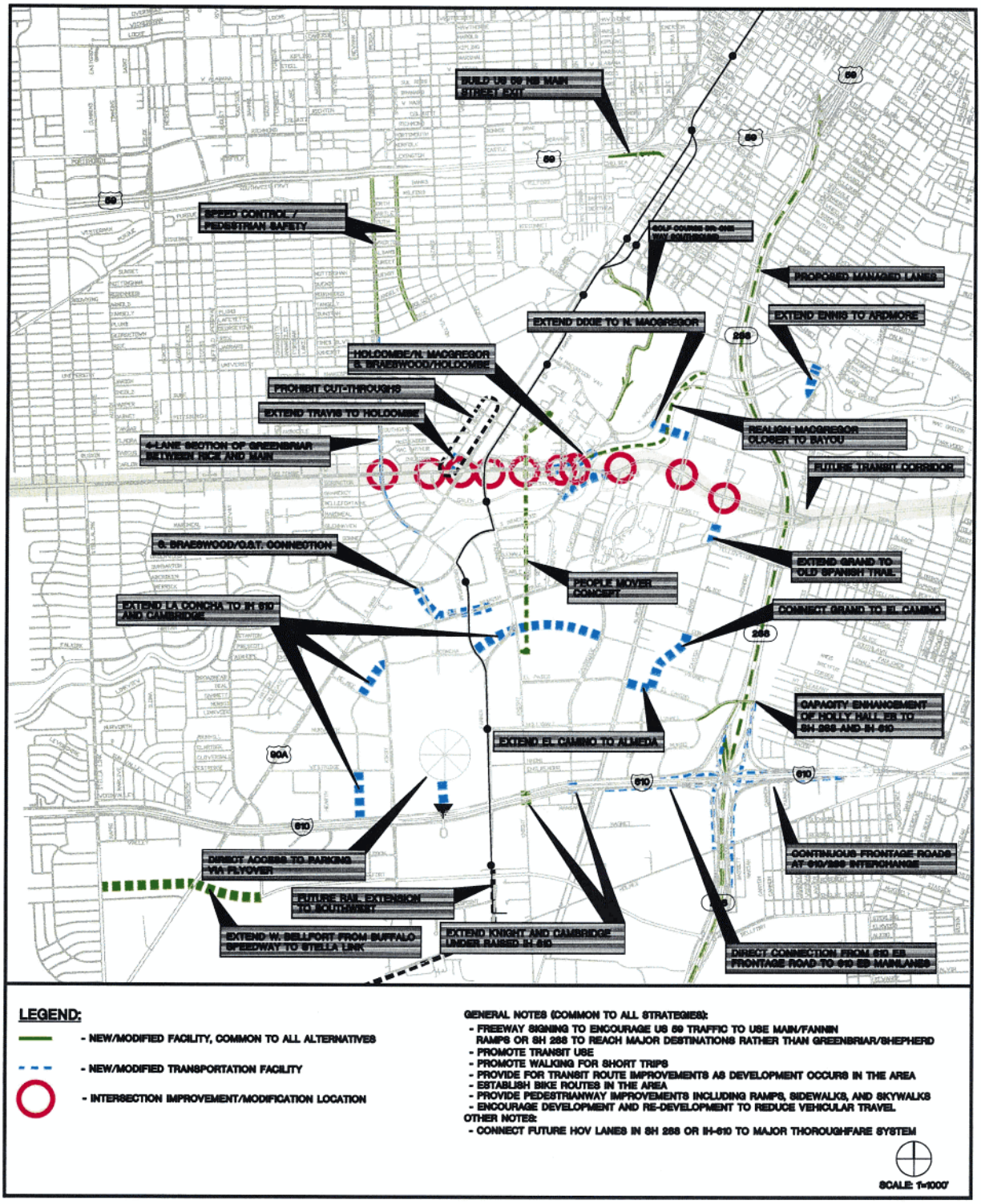

Figure 3. System Connectivity Map

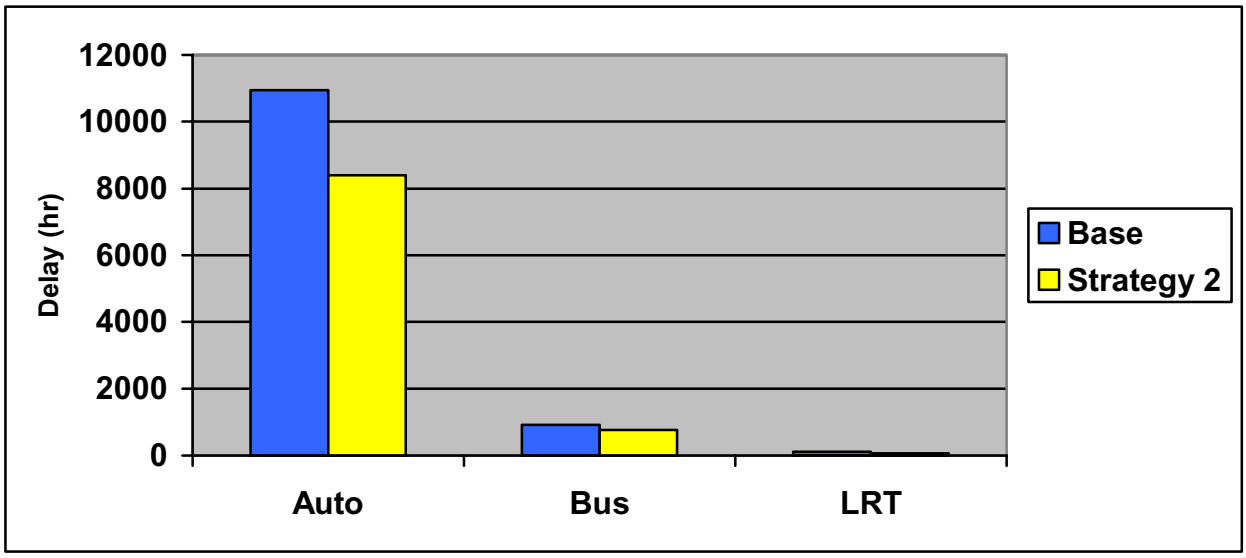

Figure 4. Delay Comparison Base vs. Strategy 2

Strategy 3: Operation Improvement and Redevelopment

Strategy 3 has its own signature elements that consist of improving all intersections between thoroughfares by adding turn lanes and expediting transit movement, providing remote parking adjacent to IH-610 and SH 288 with direct shuttle service to the major activity centers, reducing pedestrian-vehicle conflicts to facilitate the movement of both, creating intersection grade separation at Holcombe-Braeswood, providing reversible lanes on Holly Hall, Murworth, McNee, and Possibly Dixie, and providing more convenience services on sites of the major institutions and developments to serve the mid-day needs of employees and patrons. Figure 5 illustrates the total person delays related to the Base Case and Strategy 3, and Figure 6 shows a map with the operational improvements pointed out. A decrease of roughly 3400 hours of vehicle delay would be the result of a successful implementation of Strategy 3.

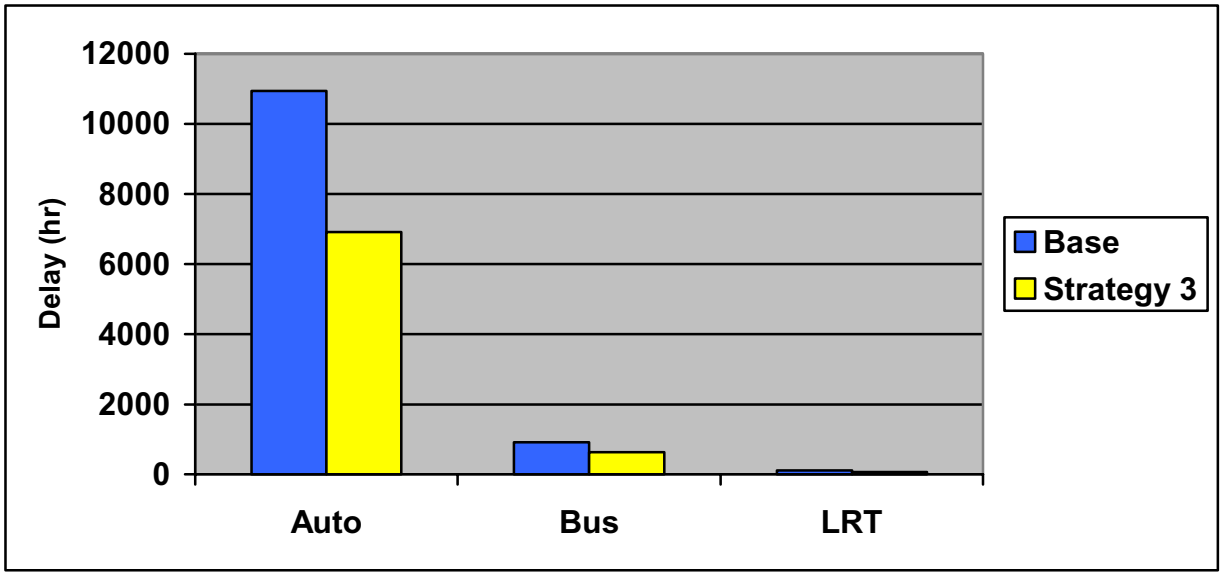

Figure 5. Delay Comparison Base vs. Strategy 3

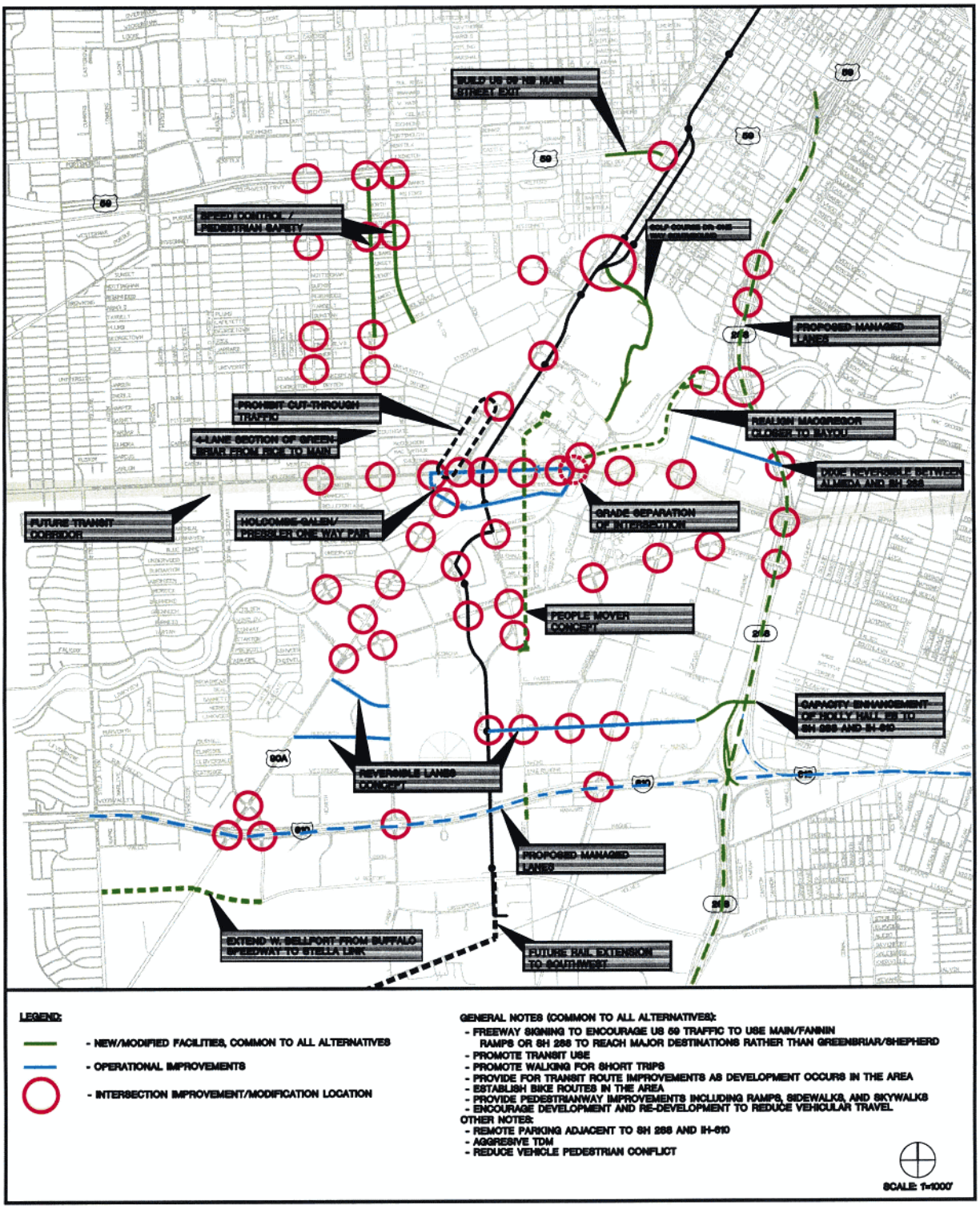

Figure 6. Operational Improvement Map

Evaluation Criteria

Once the delays associated with the individual improvement strategies were determined, they were evaluated against the approved criteria of the project. Then, after being satisfied that the criterion had been followed, the strategies were comparatively evaluated.

Table 1 below shows the outcome of the evaluation of the strategies to the criteria. Each goal of the criteria is listed and the best possible rating is chosen based on the criteria. The dark shading on the individual pie chart represents the magnitude of success for that strategy to satisfy the criteria. The more amount of dark shading there is, the greater the strategy's ability to comply with the criterion.

Table 2. Evaluation Results

Goal and Criteria	Strategy 1	Strategy 2	Strategy 3
Improve access (Peak hour intersection delay reduced)			
Improve activity area circulation (Additional blocks less than 7,000 ft)			
Increase parking according to demand (% Parking demand provided along approach routes)	—	—	—
Balance transportation system (% Transit entering at LOS D or better) (Bikeway evaluation - total possible score = 1520)	 	 	 
Enhance neighborhood character (VMT at residential and park frontage)			
Emergency access during flood conditions (Miles of key routes above 100-yr flood plain)	—	—	—
Use transportation resources effectively (Improvement cost index/delay reduced)			

There was no way to evaluate the strategies and their relationship to parking and emergency accessibility because there was no difference to quantify among the three strategies. Thus, it was assumed that the outcome would be the same for each of the transportation improvement strategies being evaluated.

Cost Analysis

Each of the three improvement strategies has several projects. Table 1 shows the total costs related to each of the strategies. These costs mainly cover construction and land acquisition. A more detailed listing of the projects and the individual project cost estimates for those projects is located in Appendix A. The criterion that includes the evaluation of the costs associated with the strategy improvements is titled "use transportation resources effectively" in Table 1. A comparison of just the total costs shows Strategy 1 as clearly being the least expensive solution, but evaluation of the cost-benefits associated with delay reduction shows otherwise.

Table 3. Project Cost Estimates for Strategies

Strategy	Total Cost Estimate
Strategy 1	$81,605,000
Strategy 2	$229,843,000
Strategy 3	$141,210,000

CONCLUSION

Though most of my time was spent mainly concentrating on the delays, all of the criteria was considered during the evaluation of the three strategies Based on the decrease in delay that is apparent for all strategies, choosing any of the three proposed strategies will result in a reduction in the experienced average delay. The evaluation of Strategies 1 and 3 resulted in similar reductions in the delay, 3,244 person-hours and 3,401 person-hours, respectively; Strategy 2 showed a decrease in delay by 2,424 person-hours. The corresponding percentage of decrease in delay for Strategy 1, 2, and 3 are 34%, 25%, and 36%. Figure 7 represents the total reduction of delay achieved by each strategy. Figure 8 shows the individual improvements associated with the new proposed strategy.

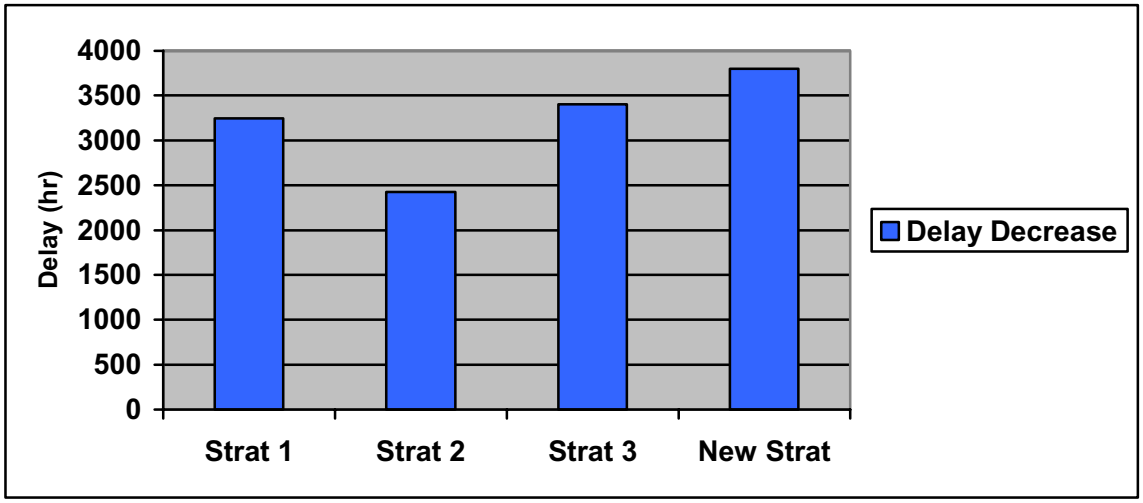

Figure 7. Delay Decrease Comparison.

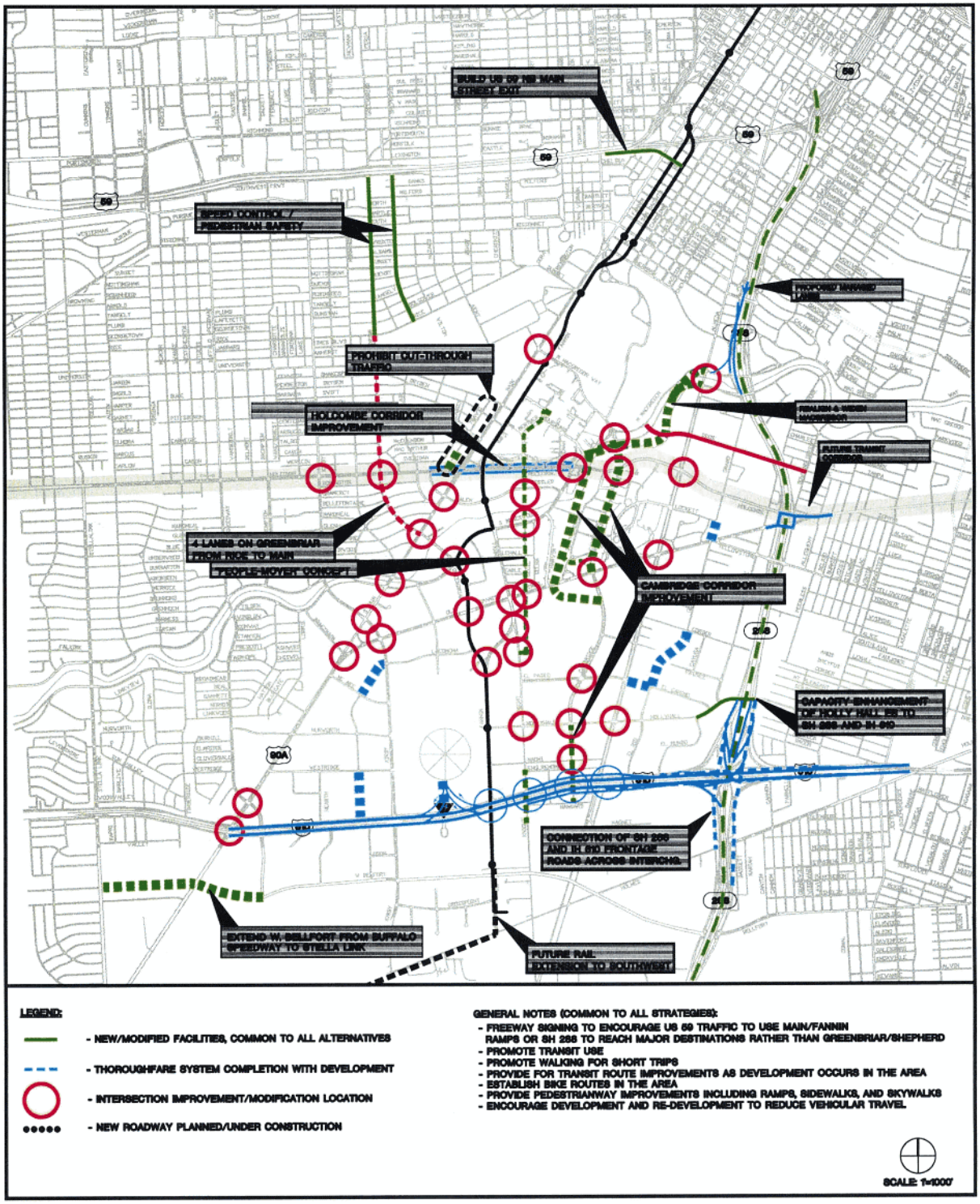

Figure 8. New Proposed Strategy Improvement Map

After evaluating the three original transportation improvement strategies, a new strategy was formulated based on considering all three of the improvement strategies. The new proposed strategy incorporates such projects as widening Holcombe for either 4 lanes of traffic on each side or a one-way pair, improving and extending Cambridge, extending Travis to Holcombe, extending Knight and Cambridge under raised IH 610, connecting La Concha, connecting Grand where it currently does not exist, and extending West Bellfort from Buffalo Speedway to Stella Link. Once this proposed strategy was devised, a traffic volume analysis was performed and the delay models were completed to determine the decrease, if any, associated with this new proposed strategy. The results of this evaluation showed an overall decrease in delay of 40% from the Base Case delay of 9,559 person-hours.

Recommendations

The results of the research show that any of the alternatives would decrease the time of delay experienced by people either in or around TMC and the surrounding activity centers. The recommended strategy is the new proposed strategy, which incorporates projects from each of the three previous transportation improvement strategies. This newly proposed strategy provides the most significant reduction in delay, and will also satisfy the other criteria of the evaluation. Lists should be created to display some short-range projects and some long- range projects, both of which will lead to the successful implementation of this new strategy.

During this research, there was no clear distinction between how each strategy improved either parking areas or emergency access during flood conditions, as compared to the other strategies. Surveys could be conducted to determine the options for the Cambridge corridor and Holcombe corridor improvements that satisfy both the stakeholders and the residents affected, but are also feasible and cost effective.

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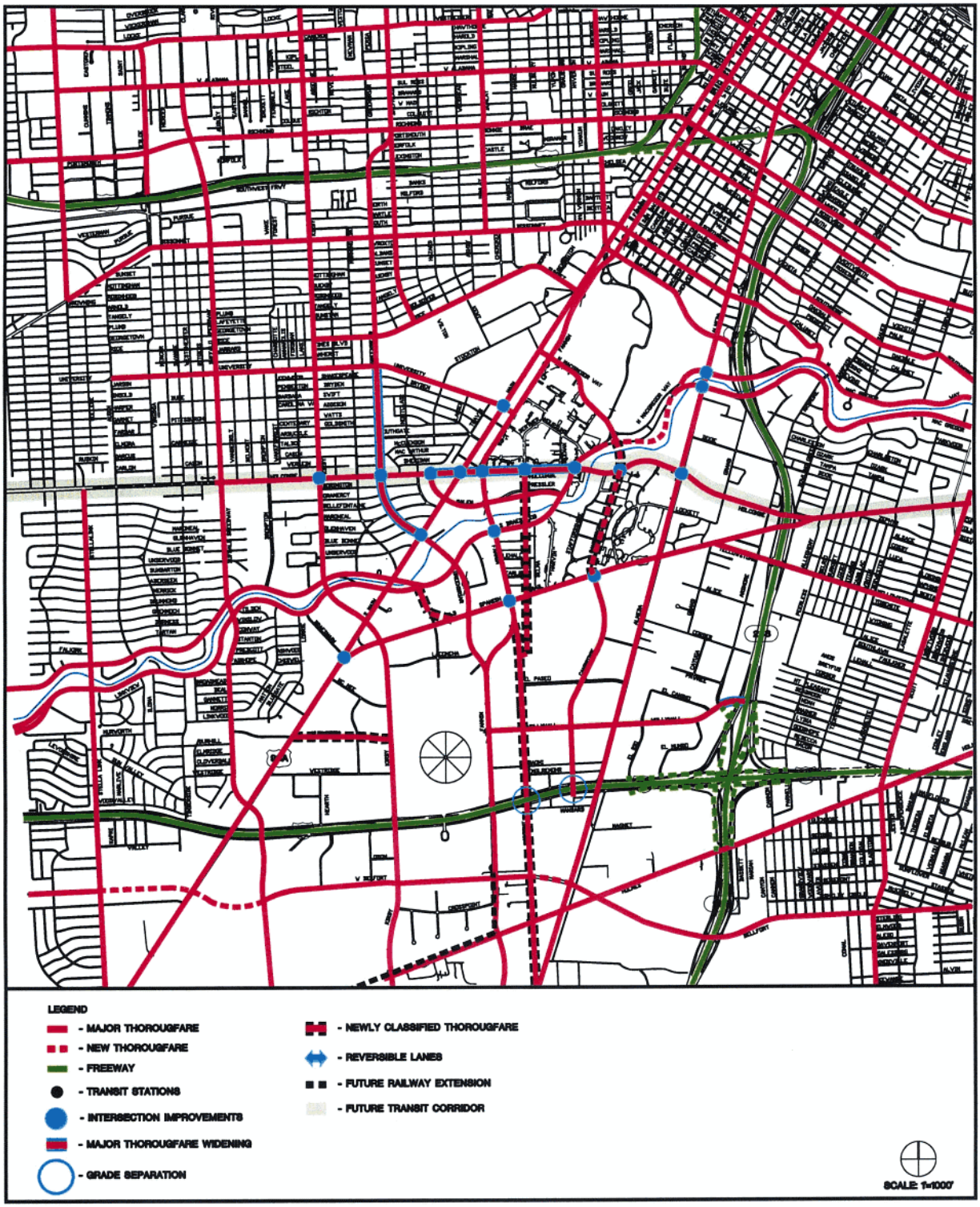
2. AECOM, Kimley-Horn and Associates, LKC, Texas Transportation Institute, and Walter P. Moore. *Transportation Master Plan for the Texas Medical Center*: *Needs Assessment*. Technical Memorandum 2. 6 May 2002.
3. AECOM, Kimley-Horn and Associates, LKC, Texas Transportation Institute, and Walter P. Moore. *Transportation Master Plan for the Texas Medical Center*: *Goals, Objectives and Guiding Principles*. Technical Memorandum 3. 27 May 2002.
4. *Highway Capacity Manual*. Special Report 290, Transportation Research Board, National Research Council, Washington, D.C., 1997.

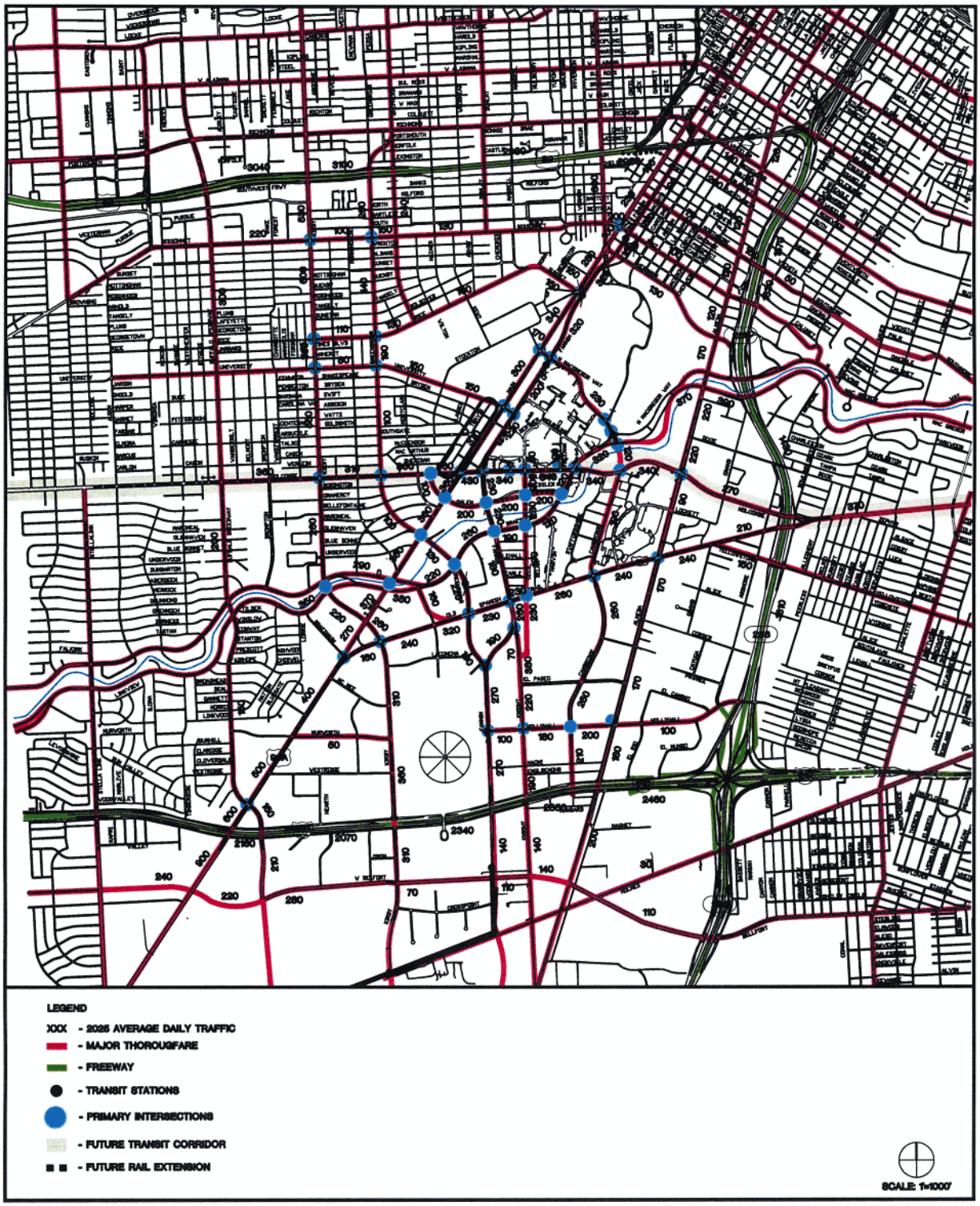
APPENDIX A

	Projects	Cost Estimate
Strategy 1	• Widen Holcombe to eight lanes	• $6,390,000
	• Cambridge extension and improvement	• $13,260,000
	• Braeswood to OST connection	• $1,305,000
	• Continuous frontage roads at 610/288 interchange	• $55,650,000
	• Various interchange improvements	• $5,000,000
Strategy 2	• Extend Ennis to Ardmore	• $4,032,000
	• Extend Dixie to MacGregor	• $4,716,000
	• Holcombe to N. MacGregor	• $2,400,000
	• Extension of S. Braeswood to Holcombe	• $6,480,000
	• Extend Travis to Holcombe	• $2,700,000
	• Improve Greenbriar from Rice to Main to 4-lanes	• $4,476,000
	• Extend Bertner south to Proposed Bio Tech Park	• $7,368,000
	• Braeswood to OST connection	• $1,305,000
	• Extend La Concha to 610 and Cambridge	• $20,340,000
	• Extend Grand to OST	• $2,604,000
	• Connect Grand to El Camino	• $6,480,000
	• Extend El Camino to Almeda	• $2,472,000
	• Continuous Frontage Roads at 610/288 interchange	• $55,650,000
	• Direct connection between 610 EB Frontage Rd. and EB Main Lane	• $6,840,000
	• Extend Knight and Cambridge under raised IH 610	• $87,840,000
	• Extend W. Bellfort from Buffalo Speedway to Stella Link	• $4,410,000
	• Direct access to Astrodome parking via Flyover	• $7,230,000
	• Various intersection improvements	• $2,500,000
Strategy 3	• Widen Holcombe to eight lanes – Main to Braeswood	• $6,390,000
	• Holcombe-Galen/Pressler One-way pair	• $6,120,000
	• Grade separation of Holcombe/Braeswood	• $53,700,000
	• Dixie reversible between Almeda and SH 288	• $2,700,000
	• Reversible lanes on McNee	• $960,000
	• Reversible lanes on Murworth	• $1,440,000
	• Proposed managed lanes on IH 610	• $54,900,000
	• Various intersection improvements	• $15,000,000

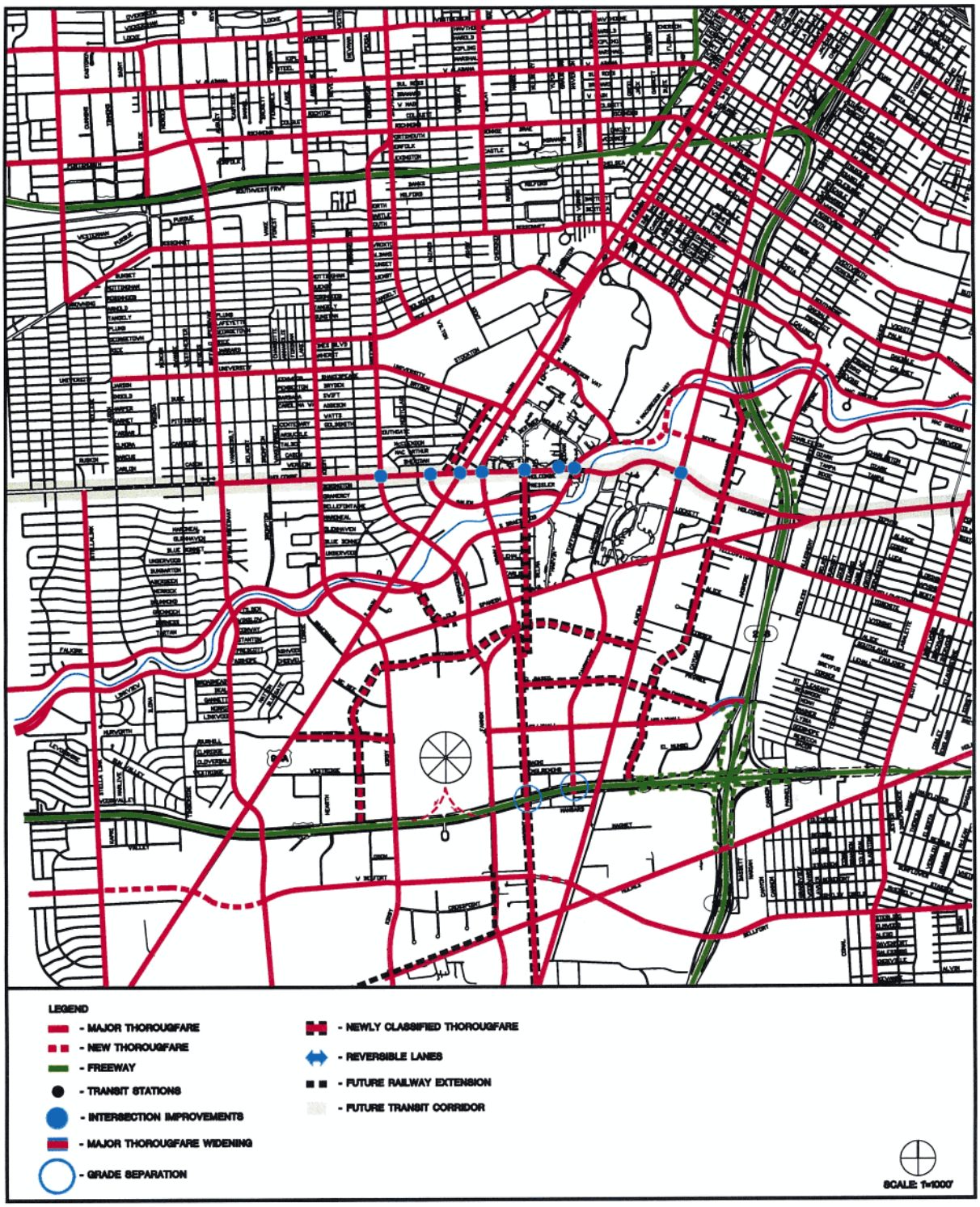
APPENDIX B



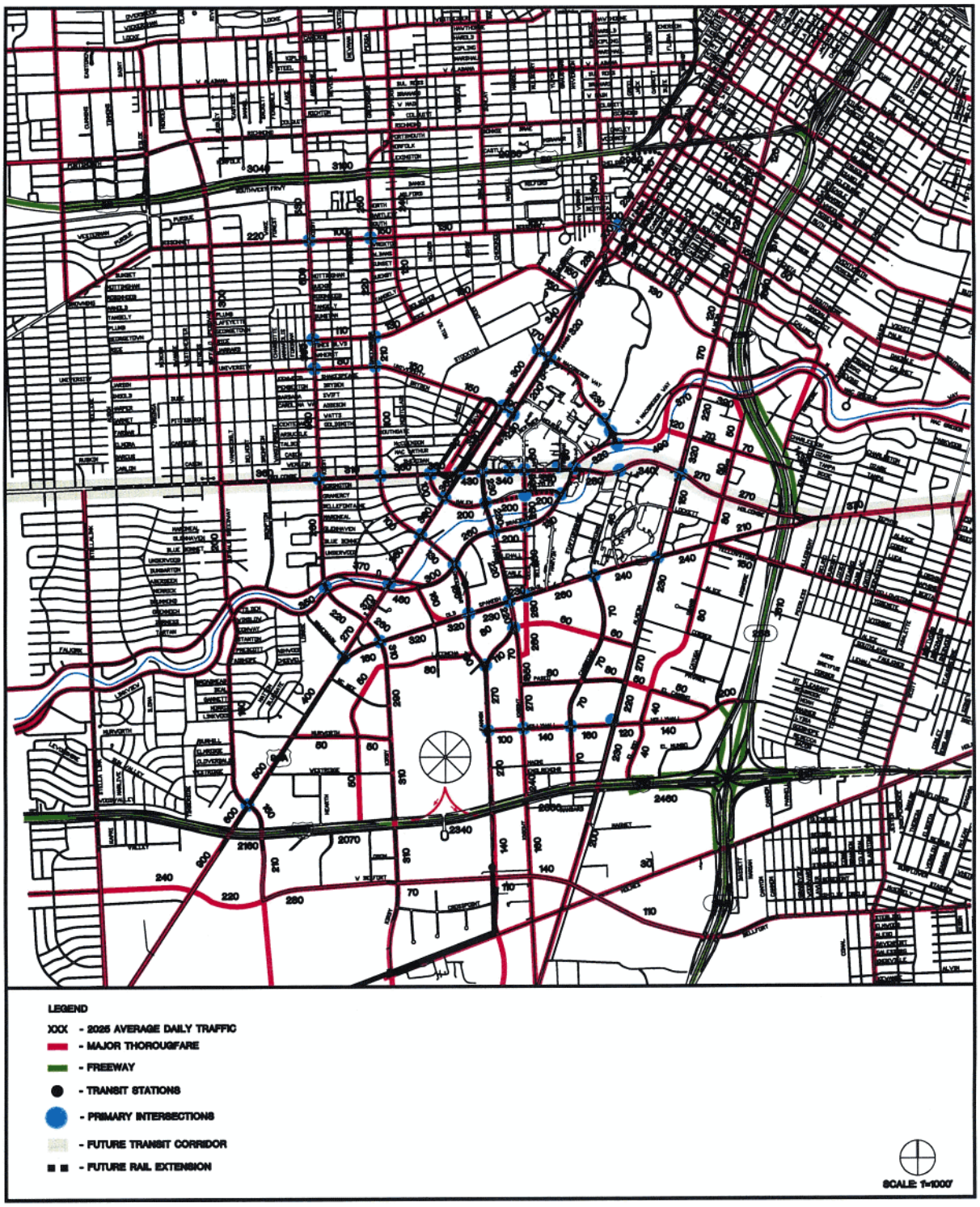
Strategy 1 Network Improvements



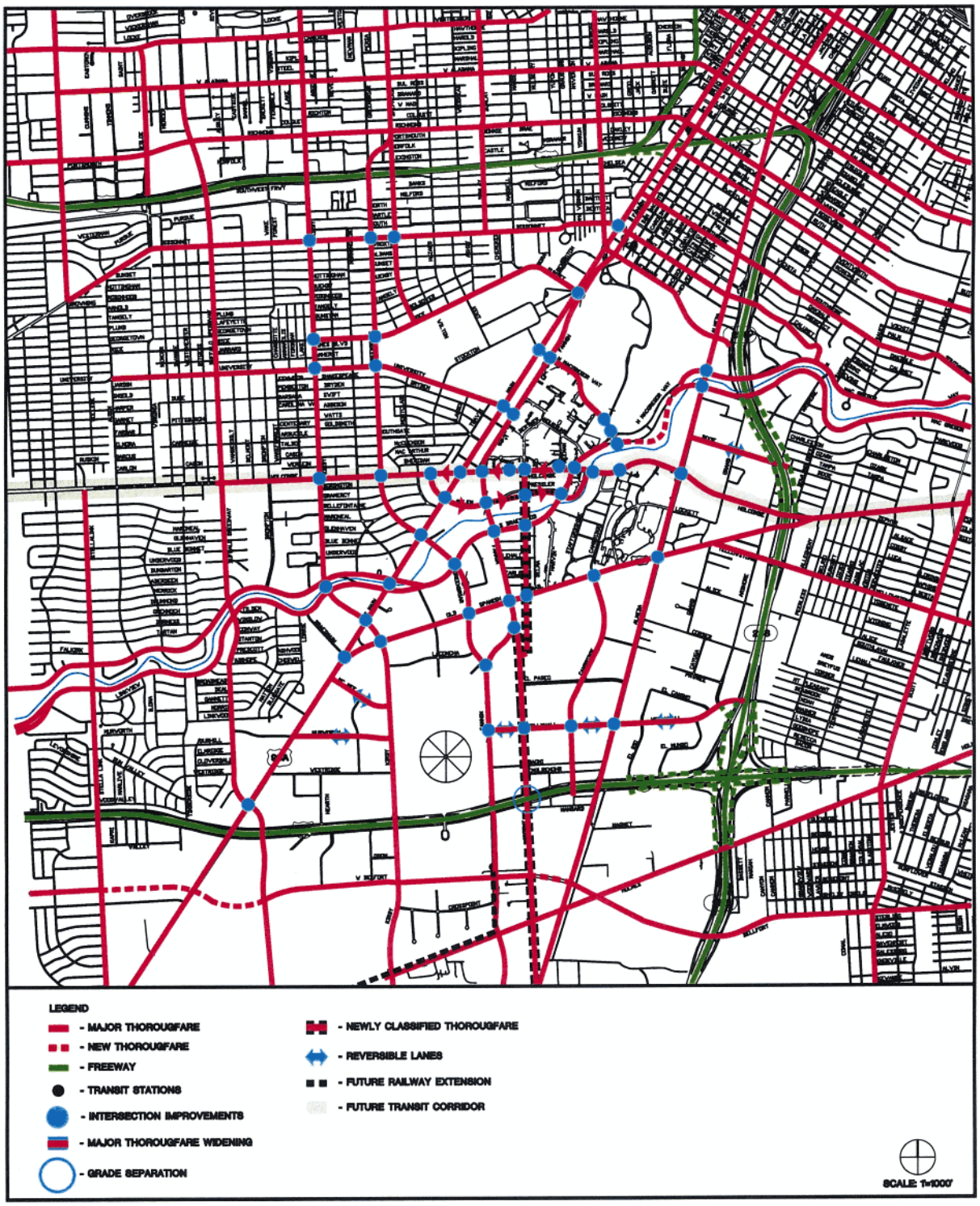
Strategy 1 Network Improvements With Traffic Volumes



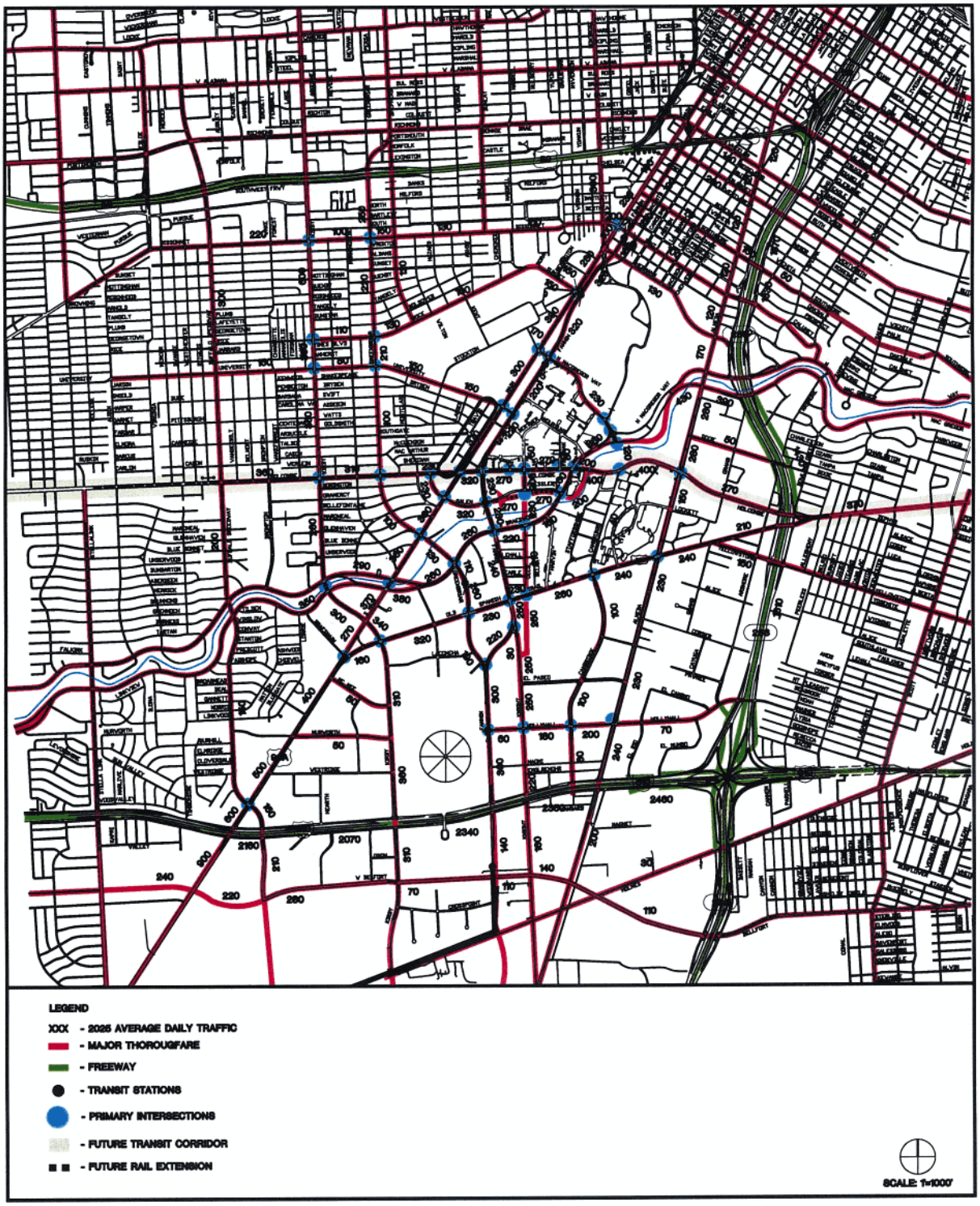
Strategy 2 System Connectivity



Strategy 2 System Connectivity With Traffic Volumes



Strategy 3. Operational Improvements



Strategy 3 Operational Improvements With Traffic Volumes

S. MATTHEW FEIL

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Upon completion of his Bachelor's Degree, Matthew plans to pursue a Master's of Engineering in Civil Engineering. His interests include smart growth, transportation planning, and roadway design and construction.

ASSESSING CRASH IMPACTS OF ACCESS MANAGEMENT: PROCEDURE AND RESULTS ALONG A SHORT ARTERIAL SEGMENT

by

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SUMMARY

The objective of this study was to conduct a crash analysis of a roadway that had been retrofitted with a raised median. The study corridor was Texas Avenue (Business State Highway 6) from two-tenths of a mile north and south of the intersecting roadway George Bush Drive (Farm to Market Road 2347). Using the "before-and-after" study method, the general finding was that the retrofit reduced the number of crashes along the roadway. Other findings will be discussed in detail in this report.

Prior to conducting the crash analysis, another primary objective of the study was to look at the quality of the available crash data. Both original crash reports and coded forms from the Department of Public Safety (DPS) were studied in detail. It was found that the overall quality of the both sources was good. The primary error in either data record was the milepost (MP) location. This error was reduced using various methods that are addressed within the body of this report.

This study is actually a smaller portion of a larger project sponsored by the Texas Department of Transportation. All of the findings of this report will be implemented in the larger project, including some of the recommendations and future areas of study.

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INTRODUCTION

As communities grow, transportation engineers and planners must adapt the infrastructure of the roadway system to meet the needs of expanding commercial and residential developments. While aging transportation facilities require routine maintenance and repairs, increased motorist volumes along roadways from community expansion will at some point exceed the capacity of these thoroughfares. Retrofitting existing roadways and/or constructing completely new facilities are two of the common methods used to address this growth issue. In either case, access management is essential to alleviating many of the issues related to city expansion.

Access management is an effective tool to efficiently increase mobility and safety while preserving the public's investment in the transportation infrastructure. The Federal Highway Administration (FHWA) defines access management as, "the process of balancing the competing needs of traffic movement and land access. Access management provides access to land development while simultaneously preserving the safe and efficient flow of traffic on the roadway system" (*1*).

Regardless of any definition, the public often wants direct access to whatever it perceives that it requires. For instance, business owners commonly believe that they need exclusive access to their business to remain economically viable. Studies have shown that this is a common misconception. One study in particular, conducted through Texas Transportation Institute (TTI), showed that consumers were more concerned with customer service and the quality and the price of the goods sold by a commercial establishment rather than direct access to the location of sale (*2*).

Access management is very important, but every type of roadway requires a particular level of entry that meets both the function of a specific type of transportation facility and to ensure the safe and efficient use of the facility. Figure 1 is a graphical explanation of the inverse relationship between mobility and land access. Freeways are an example of high mobility and limited land access. The controlled access is part of the reason that this type of facility has high capacity and high speeds, which lead to high mobility. At the other end of the continuum, cul-de-sacs provide access to individual family dwellings while offering low capacity and very low mean speeds.

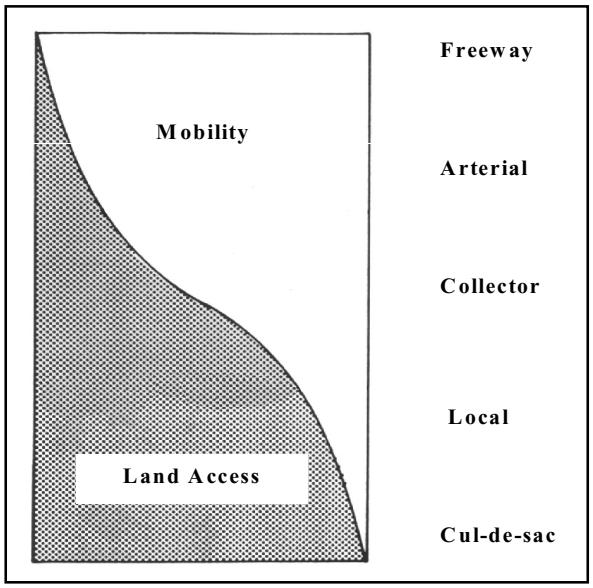

Figure 1: Mobility versus Land Access (*3*)

Access management should be studied in detail in order to provide information to state, local and commercial entities that influence roadway design. Research of both the successes and failures in access management will provide agencies, such as the Texas Department of Transportation (TxDOT), with the data to make an informed decision about how to effectively retrofit congested and inefficient roadways. In turn, less congested and more efficient facilities reduce the probability of crashes. However, as researcher study access related issues, they should further investigate the safety improvements that may be associated solely with the access treatment and that cannot be attributed to something else such as a decrease in traffic congestion (i.e., removing unsafe options that motorists invariable choose).

LITERATURE REVIEW

A crash analysis is one of the common methods used to look at the changes in traffic flow characteristics associated with new construction and retrofits to the roadway infrastructure. The before-and-after crash study is a specific type of crash analysis method. This type of analysis is typically used when investigating modifications to access. With this research approach, the researchers should analyze the quality of their data to be well aware of any limitations with the data before conducting the crash analysis (*4*).

The quality analysis process is fairly simple; however, it becomes increasingly more difficult with size and detail of the data involved. The process starts with collecting the data. Then the data is studied for inaccuracies by various methods. For police reports, the researcher must look for the obvious mistakes that can be recognized through internal and external incongruities. An example of an external incongruity would be a drawing with the incorrect reference distances.

While the best source is the original crash report (*3*), sorting through individual crash reports becomes extremely difficult for larger studies. Coded crash reports that are stored in a computer database are another option.

In the coded form, it is not as easy to find inaccuracies, but the advent of computers reduces the complications. The problem with the coded forms is that they may come in various forms, but one particular form used by the Accident Records Bureau (ARB) of the Texas Department of Public Safety (DPS) is the data stream format. The data stream format is just a line of continuous numbers that make sense when interpreted with a codebook. Initially, even with the codebook, reading and understanding one record has a substantial learning curve. This process is simplified with powerful computer programs such as SAS. SAS is a statistical analysis package that is used by the Texas Transportation Institute (TTI) to sort the data into a more legible arrangement. This new arrangement can then be read into a spreadsheet software package like Microsoft Excel. While it is possible to check for oddities in the data with a computer, the more legible format makes it easier to take a sample of the raw coded data and compare it directly to the original crash report. While the researcher of this report has not found a direct literature source to sight this method, the researcher feels this is one of the most accurate approaches when conducting a quality study on the crash data.

The primary limitation with the coded crash report is that the narrative and associated graphic are generally not encoded. This specific portion of the original crash report almost answers every question associated with the crash. Some states have encoded narratives, but it appears that the graphic coding is still beyond the abilities of state crash coding agencies (*4*). In Texas, the ARB codes have neither the narrative nor the graphic.

One of the fundamental details that all researchers must first be made aware of is that many crashes go unreported. It was found that 71 percent of automobile crashes and 80 percent of truck crashes go unreported in the state of Illinois (*3*). This is a trend by motorists to avoid increasing insurance premiums

by settling outside of the insurance channels (*3, 4*). In some cases, the drivers may also be avoiding police citations. Another study by Hauer and Hakkert in 1988 suggests the following:

- "The number of fatalities is generally known within ±5 percent of the true number.
- The number of injuries requiring hospitalization is underreported by about 20 percent.
- Only about half of all injuries occurring in accidents are reported.
- Motorists report fewer than half of all property damage only (PDO) accidents." (*4*)

The author of this report does not feel the lack of reporting invalidates the use of crash reports for crash analysis. One perspective is that the number of crashes reported should be a sample of entire number of crashes and thus representative of the whole. The other possibility is that the portion of crashes that is reported is not characteristic of the whole.

The later case appears to be typical for crash data. Crashes that do not contain injury or excessive property damage (deemed for this report as more than $1,000) are typically the incidents that go unreported (*3*). This would imply that the portion of reported crashes is not representative of the whole. One of the main goals in improving safety and reducing crashes is decreasing the severity of the crashes. Severity is regularly associated with the level of the injuries, but may also be used to describe the level of property damage only (PDO). Consequently, reported crashes are of the severity that should be studied. Furthermore, many of the severe crashes may be avoided by implementing new roadway designs, control or operational features (*3*).

In either of the above cases, the data remains useful for conducting a safety assessment. If the crashes that are reported are representative of the whole, then any associated drops in crashes for the sample should indicate a drop in the number of overall crashes. A 40 percent drop in crashes for the sample would suggest a 40 percent drop in the all of the crashes, both reported and unreported. If the reported crashes lean more towards severe crashes and are not characteristic of the whole, researchers may use the data to focus more on reducing a specific type of crash that may coincide with severe crashes, such as right-angle crashes.

All crashes that are reported are stored and may be accessed through various means and with varying degrees of information. The local reporting authority, usually the police department, will retain the reports for a period of time from 2 to 5 years (*4*). Hard copies of the reports are sent to the applicable people and agencies. The Texas DPS is one specific agency. Within this agency is the Accident Records Bureau (ARB) and its function is to code and house all of the crash records for the State of Texas. This individual agency sorts and studies the original crash reports and creates a database of information to draw crash data from. Regardless of the state, the original records are kept on hardcopy (i.e., microfilm) and commonly stored by location and date, such as county and year (*4*).

Current technologies allow for access to large amounts of crash data through computer files kept by various agencies. All states, in varying levels, generate computerized files of the crash reports (*4*). For instance, the DPS is one state agency that takes original reports and codes them into a database. As computers become more affordable smaller agencies may do the same. Some cities with large jurisdictions have already begun coding crash reports into a database format (*4*). Without a survey, the researcher were unable to ascertain any specific numbers as to whether the current trend is for local police departments to keep a database of crash report information. With the drop in price of computers and the reduction in labor costs associated with data entry and analysis, it does appear that the local agencies will move in that direction (*4*), but no further conclusive remarks on this issue will be discussed in this report.

It does not matter whether researchers use the original crash reports, coded data, or both; errors are inevitable. The crash location is one of the more typical errors. These locations may be incorrect because of inaccurate distances (the most common), misspelled street names (this includes multiple streets with

similar names), wrong block numbers and mistakes in reference to the roadway geometry (4). It helps if the researchers conducting the study are familiar with the roadway geometry of their facility under investigation. This better enables the investigators to locate and correct erroneous information.

Once the assessment of the quality and the limitations of the crash data involved are complete and understood, the data analysis and safety study may begin. One step in a crash analysis study is the generating of crash diagrams for the study corridor(s). Crash diagrams are a good way to find crash types that may be associated with a design related issue for a roadway (4). For instance, there may be a number of sideswipes or head-on crashes in a TWLTL that may be reduced by the installation of a raised median. A table of the examples of crash diagrams symbols presented by Hummer in his chapter on "Traffic Accident Studies" in the ITE *Manual of Transportation Engineering* is below in Figure 2 (4).

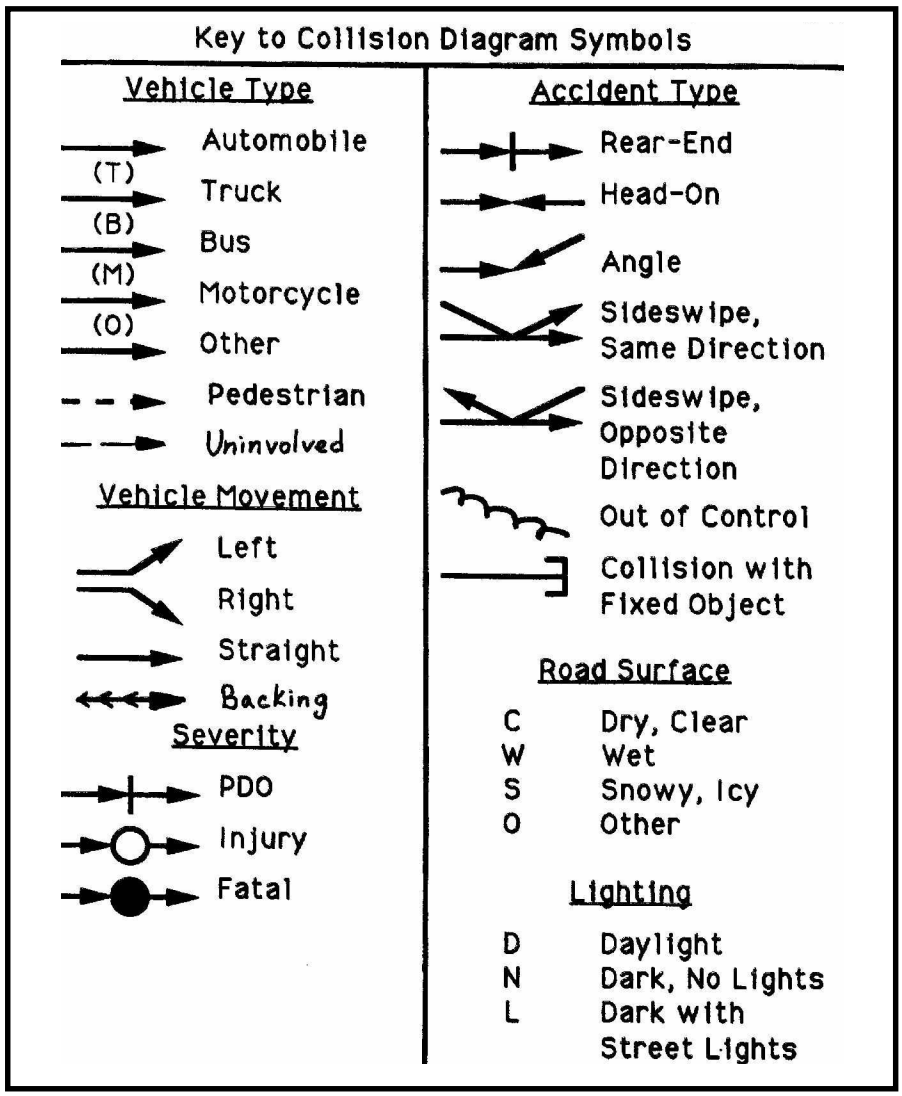

Figure 2: Crash Diagram Symbols (4)

When generating crash diagrams, researchers must decide what they desire to investigate. In some studies, researchers want to try and completely isolate a certain environment. They may look to remove

data that does not meet their restrictions with respect to the time of day or year, the location, the weather, the types of vehicles involved, or the specific driving conditions. It is essential to only remove data that that a researcher is certain he or she will not need later. It is likely that more time will be lost in re-incorporating removed data than in sorting and analyzing a larger data set (*4*).

Another method that is used in the assessment of safety along a roadway facility is conflict diagramming. "Traffic conflicts are interactions between two or more vehicles or road users when one or more vehicles or road users take evasive action, such as braking or weaving, to avoid a collision" (*4*). Traffic conflicts have served as surrogates in safety studies in place of crash analysis. It has been shown in various studies that by reducing conflicts, one reduces the number of crashes (*4*). Two-way-left-turn-lanes (TWLTL) and raised medians both decrease the number of roadway conflicts versus an undivided two-way roadway.

One of the primary findings for TWLTL and raised median treatments is that both reduce crashes significantly versus an undivided two-way roadway. One recent report issued by the National Cooperative Highway Research Program (NCHRP Report 420) has stated that research has shown raised medians to be safer than TWLTLs (*5*). In particular, the raised median is safer than TWLTL on roads with larger average daily traffic (ADT) and that contain concentrated sources of entering traffic (*6*). Raised median treatments reduce crashes the most for facilities with volumes greater than 20,000 vehicles per day (vpd) (*7*).

For this report, a conflict analysis will not be conducted, but the author sees the merits of such a study and believes that the reduction of conflict points by controlling access with raised median treatments will reduce crashes. The author will refer to conflict point reductions in the Results section.

A before-and-after study is used in many crash analysis studies using crash reports. This type of investigation utilizes data from equivalent lengths of time before and after a retrofit. There is a period of adjustment following any construction. That period is of particular concern, because the drivers will not be driving under the standard driving conditions that must be studied (*3*). Furthermore, the natural trends in motorists and how they drive may have already started to decrease crashes in the before period. While the author of this report does not find that likely, it is something that must be considered in any study. Consequently, the before-and-after periods should be long enough to rule out or show these trends. It is common to have periods from 3 months to 1 year (*3*), but longer periods may help address issues such as regression-to-the-mean. Researchers should avoid periods longer than 4 years, because it becomes increasingly difficult to guarantee similar before-and-after conditions with the exception of roadway redesign (*4*). For example, changes to commercial centers such as adding a gas station may include access modifications that will affect traffic flow characteristics. Researchers must also consider whether there are any other factors that are changing in conjunction with the focus of their research that may impact any changes that they wish to investigate. In particular, the study of reductions in crashes along a roadway that has be redesigned with a raised median in place of a TWLTL may be complicated if the facility was widened simultaneously with raised median installation (*7*).

In studying traffic flow changes resulting from a retrofit, it is helpful to find a roadway that is virtually identical in geometry and in ADT counts to the study corridor for use in a before-and-after study. The goal of using a control section is to look at whether the changes can be attributed to the retrofit, or whether the traffic flow characteristics where changing regardless (*7*). If the control section shows similar changes in crash reduction as the redesigned study corridor, then it is less likely that there is a casual relationship between the retrofit and the crash reduction. The author did not include a control section in his study, but further comments will be made in the Results and Recommendations sections of this report.

PROBLEM STATEMENT

This research further studied the crash impacts associated with the installation of a raised median treatment where a two-way-left-turn-lane (TWLTL) was previously located. The particular study site was along Texas Avenue in College Station, Texas. The researcher looked at the original crash reports filed by the police from a time period before and after the installation of the raised median. One specific focus of this study was to investigate crash reporting. Crash report analysis is one of the main methods used to conduct a safety impact study. Consequently, the researcher had to first look at the recording accuracy and any errors related to crash report coding.

Time constraints restricted the research to a "before-and-after" study only. However, it is the intent of the researcher to include a crash analysis of the construction period in the findings of the larger project sponsored by the Texas Department of Transportation (TxDOT).

RESEARCH OBJECTIVES

A list of bulleted objectives for the research is listed below.

- Summarize the differences in the original crash reports and Department of Public Safety (DPS) records, including coding efforts for the original report by the police officer.
- Analyze crash reports from January of 1993 through June of 2000 for the site in question and categorize crashes by type and location and summarize any changes in crashes. Look specifically at the periods prior to construction of the raised median and after the installation.
- A collision diagram of the study corridor will be created and the number of crashes by crash type will included by location.

PROCEDURE

This portion of the report describes the steps in which the research was performed.

Step 1: Literature Review

The goal of the literature review is to establish the information base for the study. The researcher searched through various resources and collected relevant information on, but not limited to:

- Access management
- Median treatments
- Roadway safety
- Crash report analysis
- Crash analysis
- Statistical analysis

The articles found under these topics will aid the project with examples of previous applicable studies and results to better focus and support the proposed research. An exhaustive search has already been completed. A list of some of the current texts and articles that relate and are applicable to the proposed study are listed below. Note that some of the sources will also be listed in the reference section.

- Meuth, H.G., and C.V. Wootan. *A Median Study of Baytown, Texas*. Progress Report Project 2-8-58-8 (HPS 1-27-I). Texas Transportation Institute, College Station, Texas. August 1963.

- Eisele, W.L., and W.F. Frawley. *A Methodology for Determining Economic Impacts of Raised Medians: Final Project Results*. Research Report 3404-4. Texas Transportation Institute, College Station, Texas. October 2000.
- Bonneson, James A., and Patrick T. McCoy. *Capacity and Operational Effects of Midblock Left-Turn Lanes*. NCHRP Report 395. Transportation Research Board, National Research Council, FHWA, 1997.
- Gluck, Jerome, et al. *Impacts of Access Management Techniques*. NCHRP Report 420. Transportation Research Board, National Research Council, FHWA, 1999.
- Parsonson, Peter S., and Christopher A. Squires. Accident Comparison of Raised Median and Two-Way Left-Turn Lane Median Treatments. In *Transportation Research Record 1239*, TRB, National Research Council, Washington, D.C., 1989, pp. 30-40.
- Dissanayake, Sunanda, et al. Should Direct Left Turns from Driveways Be Avoided? A Safety Perspective. *ITE Journal*, June 2002, pp. 26-29.
- Bernhardt, Kristen L. Sanford, and Mark R. Virkler. Statistical Concerns for High-Crash Locations. *ITE Journal*. January 2002, pp. 73-76.
- Griffin, Anna. *Literature Review: Methods and Procedures for Conducting Crash Analysis Studies*. CVEN 617 Paper. Texas A&M University: College Station, Texas, November 19, 2001.
- McShane, William R., and Roger P. Roess. *Traffic Engineering*. Prentice Hall, NJ. 1990.
- ITE (2000). *Manual of Transportation Engineering Studies*. Edited by Robertson, H.G., J.E. Hummer and D.C. Nelson. Institute of Transportation Engineers, Washington, D.C.
- *Access Management, Location and Design*. NHI Course No. 133078. National Highway Institute, U.S. Department of Transportation, Federal Highway Administration. April 2000.

Step 2: Proposal

This step of the research project includes this document and an oral presentation. At the time of the submittal of this document, the oral portion will have been completed. The oral portion of the proposal was on June 14, 2002 at 9 a.m. Copies of the visual presentation may be requested by e-mail to j-miles@ttimail.tamu.edu. The proposal draft was completed after addressing formal comments from the question and answers section of the oral presentation and any other concerns with respect to the proposed study and submitted on June 19, 2002 by 5 p.m.

Step 3: Data Collection

The data collection consisted of gathering the original police crash reports kept by the Accident Records Bureau (ARB) of the DPS in Austin, Texas and the average daily traffic (ADT) data for the site to be studied. The researcher also obtained the coded data of the original crash reports that is generated by the ARB. A flowchart for the data collection process is below in Figure 3 and a current list of the people and agencies contacted for the information is in Table 1.

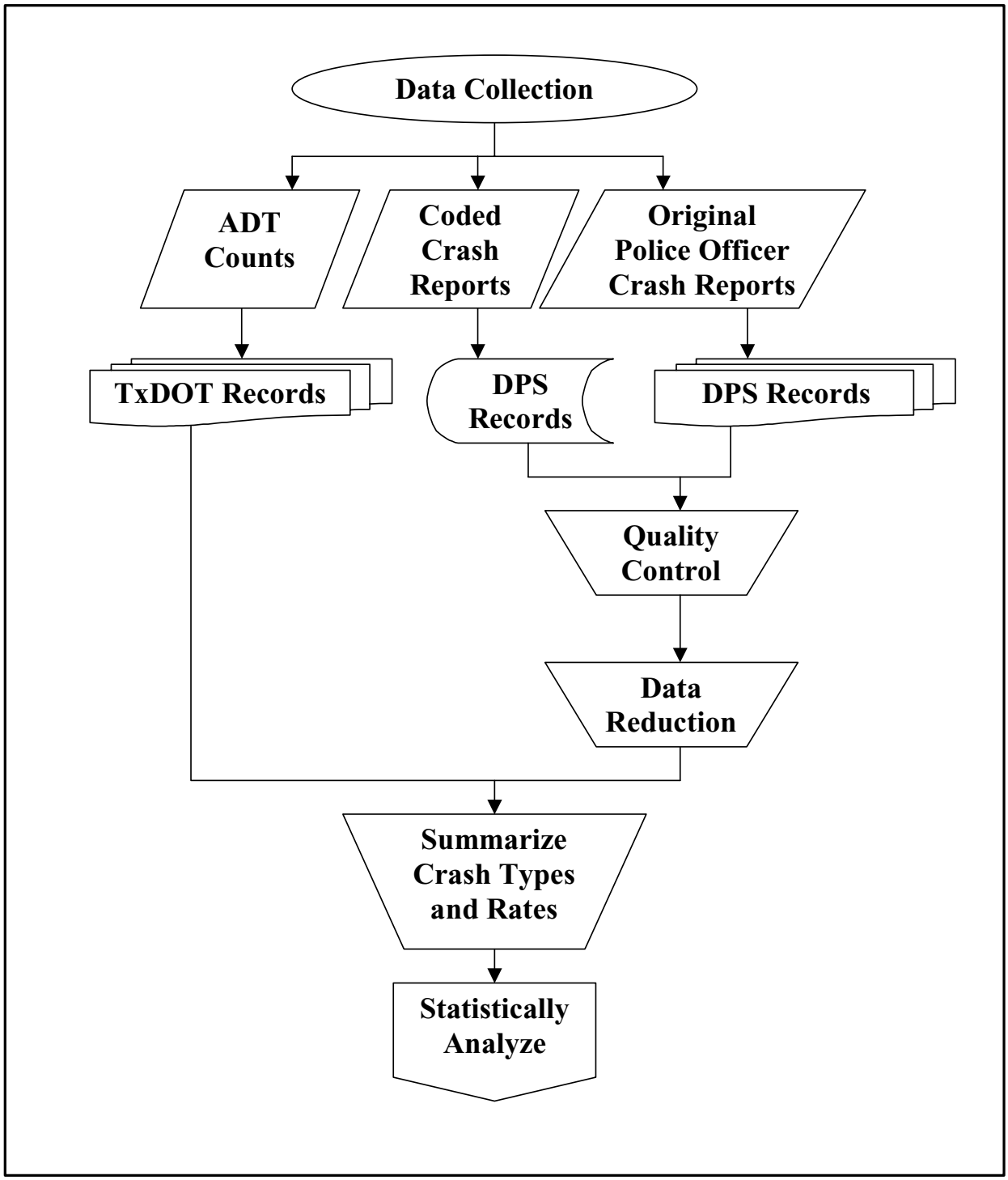

Figure 3: Data Collection Flow Chart

Table 1: Contact Information

Contact	Organization
Robert Appleton	Texas Department of Transportation (TxDOT)
Randy Grones	Texas Department of Transportation (TxDOT)
Jay Page	Texas Department of Transportation (TxDOT)
Michael Parks	Metropolitan Planning Organization (MPO)
Danny Morris	Texas Transportation Institute (TTI)
Julie Arldt	Texas Department of Public Safety (DPS)
Deborah Cartwright	Texas Department of Public Safety (DPS)
Laura Gomez	Texas Department of Public Safety (DPS)

Step 4: Quality Control

Clear and proper crash report analysis is essential to the success of a crash analysis study (*4*). The data from the reports was categorized by location, type of crash, and severity. While the summaries of the crash reports were tallied into absolute values and crash rates, the researcher studied the quality of the original reports in areas such as location, crash type and probable cause. Summaries of the data from the original crash reports were compared to the coded crash records kept by the DPS to investigate the quality control in reference to crash report coding. The researcher used the direct wording of the original reports to help revise any of the coded data and to make informed concluding remarks in this report.

Step 5: Summaries and Statistical Analysis

This section of the project focused on summarizing the absolute numbers by crash type and the crash rates taken from the reduced crash report data and the ADT data, which were then be used to conduct statistical analyses. The range of the number of reports per year was from 19 to 92 for this study. In addition, the absolute number and the associated crash rates broken down into crash type further lowered the sample size for a given year. In 1995 the State of Texas stopped requiring that the ARB code every crash report. The ARB only codes incidents that involve estimated damages in excess of $1,000 and when injuries are documented. Hence, 92 reports were coded in 1994 and only 37 were coded in 1996. The change in the coding of crash data was considered in the analysis and the researcher removed any record from the studied timeframe prior to the new coding requirement that would not have been coded under the new guideline.

It is the intention that summary remarks and conclusions were made in reference to the accuracy of the reduced crash report files. It is the hope that the findings will aid future researchers who may use crash reports to study safety impacts.

Step 6: Write Preliminary Report (First Draft)

The preliminary report addressed the current results and conclusions that were presented during the oral presentation (Step 7) and was comprised of a Microsoft PowerPoint presentation, a summary of the crash reporting process and various Microsoft Excel data analysis spreadsheets. The summary of the crash reporting process will be explained in detail later in the Results and Conclusions sections of this report.

Step 7: Present Findings (Oral Presentation)

An oral presentation was conducted on July 31, 2002. The presentation included formal conclusions and supporting material. A question and answer session followed the presentation. All of the comments were addressed and/or answered in this report.

Step 8: Write Report (Final Draft)

The final report was submitted on August 8 by 5 p.m. The general parts of the final report will include the following sections:

A. Table of Contents
B. Introduction
   This section gives a terse summary of issues that directly lead to this research proposal.
C. Literature Review
   This section gives background from previous reports, studies and texts that addressed issues analogous to the scope of this research project.
D. Objectives
   The objectives are clearly defined as in the above "Objectives" section.
E. Procedure
   A detailed overview of the entire project from start to finish is discussed in this section.
F. Results
   The statistical results and data summaries are highlighted in this section. The author also remarks on the analysis.
G. Conclusions
   This section lists and/or expresses any and all conclusive remarks that can be established by the data, statistical analyses and empirical inferences by the researcher based upon personal experience and applicable literature resources.
H. Recommendations
   The author discusses the overall scope of the findings and the impacts associated with them in this section. Furthermore, the areas requiring further research and study are addressed in this section.

RESULTS

Quality Control Study

The summary findings for the quality control study include:

- Summary of crash reporting process
- A step-by-step outline of the crash record reporting process with flowchart diagram
- Summary of crash report errors

Summary of Crash Record Reporting Process

The crash reporting process in the State of Texas was the specific focus for this portion of the study. The first step is the crash report. Traditionally, the reports are written at the scene of the crash. Some of the crashes are reported in cases without police involvement. Those reports are filed directly to the police department or to the DPS by one or more of the participants for differing reasons. The two most common reasons for not reporting a crash were related to insurance and to hit-and-run crashes.

There are two standard options for reporting crashes in the State of Texas. The most common is that an investigating officer files a crash report. If a crash occurs and there is no law enforcement to file a report or if the investigating officer chooses to not submit a report, the motorist(s) involved in the crash may fill out a "blue form". The blue form is submitted directly to the DPS (see Appendix A for a copy of the blue form). No detailed investigation was conducted on the private reporting with blue forms. It is the assessment of the author that such a report would be biased compared to an uninvolved police officer. If there is an investigating officer and a report is filed, the report is processed both locally and by the state.

Locally in College Station, Texas, the police department uses computer processors in their crash recording efforts. First, the hardcopy from the reporting officer arrives at the police department. Once the investigation is complete and the report is not being used for legal reasons, the report is coded into a computer and the hardcopy is kept on file for three years. The coded information is not elaborate, but can be used to give some indication as to the state of the city and its motorists. Some of the information provided to the researcher included the date, the major roadway and the severity of the crash. This information did not contain the amount of information nor the detail for this study. Furthermore, their database did not go back far enough to the period prior to the median installation. The final step for the local police department is to send a copy of the original report to the DPS ARB office in Austin, Texas. A copy of a blank crash report form used by Texas law enforcement and of a print of data offered locally by the College Station Police Department (CSPD) are located in Appendix A.

The crash reporting process on the state level takes place in the ARB. As the crash reports arrive directly from the police department or from the individual motorists, they are sorted into two primary groups. For this study, the author has labeled these groups, Group I and II. Group I contains all of the PDOs under $1,000 in estimated damages. Reports in Group I are filed, but no further processing is conducted on them within the ARB. Group II contains all of the PDOs in excess of $1,000 in estimated damages and crashes containing injuries. This group has a far lengthier processing period of approximately 18 months. All of the reports from Group I and II are placed on microfilm, stored by location (county) and date and retained for 10 years. Both Group I and II files are transferred to microfilm after all of the records of a particular year have been completely processed.

A step that occurs prior to the sorting is the coding of applicable data to a driver's individual driving record/traffic history. Both of these groups contain data that may be added to a motorist's individual driving record. Crashes that involve injuries, PDOs in excess of $1,000, and/or traffic violations are coded to a driver's traffic history. Crashes that do not fall into the three previous categories are not

inserted into a person's driving record. All records in Group II are coded to the applicable traffic histories. The only case in which a Group I crash would be coded to a driver's record would be in the case of a PDO crash of less than $1,000 that involved a traffic violation.

Group II reports go through a coding phase that will ultimately be uploaded into the DPS mainframe. The longest stage occurs with the handwritten coding. During this phase, the coding of the records is broken down into different stages. The reports are coded in an assembly line fashion with ARB staff coding only certain information as the record passes through. For instance, a specific person would be tasked to code only the driver data onto a hardcopy sheet. Once the coding phase is complete, the sheets are input through a dual data entry method. While in the earlier stages, there are varying levels of checks to minimize data entry errors; the dual data entry method is one of the best ways to reduce mistakes. Two different people input the data into a computer; the computers compare the records; matching records are set aside for the final mainframe upload; and the records that do not match are set aside for checking. Out of 307 records studied by the researcher only 8 errors (2.6 percent) were recorded between the data contained by the DPS mainframe and the data provided by the original police reports. Of the 2.6 percent coding mistake, each error recorded dealt with the original written coding stage by someone who was unfamiliar with the peculiarities of College Station. For example, FM30 is Harvey Road and FM2818 is Harvey Mitchell Parkway. There was a crash that occurred at Harvey Mitchell Parkway, but was coded for Harvey Road. For someone from College Station, the error is obvious, for someone from Austin there may be no question. As a result, the report is coded incorrectly. Again, there were not many errors of this nature. Of those that did occur, the original crash report removed any doubt as to the location of the crash. There is a more in depth description of the process in an outline format in Appendix A. Figure 5 below is a graphical representation in a flowchart format of the above crash reporting process.

One further note should address the fact that the ARB is further working to improve the crash reporting system with the Crash Record Information System (CRIS). The DPS is working with TxDOT to fund this new system that will help automate the reporting process and make the information more accessible in a timely manner. Members of the ARB work hard to help improve transportation engineering and planning by getting the crash data to differing agencies that try to make informed infrastructure safety improvements. CRIS will further enhance the abilities of the ARB and of any other agencies that require such information. A copy of a CRIS Newsletter is located in Appendix A.

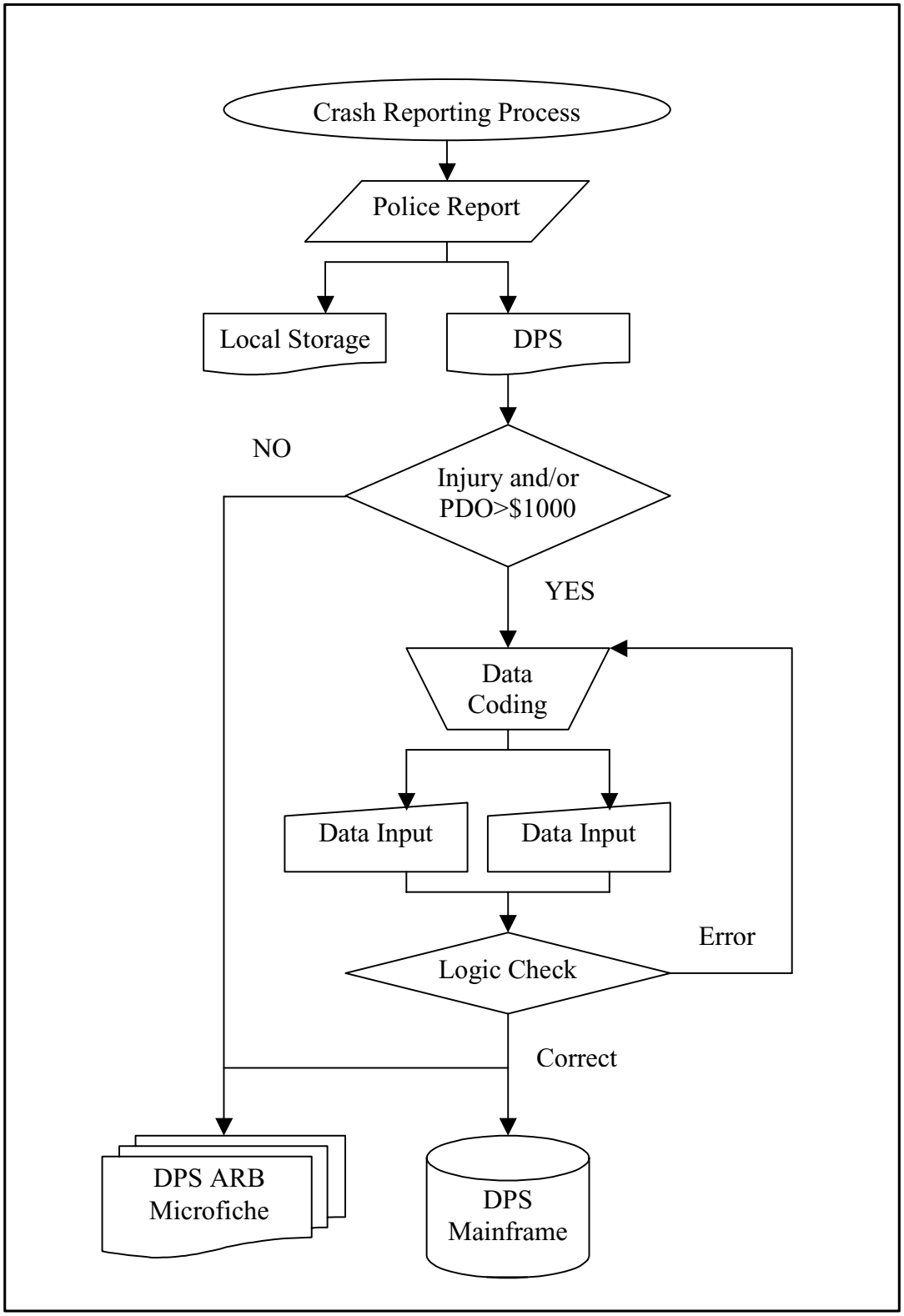

Figure 5: Crash Reporting Process Flowchart

Summary of Crash Report Errors

The consistent errors that occur in the crash reporting process originate with the police report. This does not mean to imply any form of carelessness with regard to the police investigating the crash, as human error is expected to some degree.

Common errors that were expected, but could not be verified, were related to the exact number of vehicles involved and the true intentions and compounding causes that attributed to the crash. In one instance, a vehicle was traveling southbound in the outside lane approaching traffic congestion associated with the Texas Avenue and George Bush Drive signal. The motorist attempted to move into the adjacent lane to avoid the longer queue of cars and sideswiped another vehicle. No comments or inferences were made by the drivers or the reporting officer. Did the driver miss his/her blind spot? Was the other driver speeding or changing lanes? Did either party have any additional stimuli distracting him/her from driving? Answers to these questions and others are normally unknown, because both eyewitness reporting and providing false information for the sake of avoiding incrimination contribute to data errors (*8*).

The main areas of concern for crash reporting errors stem from the location, orientation, crash type and severity of the incident. After carefully looking through over 300 individual crash reports, the researcher feels confident in saying that location inaccuracies are the most prevalent. To be more specific, the police occasionally reported erroneous distances from reference points. For example, one police officer recorded a crash 100 feet south of Dominik Drive. After further investigation, the researcher was able to determine through other reported information that the crash occurred 600 feet south of Dominik Drive Orientation, crash type and crash severity data overall appeared to not contain errors. Errors of this type were looked at from the perspective of internal discrepancies within the report itself. For instance, it would be considered an error if a police officer had coded the crash severity with a death, but he/she did not record a death with any one of the drivers or their passengers. Fortunately, the police reports studied for this project contained more than enough additional information that enabled the researcher to clarify and verify any perceived discrepancies in the reporting of the location, orientation, crash type or severity of a crash.

Another error is in relation to the location of the crash along the main roadway as presented by the coded data from the DPS. This error also occurs in the handwritten coding stage; however, this fault cannot be solely attributed to the ARB, nor is it considered a significant problem by the researcher. First of all, most of the error is a result of the fact that the researcher needs accuracy to be within 1/100 or 1/1000 of a mile. It would be a daunting, if not impossible, task for the coders to relay that type of accuracy by handwritten methods for the entire State of Texas. The researcher created a calculator spreadsheet in Microsoft Excel to simplify the process. When comparing the mainframe data with the recalculated data, the researcher rounded his new values to 1/10 of a mile, the same accuracy offered by the DPS coded data. As presented in Table 2, this error was around 18-24 percent. If the researcher removes the errors associated with wrongly coding the primary roadway, the error drops to as low as 5 percent.

The errors calculations in regard to the quality control aspect of the study were limited to the miscalculations in the milepost location of the crashes along the main roadway. The mean error and the standard deviation in the table below are in miles. Therefore, 0.001 is equivalent to 5.28 feet.

Table 2: Distance Error Calculations

Roadway	Raw Data		Revised Data	
	Texas Avenue	George Bush Drive	Texas Avenue	George Bush Drive
Total Entries	310	29	173	20
Correct Entries	254	22	147	19
Incorrect Entries	56	7	26	1
Percent Error	18.1	24.1	15.0	5.0
Mean Error	-0.008	0.322	-0.009	0.007
Standard Deviation	0.052	0.851	0.045	0.032
Variance	0.003	0.723	0.002	0.001

In the above calculations, the revised data section refers to the reduction in the original raw data. The reduction included removing all 1995 through 1997 data (construction data and data during the year of the ARB coding criteria change) because time restrictions limited the analysis to only before-and-after calculations. Furthermore, the researcher removed data that was incorrectly input into the original police report and data that the DPS coded for the wrong roadway. These cases dealing with the DPS coding the wrong roadway were not included because it was difficult to ascertain the intent of the coding official. The coding of a wrong roadway occurred approximately 5 percent of the time.

The researcher looked at coded data collected locally by the College Station Police Department. The local data files were not used because the information was not detailed enough for this study's purposes.

Crash Analysis

The crash analysis in the results section will be broken down into these categories:

- Crash Diagrams
- Crash Rates
- Summary Statistics
- Statistical Significance

One of the earlier comments presented in the Literature Review was that a portion of crashes go unreported and that a researcher should keep this in mind in a crash analysis. For this report, it will be assumed that the crashes are not representative of the whole and that most of the unreported crashes are predominately PDOs less than $1,000.

The author had intended to find and to collect data from another roadway for use as a control section. Time was the key factor for not conducting a control section study. However, it should be noted that the researcher did include an adjacent section of roadway that remained almost completely unchanged. This section of roadway is south of Dominik Drive and located from MP 6.190 to 6.255. The raised median installation stopped at MP 6.190. The ADT for south of George Bush to Dominik and the adjacent section are the same. The only change along this section to the geometry was the adding of a third lane for SB traffic, which the lane closes from MP 6.190 to 6.255. While this section of road cannot be used

as a true control section, the author did find it useful for his research and his findings will be discussed later in the results section.

Crash Diagrams

Figures 6a and 6b are two examples of the researcher's crash diagrams for this study. Each tally mark represents one crash. The researcher used techniques provided by ITE (4). A copy of a table out of the chapter on, "Traffic Accident Studies," of the ITE *Manual of Traffic Engineering* is in Figure 2 in the Literature Review section of this report. This table was essential in the diagramming process.

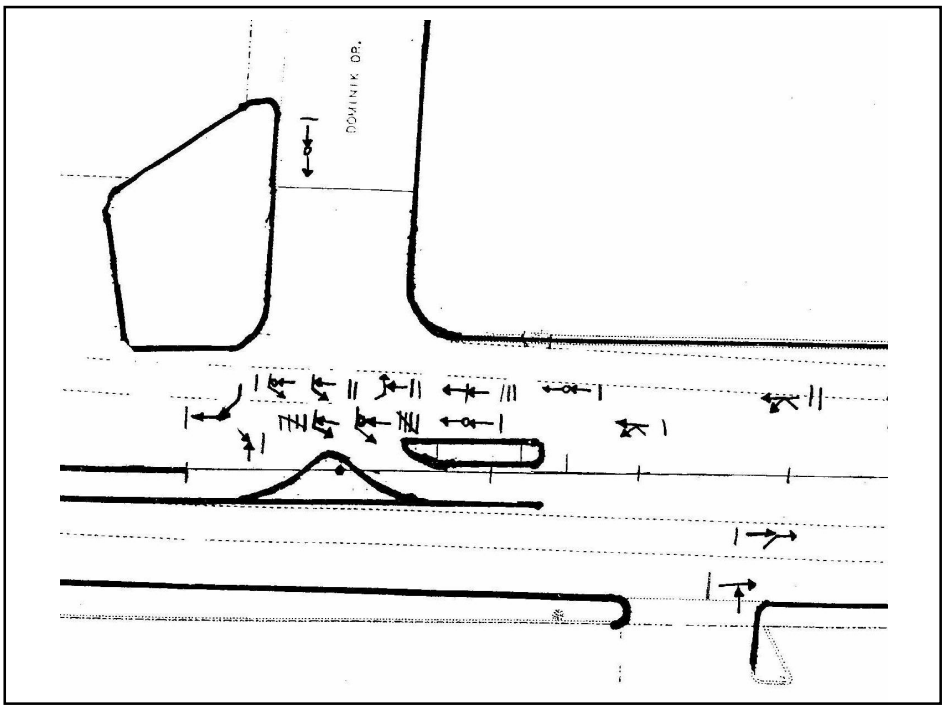

Figure 6a: Before Period at Texas Avenue and Dominik Drive

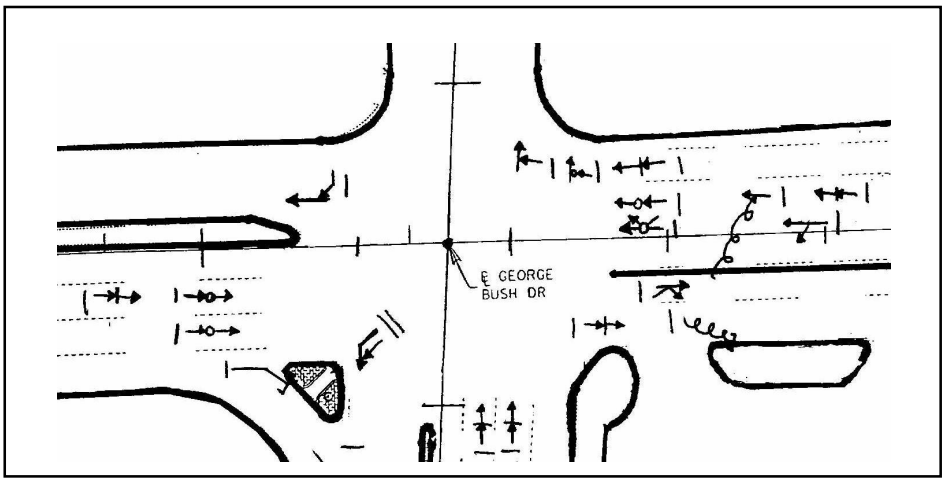

Figure 6b: Before Period at Texas Avenue and George Bush Drive

Crash Rates

The values below are the crash rates associated with the study corridor. The intersection rate is for the Texas Avenue and George Bush Drive intersection and the roadway section is from MP 6.100 to MP 6.255, the southern end of the study corridor just south of George Bush along Texas. The starred (*) values in the table indicate that the years run from July of the specific year shown to June of the next year. For example, 1998* stands for the time frame from July 1998 through June 1999. The researcher chose this time frame to avoid collecting crash data associated with the end of the construction along the study roadway. In addition, the after period comprised of the time from July 1998 through June 2000.

Also, included in Table 3 is a percent change value. This value shows the percent change from the before period to the after period. For instance, there was a 70 percent reduction in the intersection rate from the before period rate.

Table 3: Crash Rates

Time Frame	Intersection	Roadway Section
1993	1.7	1124
1994	2.4	891
1998*	0.5	130
1999*	0.7	314
Before	2.1	1005
After	0.6	225
Percent Change	-70	-78

- Indicates that the timeframe goes from July of that year through June of the next.

The formulas used to calculate these rates and the percent change values are as follows:

$$RSP = \frac{1{,}000{,}000C}{365TV}$$

$$RSEC = \frac{100{,}000{,}000C}{365TVL}$$

$$\%Change = \frac{A-B}{B} * 100$$

RSP = The rate of the spot (intersection)

RSEC = The rate of the roadway section

C = Total number of crashes for the associated location and time frame

T = Time frame in years

V = Annual Average Daily Traffic (AADT) counts entering the study location

L = Length of the section of roadway under investigation

A = Value of the after rate/absolute number

B = Value of the before rate/absolute number

The AADT values used for the above calculations are in Table 4a and were retrieved by Mr. Danny Morris at TTI. These values originated from TxDOT data collection efforts. The values for the entering volumes listed in Table 4b were calculated assuming a 50/50 directional split. These values were used to formulate the total entering volumes for the intersection rate calculations.

Table 4a: AADT Counts

Year	Texas	George Bush	Texas
	MP 5.7	MP 3.232	MP 6.056
1993	40,000	22,000	39,000
1994	42,000	22,000	41,000
1998*	38,500	26,500	42,000
1999*	43,000	28,000	46,500
Before	41,000	22,000	40,000
After	40,750	27,250	44,250

- Indicates that the timeframe goes from July of that year through June of the next.

Table 4b: Entering AADT Counts

Year	Vehicles Entering the Intersection of Texas and George Bush			
	EB	WB	NB	SB
1993	11,000	11,000	19,500	20,000
1994	11,000	11,000	20,500	21,000
1998*	13,250	13,250	21,000	19,250
1999*	14,000	14,000	23,250	21,500
Before	11,000	11,000	20,000	20,500
After	13,625	13,625	22,125	20,375

- Indicates that the timeframe goes from July of that year through June of the next.

Summary Statistics

The values found in the following tables are the summation of the exact values from the DPS coded crash data and represent any manual corrections to erroneous values discovered through intense researcher investigation. The tables are broken down by the years being studied and include the before-and-after periods. The next categorical separation was by focus area. The focus areas are:

- The intersection of Texas Avenue and George Bush Drive
- The intersection of Texas Avenue and Dominik Drive
- The roadway section south of George Bush Drive along Texas Avenue from MP 6.100 to MP 6.255
- The entire Texas corridor being researched from MP 5.82 to MP 6.255

Summaries of the data and the findings are presented in this section of the report. All of the focus areas and their respective data are represented in the Appendix B.

Table 5a: General Summary of Crash Data for the Entire Study Corridor

Time Period	Total			
	Crashes	Vehicles	People**	Injuries
1993	69	149	217	36
1994	82	191	352	45
1998*	19	44	47	19
1999*	28	64	78	32
Before	151	340	569	81
After	47	108	125	51

*Indicates that the timeframe goes from July of that year through June of the next.
**Indicates that the values after June 1995 do not include non-injured passengers.

Table 5b: Crash Data by Crash Type for the Entire Study Corridor

Time Period	Total					
	Rear-Ending	Sideswipe	Right-Angle	Head-On	Single Vehicle	Other
1993	41	6	20	0	1	1
1994	52	2	23	1	2	2
1998*	10	2	6	0	1	0
1999*	14	0	11	0	3	0
Before	93	8	43	1	3	3
After	24	2	17	0	4	0

*Indicates that the timeframe goes from July of that year through June of the next.

The author generated ratios of some of the findings to add further depth to this study. In particular, he focused on how injuries compared to different factors, such as injuries versus the number of vehicles involved. The ratios are presented below in Tables 6a to 6c.

Table 6a: Ratio of Injuries to People Involved

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	0.15	0.16	0.13	0.14
After	0.41	0	0.36	0.41
Percent Change	177	-100	183	187

Table 6b: Ratio of Injuries to Vehicles Involved

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	0.27	0.22	0.18	0.24
After	0.48	0	0.42	0.47
Percent Change	77	-100	132	98

Table 6c: Ratio of People to Vehicles Involved

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	1.84	1.41	1.43	1.67
After	1.17	0	1.17	1.16
Percent Change	-36	-100	-18	-31

Pie charts have been created that breakdown the injuries in the before-and-after period by severity. Figures 7a and 7b correspond with Tables 7b to 7e.

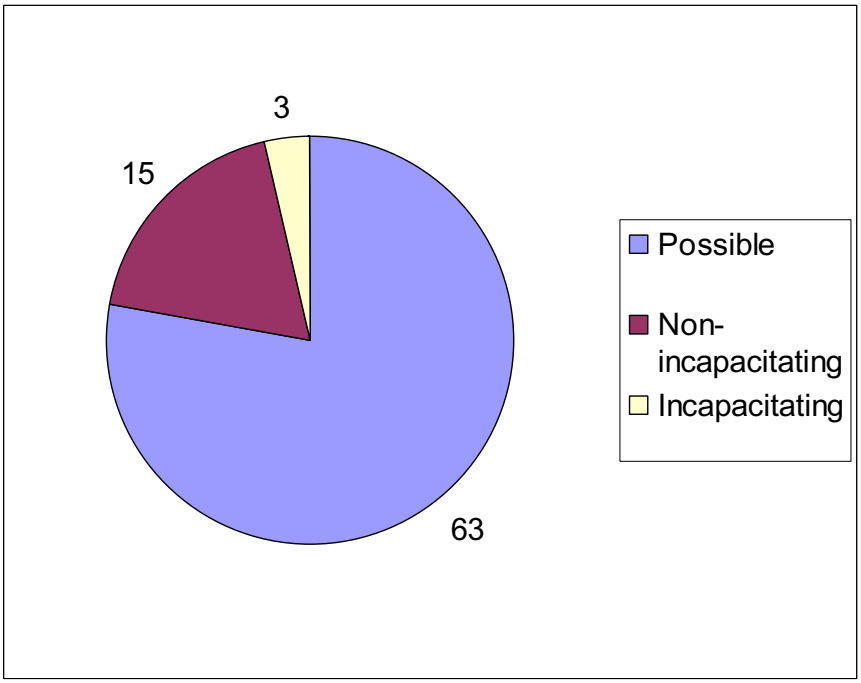

Figure 7a: Before Period Injuries for the Entire Study Corridor

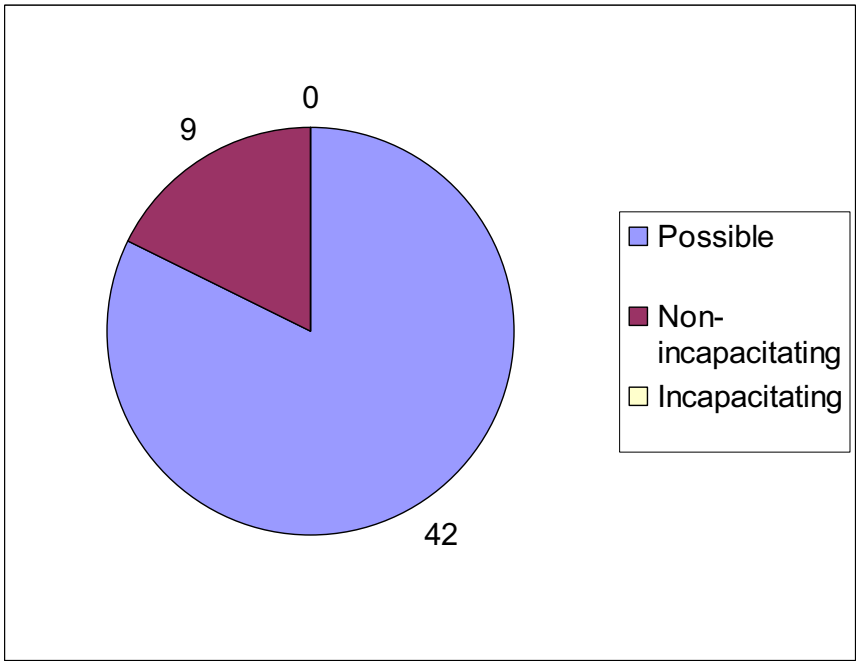

Figure 7b: After Period Injuries for the Entire Study Corridor

As shown in Figure 7b, there were no incapacitating injuries in the after period. The author did not add fatalities to Figures 7a and 7b because there were no fatalities for the before-and-after periods.

Statistical Significance

The statistical significance testing was conducted using a significance formula commonly used in safety analyses as introduced in the text *Traffic Engineering* (3). In all of the following tables, the percent change formula, introduced in the "Crash Rates" section was used. It should be noted that this significance test could not be performed for the ratio of variables generated in this section of the report. Furthermore, regression-to-the-mean was studied and not considered to be a concern with the data presented in this reports findings. The significance formula used is listed below:

$$Z = \frac{|A - B|}{\sqrt{A + B}}$$

Z = The Z-value is looked up in a normal distribution table to find the associated *p*-value.

The values for A and B were defined in the "Crash Rates" section of this report. The *p*-value is shown in the tables below. The *p*-value was generated using the Microsoft Excel function, "NORMSDIST(Z-value)," and the Z-value calculated from the above significance formula.

The most obvious finding was the total reduction in the number of reported crashes. The decrease ranged from 56 percent in certain locations to as high as 69 percent for the entire corridor (see Table 7). While both extremes were found to be statistically significant, the researcher believes further analysis is warranted for reasons that are addressed in the remaining comments in the Results section

.

Table 7: Crashes

Time Period	Texas Corridor	Texas & George Bush	Texas & Dominik	Test
Before	151	94	16	16
After	47	31	0	7
Percent Change	-69	-67	-100	-56
***p*-value**	1.00	1.00	1.00	0.97

Rear-ending, sideswipe, right-angle and head-on crashes are of particular interest to the researcher because reductions in these types of crashes should be associated with the installation of a raised median in place of a TWLTL.

While TWLTLs decrease the number of roadway conflict points by removing turning traffic from the main lanes, it is a fact from the author's own driving experience that motorists will take advantage of the unrestricted TWLTL. For example, drivers have un-prohibited access to private drives and local streets and will abruptly pull into the TWLTL immediately following a signalized intersection. Along the study corridor, this would be equivalent to people attempting to turn left onto Dominik Drive from the SB traffic on Texas Avenue. With the two left turn lanes on NB Texas Avenue, there is less than 150 feet of SB turn lane to Dominik and less than 150 feet of roadway between the intersection and the turn lane. Traffic could buildup into the signalized intersection causing a rear-ending; a rear-ending could occur as a result of a motorist on George Bush turning right going SB onto Texas and trying to cut across traffic when SB Texas has the right-of-way (ROW); or a rear-ending could occur because of SB, left-turning, George Bush vehicles that try to access Dominik. While these situations more than likely happened, there was no supporting data. This does not rule out the scenario for the study location in the before period or any other similar study location that may be done by others.

However, there was useful data gathered on rear-endings in general. For the study corridor, the rear-endings were all attributed to the intersection of George Bush Drive and Texas Avenue. This conclusion was based on the actual police report information. The rear-endings resulted from the congestion associated with the signalized intersection. The author believes that the road widening, that also took place at the same time as the raised median retrofit, may have reduced roadway congestion. Considering that the rear-endings were influenced by congestion, the widening of the facility may have impacted the 74 percent decrease (see Table 8a) in the number of rear-endings that took place within the study site.

Even though the data does not support the idea that the raised median impacted this safety-related issue, the researcher does believe that the raised median did have an effect. Videotaping and driver surveys would have aided in the testing of this hypothesis, but are not possible because of time limitations. The time limitations refer to both the fact that there was not enough time to conduct such a study, but also, that at this point, neither video footage nor accurate survey comments may be attained in reference to the before period. The researcher thinks that the raised median reduced the motorist routing options and theoretically may have influenced safer routing and driving decisions. This idea will be discussed more when addressing right-angle crashes.

Table 8a: Rear-Ending

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	82	0	4	93
After	21	0	1	24
Percent Change	-74	0	-75	-74
***p*-value**	1.00	N/A	0.91	1.00

Another scenario is that a sideswipe would occur as result of uncontrolled access. This specific crash type did occur. There was a motorist that exited a private drive south of Miliff Road and proceeded to use the TWLTL to travel NB on Texas Avenue. Using the TWLTL, the driver gained speed to enter the main inside lane and sideswiped another vehicle. This specific location was not included in the raised median retrofit and a virtually identical crash occurred in both the before-and-after study timeframes. Sideswipes decreased by 75 percent for the entire corridor and this drop had a corresponding *p*-value of 0.97 (see Table 8b).

Table 8b: Sideswipe

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	2	0	3	8
After	0	0	1	2
Percent Change	-100	0	-67	-75
***p*-value**	0.92	N/A	0.84	0.97

Right-angle crashes are another crash type that is usually associated with more severe injuries and property damage. There was a 60 percent drop in right-angle crashes for the entire corridor (see Table 8c). One concern appeared when the researcher separated the crashes of this type into smaller portions of the study corridor. There was the obvious 100 percent reduction at Texas and Dominik, but there was a 33 percent increase at the adjacent signal, George Bush and Texas. This suggests that crash migration has taken place.

One of the initial issues of focus was the left-in/left-out closure at Texas Avenue and Dominik. The researcher hypothesized that there would be a variety of crash types associated with this location; however, after looking at each record individually, the researcher could only attribute right-angle crashes to this location. The installation of the raised median produced a 100 percent reduction in this crash type. While that is an ideal result, the researcher is not confident in stating that the crashes related to this intersection have been reduced 100 percent. Other crash types such as rear-endings and sideswipes could not be attributed to this location, but the author believes that the associated conflicts did occur in the before period and were either reduced or removed altogether in the after period. Furthermore, the researcher believes that right-angle crashes have migrated.

Hence, the researcher studied the right-angle crashes at adjacent points to Dominik and Texas. These points include the signal north of Dominik at George Bush Drive and Texas Avenue and south of Dominik at the private drives and Miliff Drive (MP 6.190 to MP 6.255). It was found that there was an

increase of 33 percent in right-angle crashes in the George Bush and Texas intersection (see Table 8c). Furthermore, while right-angle crashes did decrease south of Dominik, the overall decrease was 4 percent less than on the entire corridor. The author believes that these two findings may be attributed to crash migration. However, the researcher looked at each record individually, and did not find any data that would allow him to ascribe the crashes to migration from Dominik. Consequently, the numbers appear to imply one thing, but the actual crash reports do not support what is implied. It is certain that the motorists still access Dominik, but how and whether there are associated crashes elsewhere can not be determined with any certainty.

Table 8c: Right-Angle

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	MP 6.190 to MP 6.255	Texas Corridor
Before	6	16	36	16	43
After	8	0	9	7	17
Percent Change	33	-100	-75	-56	-60
***p*-value**	0.70	1.00	1.00	0.97	1.00

Head-on crashes are commonly one of the most severe types of crashes and both raised medians and TWLTLs have helped reduce this type of crash. With only one crash in the before period and none in the after, there is little information to make statistical inferences (see Table 8d). The researcher indicates that TWLTL only decreases the chance of head-on collisions by giving a motorist a lane to swerve into when taking a defensive action in response to another unsafe driving condition such as another swerving car or debris on the roadway. TWLTLs have been affectionately dubbed "chicken lanes", because motorists run head-to-head. The turning gap provided by signalized intersections is larger than with access to private drives and local streets. As driveway densities increase, the problem compounds and becomes increasingly hazardous. On the other hand, the geometry of raised medians virtually removes the possibility of a head-on crash. The researcher concludes that raised medians significantly reduce the chance of head-on collisions based upon the data and the shear geometric differences between a raised median and a TWLTL. The author must note that head-on crashes were of particular concern for the intersection of Texas and Dominik, but after further research, it was discovered that that specific location's geometry was channelized. Hence, head-on crashes were not a concern at that particular location. This does not affect the earlier statement by the researcher that the raised median positively impacted the reduction of head-on crashes.

Table 8d: Head-On

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	0	0	0	1
After	0	0	0	0
Percent Change	0	0	0	-100
***p*-value**	N/A	N/A	N/A	0.84

With the exception of single vehicle crashes, each of the crash types encountered for the study corridor, as a whole, decreased. The 60 to 70 percent reductions suggest an improvement in safety.

The researcher investigated the increase in single vehicle crashes and the crashes in both the before-and-after period could not be attributed to the layout of the roadway. In each case, there were extenuating circumstances that influenced the crash that were not related to the median installation or to the overall design of the facility. The causal issues include: failure to control speed; weather-related problems; and lighting problems associated with nighttime.

Table 8e: Single Vehicle

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	3	0	0	3
After	2	0	0	4
Percent Change	-33	0	0	33
***p*-value**	0.67	N/A	N/A	0.65

All other crashes that do not fit into the previously mentioned crash types are listed in Table 8f. These values were not studied in detail, but there is a 100 percent reduction. Further study would be required to make any additional comments. For this report, no further investigation of these crashes will be conducted or discussed.

Table 8f: Other

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	1	0	1	3
After	0	0	0	0
Percent Change	-100	0	-100	-100
***p*-value**	0.84	N/A	0.84	0.96

Injuries showed both a decrease in quantity and in severity. In each case, the researcher considers both reductions a success of the retrofit as a whole. Total injuries dropped by 37 percent (see Table 9a). There were reductions of 33, 40 and 100 percent for possible, non-incapacitating and incapacitating injuries, respectively (see Tables 9b to 9d). There were no deaths in either the before or after periods. In all but the non-incapacitating injuries, the *p*-values were 0.96 or better. The researcher believes that the 100 percent reduction in incapacitating injuries and the overall 37 percent drop in injuries relate the most beneficial aspects of the results of the retrofit.

Table 9a: Injuries

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	60	7	16	81
After	36	0	10	51
Percent Change	-40	-100	-38	-37
***p*-value**	0.99	1.00	0.88	1.00

Table 9b: Possible Injuries

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	47	7	11	63
After	31	0	8	42
Percent Change	-34	-100	-27	-33
***p*-value**	0.96	1.00	0.75	0.98

Table 9c: Non-Incapacitating Injuries

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	12	0	3	15
After	5	0	2	9
Percent Change	-58	0	-33	-40
***p*-value**	0.96	N/A	0.67	0.89

Table 9d: Incapacitating Injuries

Time Period	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
Before	1	0	2	3
After	0	0	0	0
Percent Change	-100	0	-100	-100
***p*-value**	0.84	N/A	0.92	0.96

Outside of the conclusion that the decrease in injuries is good and is associated with the retrofit, there are underlying concerns that the researcher must state, but that can not be answered in this report.

During the oral review of the preliminary findings, some questions were asked related to the smaller reduction in injuries versus crashes. When the researcher tried to assess the concerns presented in the oral presentation, the issue was further complicated when he generated ratio comparisons between injuries and

other factors. The ratio of injuries to people involved (Table 6a) and injuries to vehicles (Table 6b) both increased, while the ratio of people to vehicles (Table 6c) decreased. This implies that a person is more likely to get injured in a crash after the retrofit, than before the retrofit even though the total number of crashes is lower. The researcher for this study feels the trend is related to the drop in persons per vehicle from 1.67 to 1.16. It was found, just prior to the report submittal, that the ARB no longer coded non-injury passengers in coded accidents after the 1995 criteria revision. Hence, these ratios cannot be guaranteed as accurate without further study. This finding could influence the ratio of injuries to people involved.

In an attempt to find some explanation, the researcher reanalyzed a portion of the data and discovered a 27 percent drop in driver injuries and a 40 percent drop in passenger injuries (see Table 9e). The *p*-values were 0.92 and 0.96, respectively. The researcher believes that the non-statistically significant drop in driver injuries versus the passenger injuries may be attributed to dangers associated specifically with drivers in crashes versus passengers. The most common danger revolves around the steering wheel. Drivers have been known to break wrists and arms on the steering wheel when they try to brace themselves. Malfunctioning airbags, cars unequipped with airbags and severe crashes may all allow for drivers to be brought forcefully up against the steering wheel.

Table 9e: Injuries by Occupant Type

Time Period	Driver	Passenger
Before	49	30
After	36	18
% Change	-27	-40
***p*-value**	0.92	0.96

The researcher believes that the results of the crash analysis represent a significant reduction in crashes and severity resulting in an overall improvement in safety along the corridor. There were various anomalies that the researcher has tried to address in this report, but further studies should be conducted for more conclusive explanations.

CONCLUSIONS

The results of this research project provide a lot of useful information with respect to both the research objectives and for future similar research studies. The findings in reference to the accuracy of the crash reporting process in its various stages were far better than anticipated. The crash analysis portion of the study revealed less conclusive results; however, the researcher still discovered beneficial insights.

Quality Control Analysis

The current crash reporting process in the State of Texas is a good system on all levels. The actual crash reports appear to be very detailed and accurate. The researcher believes that the entire record keeping process by the ARB is a good system that works to and succeeds at minimizing data error. The author will suggest future research in the Recommendation section.

Crash Analysis

After taking into account all of the anomalous data, the researcher believes that between the data and intuitive reasoning, the retrofit as a whole and the raised median specifically have improved safety along the study section. The reduction in the crashes, in general, and in the crash types individually, for the study corridor, supports the conclusion that facility improvements have significantly benefited safety. In addition, the researcher feels that the decrease in crash severity and a statistically significantly drop in the overall injuries and in the injuries to passengers further support the previous conclusion. Finally, the researcher reasons that, while the widening may have contributed to safety, the installation of a raised median has significantly improved the safety of the study corridor.

RECOMMENDATIONS

This research process has revealed many insights; however, there were areas of research that the author was unable to study based upon time constraints and further areas that were uncovered that require additional investigation. A bulleted list of these recommendations is as follows:

- A study should be conducted that further investigates the percentage of unreported crashes. This study should be expanded to include the entire roadway retrofit project, the construction period and a control section should be found and analyzed for comparison.
- A comprehensive study should be conducted in reference to crash analysis studies and the various anomalies that are found in the data as mentioned in the conclusion of this paper. In particular, there should be further investigation of the ratios generated in the Results in Tables 6a to 6c.
- The entire CRIS project and other automated crash reporting systems should be studied in detail and the research findings should be published and distributed to the people using/creating these programs and for those who use the information generated by these programs. In this study, the coding categories should be compared to help improve coding accuracy. Furthermore, the various methods for coding crash locations both in the field and at the state coding agency should be studied to generate recommendations to reduce the associated error. In particular, researchers should look at the use of new technology in crash reporting, such as the use of GPS in the field by police officers for coding crash locations (*9*).

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APPENDIX A: CRASH REPORTING PROCESS

Crash Reporting Process

Police Crash Report Documents

TEXAS PEACE OFFICER'S ACCIDENT REPORT ST-3 (Eff. 9/1/01) MAIL TO: ACCIDENT RECORDS, TEXAS DEPARTMENT OF PUBLIC SAFETY, PO BOX 4087, AUSTIN, TX 78773-0351

PLACE WHERE ACCIDENT OCCURRED
COUNTY ____ CITY OR TOWN ____ SHOW ONLY IF INSIDE CITY LIMITS
IF ACCIDENT WAS OUTSIDE CITY LIMITS, INDICATE DISTANCE FROM NEAREST TOWN ____ MILES NORTH S E W OF ____ CITY OR TOWN

LOC. ____

DO NOT WRITE IN THIS SPACE
LOC. ____
CODE ____
SEVERITY ____
FAT. REC. ____
DR. REC. ____

DPS USE

ROAD ON WHICH ACCIDENT OCCURRED ____ BLOCK NUMBER STREET OR ROAD NAME ROUTE NUMBER OR STREET CODE — CONSTR. ZONE ☐ YES ☐ NO SPEED LIMIT ____
INTERSECTING STREET OR RR X'ING NUMBER ____ BLOCK NUMBER STREET OR ROAD NAME ROUTE NUMBER OR STREET CODE — CONSTR. ZONE ☐ YES ☐ NO SPEED LIMIT ____
NOT AT INTERSECTION ____ ☐ FT. ☐ MI. ☐ N ☐ S ☐ E ☐ W OF ____ SHOW MILEPOST OR NEAREST INTERSECTING NUMBERED HIGHWAY. IF NONE, SHOW NEAREST INTERSECTING STREET OR REFERENCE POINT.

DATE OF ACCIDENT ____ 20____ DAY OF WEEK ____ HOUR ____ ☐ A.M. ☐ P.M. IF EXACTLY NOON OR MIDNIGHT, SO STATE

UNIT NO. 1 - MOTOR VEHICLE VEHICLE IDENT. NO. ____ IF BODY STYLE = VAN OR BUS, INDICATE SEATING CAPACITY ____
YEAR MODEL ____ COLOR & MAKE ____ MODEL NAME ____ BODY STYLE ____ LICENSE PLATE ____ YEAR STATE NUMBER
DRIVER'S NAME ____ LAST FIRST MIDDLE ADDRESS (STREET, CITY, STATE, ZIP) PHONE NUMBER ____
DRIVER'S LICENSE ____ STATE NUMBER CLASS/TYPE DOB ____ MO DAY YEAR RACE ____ SEX ____ OCCUPATION ____
SPECIMEN TAKEN (ALCOHOL/DRUG ANALYSIS) 1-BREATH 2-BLOOD 3-OTHER 4-NONE 5-REFUSED ☐ ALCOHOL/DRUG ANALYSIS RESULT ____ PEACE OFFICER, EMS DRIVER, FIRE FIGHTER ON EMERGENCY? ☐ YES ☐ NO
LESSEE ☐ OWNER ☐ ____ NAME (ALWAYS SHOW LESSEE IF LEASED, OTHERWISE SHOW OWNER) ADDRESS (STREET, CITY, STATE, ZIP)
LIABILITY INSURANCE ☐ YES ☐ NO ____ INSURANCE COMPANY NAME POLICY NUMBER VEHICLE DAMAGE RATING ____

UNIT NO. 2 - MOTOR VEHICLE ☐ TRAIN ☐ PEDALCYCLIST ☐ TOWED ☐ PEDESTRIAN ☐ OTHER ☐ VEHICLE IDENT. NO. ____ IF BODY STYLE = VAN OR BUS, INDICATE SEATING CAPACITY ____
YEAR MODEL ____ COLOR & MAKE ____ MODEL NAME ____ BODY STYLE ____ LICENSE PLATE ____ YEAR STATE NUMBER
DRIVER'S NAME ____ LAST FIRST MIDDLE ADDRESS (STREET, CITY, STATE, ZIP) PHONE NUMBER ____
DRIVER'S LICENSE ____ STATE NUMBER CLASS/TYPE DOB ____ MO DAY YEAR RACE ____ SEX ____ OCCUPATION ____
SPECIMEN TAKEN (ALCOHOL/DRUG ANALYSIS) 1-BREATH 2-BLOOD 3-OTHER 4-NONE 5-REFUSED ☐ ALCOHOL/DRUG ANALYSIS RESULT ____ PEACE OFFICER, EMS DRIVER, FIRE FIGHTER ON EMERGENCY? ☐ YES ☐ NO
LESSEE ☐ OWNER ☐ ____ NAME (ALWAYS SHOW LESSEE IF LEASED, OTHERWISE SHOW OWNER) ADDRESS (STREET, CITY, STATE, ZIP)
LIABILITY INSURANCE ☐ YES ☐ NO ____ INSURANCE COMPANY NAME POLICY NUMBER VEHICLE DAMAGE RATING ____

DAMAGE TO PROPERTY OTHER THAN VEHICLES
____ OBJECT NAME AND ADDRESS (STREET, CITY, STATE, ZIP) OF OWNER FEET FROM CURB $ ____ DAMAGE ESTIMATE

LIGHT CONDITION ☐	WEATHER ☐ ☐	SURFACE CONDITION ☐	TYPE ROAD SURFACE ☐	DESCRIBE ROAD CONDITIONS (INVESTIGATOR'S OPINION)
1-DAYLIGHT 2-DAWN 3-DARK-NOT LIGHTED 4-DARK-LIGHTED 5-DUSK	1-CLEAR/CLOUDY 2-RAINING 3-SNOWING 4-FOG 5-BLOWING DUST 6-SMOKE 7-SLEETING 8-HIGH WINDS 9-OTHER	1-DRY 2-WET 3-MUDDY 4-SNOWY/ICY 5-OTHER ____	1-BLACKTOP 2-CONCRETE 3-GRAVEL 4-SHELL 5-DIRT 6-OTHER ____	____

IN YOUR OPINION, DID THIS ACCIDENT RESULT IN AT LEAST $1,000.00 DAMAGE TO ANY ONE PERSON'S PROPERTY? ☐ YES ☐ NO

CHARGES FILED
NAME ____ CHARGE ____ CITATION NUMBER ____
NAME ____ CHARGE ____ CITATION NUMBER ____

TIME NOTIFIED OF ACCIDENT ____ DATE HOUR M HOW ____ TIME ARRIVED AT SCENE OF ACCIDENT ____ DATE HOUR M
TYPED OR PRINTED NAME OF INVESTIGATOR ____ DATE REPORT MADE ____ IS REPORT COMPLETE ☐ YES ☐ NO
SIGNATURE OF INVESTIGATOR ____ ID NO. ____ DEPARTMENT ____ DIST/AREA ____

Figure A1: Page 1 of a ST-3, Police Crash Report

SOLICITATION (SOL)	EJECTED	CODE FOR TYPE RESTRAINT USED	AIRBAG CODE	HELMET USE	CODE FOR INJURY SEVERITY	ALCOHOL/DRUG ANALYSIS (COMPLETE IF CASUALTIES NOT IN MOTOR VEHICLE)
INDICATES PERSON'S DESIRE TO RECEIVE CONTACT FROM PERSONS SEEKING PROFESSIONAL EMPLOYMENT AS/FOR AN ATTORNEY, CHIROPRACTOR, PHYSICIAN, SURGEON, PRIVATE INVESTIGATOR, OR ANY OTHER PERSON REGISTERED OR LICENSED BY A HEALTH CARE REGULATORY AGENCY. Y=O.K. TO SOLICIT N=NO SOLICITATION	A-NOT APPLICABLE Y-YES N-NO P-PARTIALLY U-UNKNOWN	A-SEATBELT & SHOULDER STRAP B-SEATBELT & NO SHOULDER STRAP C-CHILD RESTRAINT E-SHOULDER STRAP ONLY N-NONE	Y-DEPLOYED N-NO DEPLOYMENT U-UNKNOWN IF DEPLOYED	1-WORN-DAMAGED 2-WORN-NOT DAMAGED 3-WORN-UNK IF DAMAGED 4-NOT WORN 9-UNKNOWN IF WORN	K-KILLED A-INCAPACITATING INJURY B-NON INCAPACITATING C-POSSIBLE INJURY N-NOT INJURED	1-BREATH 2-BLOOD 3-OTHER 4-NONE 5-REFUSED

UNIT NO. 1 DAMAGE RATING _____ TOWED DUE TO DAMAGE ☐ YES ☐ NO VEHICLE REMOVED TO _____ BY _____

COMPLETE ALL DATA ON ALL OCCUPANTS' NAMES, POSITIONS, RESTRAINTS USED, ETC.; HOWEVER, IT IS NOT NECESSARY TO SHOW ADDRESSES UNLESS KILLED OR INJURED

ITEM NO.	OCCUPANT'S POSITION	NAME (LAST NAME FIRST)	ADDRESS (STREET, CITY, STATE, ZIP)	SOL	EJECTED	TYPE RESTRAINT USED	AIRBAG	HELMET	AGE	SEX	INJURY CODE
1	DRIVER	SEE FRONT									
2											
3											
4											
5											

UNIT NO. 2 (COMPLETE ONLY IF UNIT NO. 2 WAS A MOTOR VEHICLE) DAMAGE RATING _____ TOWED DUE TO DAMAGE ☐ YES ☐ NO VEHICLE REMOVED TO _____ BY _____

COMPLETE ALL DATA ON ALL OCCUPANTS' NAMES, POSITIONS, RESTRAINTS USED, ETC.; HOWEVER, IT IS NOT NECESSARY TO SHOW ADDRESSES UNLESS KILLED OR INJURED

ITEM NO.	OCCUPANT'S POSITION	NAME (LAST NAME FIRST)	ADDRESS (STREET, CITY, STATE, ZIP)	SOL	EJECTED	TYPE RESTRAINT USED	AIRBAG	HELMET	AGE	SEX	INJURY CODE
6	DRIVER	SEE FRONT									
7											
8											
9											
10											

COMPLETE IF CASUALTIES NOT IN MOTOR VEHICLE

PEDESTRIAN, PEDALCYCLIST ETC.	CASUALTY NAME (LAST NAME FIRST)	CASUALTY ADDRESS (STREET, CITY, STATE, ZIP)	SOL	TYPE SPECIMEN TAKEN	RESULT	HELMET	AGE	SEX	INJURY CODE

DISPOSITION OF KILLED AND/OR INJURED

ITEM NUMBERS	TAKEN TO	BY	IF AMBULANCE USED, SHOW: TIME NOTIFIED	TIME ARRIVED AT SCENE	NO. ATTENDANTS INCLUDING DRIVER

COMPLETE THIS SECTION IF PERSON KILLED

ITEM NUMBER	DATE OF DEATH	TIME OF DEATH	ITEM NUMBER	DATE OF DEATH	TIME OF DEATH	ITEM NUMBER	DATE OF DEATH	TIME OF DEATH

INVESTIGATOR'S NARRATIVE OPINION OF WHAT HAPPENED (ATTACH ADDITIONAL SHEETS IF NECESSARY)

DIAGRAM ☐ ONE WAY ☐ TWO WAY ☐ DIVIDED

INDICATE NORTH

FACTORS AND CONDITIONS LISTED ARE THE INVESTIGATOR'S OPINION

FACTORS/CONDITIONS CONTRIBUTING

UNIT 1	1	2	3
UNIT 2	1	2	3

OTHER FACTORS/CONDITIONS MAY OR MAY NOT HAVE CONTRIBUTED

UNIT 1	1	2
UNIT 2	1	2

TRAFFIC CONTROL

0-NO CONTROL OR INOPERATIVE
1-OFFICER OR FLAGMAN
2-STOP AND GO SIGNAL
3-STOP SIGN
4-FLASHING RED LIGHT
5-TURN MARKS
6-WARNING SIGN
7-RR GATES OR SIGNALS
8-YIELD SIGN
9-CENTER STRIPE OR DIVIDER
10-NO PASSING ZONE
11-OTHER CONTROL ☐

1. ANIMAL ON ROAD - DOMESTIC
2. ANIMAL ON ROAD - WILD
3. BACKED WITHOUT SAFETY
4. CHANGED LANE WHEN UNSAFE
5. DEFECTIVE OR NO HEADLAMPS
6. DEFECTIVE OR NO STOP LAMPS
7. DEFECTIVE OR NO TAIL LAMPS
8. DEFECTIVE OR NO TURN SIGNAL LAMPS
9. DEFECTIVE OR NO TRAILER BRAKES
10. DEFECTIVE OR NO VEHICLE BRAKES
11. DEFECTIVE STEERING MECHANISM
12. DEFECTIVE OR SLICK TIRES
13. DEFECTIVE TRAILER HITCH
14. DISABLED IN TRAFFIC LANE
15. DISREGARD STOP AND GO SIGNAL
16. DISREGARD STOP SIGN OR LIGHT
17. DISREGARD TURN MARKS AT INTERSECTION
18. DISREGARD WARNING SIGN AT CONSTRUCTION
19. DISTRACTION IN VEHICLE
20. DRIVER INATTENTION
21. DROVE WITHOUT HEADLIGHTS
22. FAILED TO CONTROL SPEED
23. FAILED TO DRIVE IN SINGLE LANE
24. FAILED TO GIVE HALF OF ROADWAY
25. FAILED TO HEED WARNING SIGN
26. FAILED TO PASS TO LEFT SAFELY
27. FAILED TO PASS TO RIGHT SAFELY
28. FAILED TO SIGNAL OR GAVE WRONG SIGNAL
29. FAILED TO STOP AT PROPER PLACE
30. FAILED TO STOP FOR SCHOOL BUS
31. FAILED TO STOP FOR TRAIN
32. FAILED TO YIELD ROW - EMERGENCY VEHICLE
33. FAILED TO YIELD ROW - OPEN INTERSECTION
34. FAILED TO YIELD ROW - PRIVATE DRIVE
35. FAILED TO YIELD ROW - STOP SIGN
36. FAILED TO YIELD ROW - TO PEDESTRIAN
37. FAILED TO YIELD ROW - TURNING LEFT
38. FAILED TO YIELD ROW - TURN ON RED
39. FAILED TO YIELD ROW - YIELD SIGN
40. FATIGUED OR ASLEEP
41. FAULTY EVASIVE ACTION
42. FIRE IN VEHICLE
43. FLEEING OR EVADING POLICE
44. FOLLOWED TOO CLOSELY
45. HAD BEEN DRINKING
46. HANDICAPPED DRIVER (EXPLAIN IN NARRATIVE)
47. ILL (EXPLAIN IN NARRATIVE)
48. IMPAIRED VISIBILITY (EXPLAIN IN NARRATIVE)
49. IMPROPER START FROM PARKED POSITION
50. LOAD NOT SECURED
51. OPENED DOOR INTO TRAFFIC LANE
52. OVERSIZE VEHICLE OR LOAD
53. OVERTAKE AND PASS INSUFFICIENT CLEARANCE
54. PARKED AND FAILED TO SET BRAKES
55. PARKED IN TRAFFIC LANE
56. PARKED WITHOUT LIGHTS
57. PASSED IN NO PASSING ZONE
58. PASSED ON RIGHT SHOULDER
59. PEDESTRIAN FAILED TO YIELD ROW TO VEHICLE
60. SPEEDING - UNSAFE (UNDER LIMIT)
61. SPEEDING - OVER LIMIT
62. TAKING MEDICATION (EXPLAIN IN NARRATIVE)
63. TURNED IMPROPERLY - CUT CORNER ON LEFT
64. TURNED IMPROPERLY - WIDE RIGHT
65. TURNED IMPROPERLY - WRONG LANE
66. TURNED WHEN UNSAFE
67. UNDER INFLUENCE - ALCOHOL
68. UNDER INFLUENCE - DRUG
69. WRONG SIDE - APPROACH OR IN INTERSECTION
70. WRONG SIDE - NOT PASSING
71. WRONG WAY - ONE WAY ROAD
72. DRIVER INATTENTION - (CELL/MOBILE PHONE USE)
73. ROAD RAGE
74. OTHER FACTOR (WRITE ON LINE BELOW)

Figure A2: Page 2 of a ST-3, Police Crash Report

DPS Crash Report Documents

DRIVER'S CONFIDENTIAL ACCIDENT REPORT
FORM ST-2 (Rev. 1/1/90)

PLEASE READ INSTRUCTIONS ON REVERSE SIDE

TEXAS DEPARTMENT OF PUBLIC SAFETY

LOCATION

PLACE WHERE ACCIDENT OCCURRED COUNTY _____ CITY OR TOWN _____

IF ACCIDENT WAS OUTSIDE CITY LIMITS, INDICATE DISTANCE FROM NEAREST TOWN _____ MILES ☐ NORTH ☐ S ☐ E ☐ W OF _____ CITY OR TOWN

ROAD ON WHICH ACCIDENT OCCURRED _____ BLOCK NUMBER _____ STREET OR ROAD NAME _____ ROUTE NUMBER CONSTR. ZONE ☐ YES ☐ NO SPEED LIMIT ____

COMPLETE ONE
- INTERSECTING STREET _____ BLOCK NUMBER _____ STREET OR ROAD NAME _____ ROUTE NUMBER CONSTR. ZONE ☐ YES ☐ NO SPEED LIMIT ____
- NOT AT INTERSECTION _____ FEET ☐ NORTH ☐ S ☐ E ☐ W OF _____ SHOW MILEPOST OR NEAREST INTERSECTING NUMBERED HIGHWAY. IF URBAN, SHOW NEAREST INTERSECTING STREET OR REFERENCE POINT.

DO NOT WRITE IN THIS SPACE

DPS NO. _____
LOC _____
CODE _____
SEVERITY _____
TYPE _____

TIME

DATE OF ACCIDENT _____ 19 ____ DAY OF WEEK _____ HOUR _____ ☐ A.M. ☐ P.M. IF EXACTLY NOON OR MIDNIGHT, SO STATE

VEHICLES

YOUR VEHICLE
YEAR MODEL _____ MAKE _____ (FORD, CHEV., ETC.) TYPE OF VEHICLE _____ (SEDAN, PICKUP, TRUCK, ETC.) LICENSE PLATE _____ YEAR STATE NUMBER

OWNER _____ NAME MAIL ADDRESS CITY AND STATE ZIP

DRIVER _____ NAME MAIL ADDRESS CITY AND STATE ZIP

DRIVER'S LICENSE _____ STATE NUMBER DATE OF BIRTH _____ SEX _____ RACE _____

APPROX. COST TO REPAIR YOUR VEHICLE $ _____

FOR OTHER VEHICLES USE ANOTHER FORM

TOTAL VEHICLES INVOLVED

OTHER UNIT - MOTOR VEHICLE, TRAIN, PEDESTRIAN, BICYCLIST, ETC. - INDICATE WHICH _____
(COMPLETE INFORMATION YOU HAVE AVAILABLE - IF UNKNOWN, MARK "NOT KNOWN")

YEAR MODEL _____ MAKE _____ (FORD, CHEV., ETC.) TYPE OF VEHICLE _____ (SEDAN, PICKUP, TRUCK, ETC.) LICENSE PLATE _____ YEAR STATE NUMBER

OWNER _____ NAME MAIL ADDRESS CITY AND STATE ZIP

DRIVER _____ NAME MAIL ADDRESS CITY AND STATE ZIP

APPROX. COST TO REPAIR THIS VEHICLE $ _____

DAMAGE TO PROPERTY OTHER THAN VEHICLES _____ NAME OBJECT, SHOW OWNERSHIP, AND STATE NATURE OF DAMAGE

APPROXIMATE COST TO REPAIR $ _____

CASUALTIES

NO. 1 (SHOW ONLY PEDESTRIANS OR PERSONS INJURED IN YOUR VEHICLE)

NAME _____ ADDRESS _____

AGE _____ SEX _____ RACE _____ WAS PERSON KILLED? _____ DATE OF DEATH _____

DESCRIBE INJURY _____

☐ DRIVER ☐ PASSENGER ☐ PEDESTRIAN ☐ RIGHT FRONT ☐ RIGHT REAR ☐ CENTER REAR ☐ LEFT REAR

SEAT BELT ☐ USED ☐ NOT USED

NO. 2

NAME _____ ADDRESS _____

AGE _____ SEX _____ RACE _____ WAS PERSON KILLED? _____ DATE OF DEATH _____

DESCRIBE INJURY _____

☐ DRIVER ☐ PASSENGER ☐ PEDESTRIAN ☐ RIGHT FRONT ☐ RIGHT REAR ☐ CENTER REAR ☐ LEFT REAR

SEAT BELT ☐ USED ☐ NOT USED

STATE BRIEFLY WHAT HAPPENED (IF SPACE IS INSUFFICIENT CONTINUE ON ANOTHER PAGE)

★ DRIVER'S SIGNATURE _____ DATE OF REPORT _____

IMPORTANT! COMPLETE REQUIRED INSURANCE INFORMATION ON OTHER SIDE

Figure A3: Page 1 of a ST-2, DPS "Blue Form"

PLEASE READ ALL INSTRUCTIONS CAREFULLY

THIS FORM CONTAINS TWO SEPARATE REPORTS WHICH WILL BE DESTROYED AFTER COMPLETION OF ALL PROCESSING

The driver of a motor vehicle involved in an accident **not investigated by a law enforcement officer** and resulting in injury to or death of any person, or damage to the property of any one person, including himself, to an apparent extent of at least Five Hundred Dollars ($500), shall within ten (10) days after such accident complete and forward these reports in accordance with the instructions below. These reports are not required when an accident is investigated by a law enforcement officer unless specifically requested by authority of Section 4, Texas Motor Vehicle Safety-Responsibility Act (Article 6701h, Vernon's Texas Civil Statutes).

INSTRUCTIONS FOR COMPLETING DRIVER'S CONFIDENTIAL ACCIDENT REPORT (FORM ST-2)
(On other side of this form)

NOTE: The Driver's Confidential Accident Report (Form ST-2) is classified by law as privileged and for confidential use in accident prevention purposes.

1. The report on the other side of this sheet should be prepared and signed by the driver; however, if the driver is unable to make the report for some valid reason, the report may be submitted by another person with a notation as to the reason the driver could not report.
2. Print all names and addresses. Include sufficient information for "Location" and "Time" so that exact date and place of accident may be determined. Answer all questions to the best of your knowledge. If unable to answer any question, mark "not known."
3. If the "other unit" is a pedestrian, bicycle, train or other non-motor vehicle, please specify and show the name of pedestrian, bicyclist, etc. on line labeled "Driver."
4. If accident involved a fixed object, describe it fully, show its exact location and state whether it was protected by flags, painting and/or lights.
5. The narrative description of the accident should contain a brief statement of the facts regarding the accident. If additional space is needed, use a full size sheet of paper for continuation.
6. An accurate original signed report will avoid the necessity for a supplemental report.

TEXAS MOTOR VEHICLE ACCIDENT INSURANCE INFORMATION (FORM SR-21) Rev. 4-88

IMPORTANT

Note: Under certain conditions, Section 5 of the Texas Motor Vehicle Safety-Responsibility Act (V.T.C.S. 6701h) requires suspension of driver's license, registration receipts and license plates of uninsured motorists involved in motor vehicle accidents resulting in bodily injury or death, or damages to the property of any one person of at least $1,000.00. The Accident Insurance Information (Form SR-21) is a public document.

1. This report may be prepared and signed by either the driver or owner of the involved vehicle.
2. Accurate, complete reporting of at least minimum liability insurance coverage will avoid additional correspondence and prevent possible suspension of your driving and registration privileges.
3. If garage estimates are attached to non-injury accidents, processing will be expedited.

DID YOU HAVE AT LEAST $20,000/40,000 BODILY INJURY AND $15,000 PROPERTY DAMAGE LIABILITY INSURANCE IN EFFECT ON THE DATE OF THE ACCIDENT? ☐ YES ☐ NO

If the above is answered "Yes" answer all the items in the box below.

Date of Accident _____ Place of Accident _____
City or Town County

Make of Vehicle
Involved in Accident _____ Year _____ Type _____ Vehicle Identification No. _____

_____ Owner's Name _____
Name of Your Liability Insurance Co. (Not the Agent)

Owner's Address _____

Policy No. _____ Driver's Name _____

☐ Owner
_____ ☐ Driver Driver's Address _____
Usual Signature

If your vehicle was operating under Texas Railroad Commission Carrier Authority, give No. _____

When completed, mail this form to: STATISTICAL SERVICES BUREAU
TEXAS DEPARTMENT OF PUBLIC SAFETY
BOX 4087. AUSTIN. TEXAS 78773-0001

Figure A4: Page 2 of a ST-2, DPS "Blue Form"

Outline of the Crash Reporting Process

1. The police file a report (a blank version of the most recent format of a police crash report, ST-3, for the State of Texas is in Figures A1 and A2 above).
2. Local Records
    a. Hardcopies are kept on file for approximately 2-5 years [3 years for the College Station Police Department (CSPD)].
    b. Depending on the size of the police department and the internal desires of the department to computerize their crash reporting system, some departments will code some of the information from the crash reports into their own internal database.
3. The report is shipped within approximately 10 days to the ARB of the DPS. The sending of the records may vary based upon the severity of the crash, the investigation required, any coding and/or logging filed within the local police department and any back-logs in the overall process at that department.
4. The ARB receives the crash reports directly from the police department through the federal mail system.
    a. In 1997, the DPS began to improve the antiquated crash report filing process. Currently, DPS and TxDOT are combining their efforts to create and fund a new, more automated crash reporting system: the Crash Records Information System or CRIS. Ms. Cathy Cioffi is the project manager (Figure A6 and A7, are a copy of a CRIS Newsletter).
5. The records are processed in an assembly line fashion with specific people focusing on particular sections
    a. The initial decision is made about whether or not to code a crash to a particular person's driving history (i.e., rear-ending = yes, hitting a tree while swerving from an animal = no).
    b. The files are then sorted for further coding/processing.
        i. Before July 1995, all crashes were coded. Not all crashes are reported. Hence, only reported crashes may be coded and this limitation should be expressed and understood in any study.
        ii. As of July 1995, non-injury crashes that do not result in property damage of greater than $1,000 [i.e., tow-away crashes will exceed this and are the usual criteria for coding property damage only crashes (PDO)] are no longer coded. Also, only injured passengers are coded. Before, all passengers were coded.
        iii. Both coded and non-coded records are stored on microfilm at the same time. They are transferred to microfilm after the coding process is complete and the records are uploaded into the DPS mainframe.
    c. Files to be coded will be further classified and numbered.
        i. The coding process is completed using in-house written documents.
        ii. These documents will be sent to another department for input into the DPS mainframe database. CD-ROM's may be made for a particular county for use outside of the DPS.
            1. These CD's are made upon request. The CD's contain the data in a data stream format. This format is impossible to read without the appropriate codebook. Furthermore, it is still difficult to read

without the proper formatting software. Texas Transportation Institute (TTI) uses a statistical analysis software package, called SAS. This program converts the data stream into a user-friendlier format that may be imported into spreadsheet software such as Microsoft Excel.

2. The format contains column headers and virtually all of the data is in numeric coding that is fairly easy to understand by anyone who has a copy of the coding sheets.

iii. The applicable information is also coded to the driving records of the motorists involved in a crash.

d. The coding process is filled with checks and editing.
   i. A double input method is used, whereby two individuals enter the same information and a computer compares the records to find possible errors.
   ii. The computer will only allow certain ranges of information to be entered in certain fields to reduce errors. For example, some entries may only allow text while some may only allow numbers and other entries may only allow one number while others allow up to 3 digits.

e. Record A contains the summarized crash report information including the location, number of vehicles involved, type of accident, orientation and other information.

f. Record B contains the driver's and the vehicle's descriptive information.
   i. This includes whether the drivers were injured, drunk, and/or considered at-fault.
   ii. The vehicle description comprises of vehicle make and model information and whether a vehicle defect could be attributed to the crash.

g. Record C contains only the information for the passengers in the vehicles involved and any pedestrians, cyclists or additional people involved (non-injured passengers are not coded, but the total people inside each vehicle is listed).

h. All of the records are kept in the mainframe database and hard copies of the reports are kept on file and organized by county and date in the ARB.
   i. Paper hard copies are transferred to microfilm hardcopies and held for 10 years. The records are destroyed after ten years.
   ii. The DPS has an internal seven digit coding system for referencing within the data in the mainframe or on CD.
      1. The DPS coding is reused at the beginning of every new 10-year period.
      2. The seven-digit code is not used in pulling actual records from the stored microfilm filing system.
      3. The seven-digit code is also coded with the driver's individual traffic record for referencing purposes.

i. Comments:
   i. The whole process takes approximately 18 months.
   ii. The actual milepost locations are accessed by the use of the Roadway Inventory (RI) logbook sheets generated by TxDOT. The logbook shows the mileposts of cross streets and important curb cuts (e.g., a fire station) along a particular roadway.

iii. The mainframe information is updated when the coding process is complete. In particular, the ARB uses a 13th month system to assess any editing issues discovered through the data entry process and to address any additional unforeseen delays.
iv. There is another form, known as the "blue form", that may be submitted directly to the DPS by individuals who were in a crash that was not reported by local law enforcement. A copy of the blue form is above in Figure A3. Depending on the crash location, severity and whether there were any violations involved (i.e., hit-and-run violation), the local police department may or may not record them information in their own database. State Law puts the responsibility on the drivers involved to report the crash and not the police department.
v. The information in the database has been used in the past to better plan police officer route scheduling to ensure timely response to crash prone areas.

CSPD Mainframe Data File Format

College Station Police Department
Texas Accidents

ID	K1	C2	C3	C4	C5	C6	C7	C8
1	01/07/1998	1300	900	TEXAS	WALTON DR	North	Clear	Possible Injury
2	01/14/1998	1540	700	TEXAS	WALTON DR	North	Clear	Non-Injury
3	01/17/1998	0834	0	TEXAS	BRENTWOOD DR	*missing*	Clear	Non-Incapacitating
4	01/18/1998	1742	0	TEXAS	VALLEY VIEW DR-CS	*missing*	Clear	Non-Injury
5	01/19/1998	1826	1500	TEXAS	MILLIFF RD	South	Clear	Non-Injury
6	01/22/1998	2238	800	TEXAS	UNIVERSITY DR	South	Clear	Non-Injury
7	01/23/1998	2120	0	TEXAS	RICHARDS ST	*missing*	Clear	Non-Injury
8	01/23/1998	2310	700	TEXAS	LIVE OAK ST-CS	South	Clear	Non-Injury
9	01/23/1998	1714	700	TEXAS	LIVE OAK ST-CS	South	Clear	Possible Injury
10	01/28/1998	1655	0	TEXAS	GEORGE BUSH DR	*missing*	Clear	Non-Injury
11	01/29/1998	1620	0	TEXAS	HARVEY MITCHELL PW S	*missing*	Clear	Possible Injury
12	01/30/1998	1226	900	TEXAS	LINCOLN AV	South	Clear	Non-Injury
13	01/30/1998	1015	1300	TEXAS	MOSS ST-CS	North	Clear	Non-Injury
14	02/05/1998	0641	1000	TEXAS	*missing*	*missing*	Clear	Non-Injury
15	02/06/1998	1735	1000	TEXAS	WALTON DR	South	Clear	Non-Injury
16	02/08/1998	1438	1300	TEXAS	GILCHRIST AV	*missing*	Clear	Possible Injury
17	02/10/1998	1410	1050	TEXAS	WALTON DR	*missing*	Clear	Non-Incapacitating
18	02/16/1998	0958	1500	TEXAS	MILLIFF RD	*missing*	Raining	Non-Injury
19	02/18/1998	1440	1000	TEXAS	WALTON DR	South	Raining	Non-Incapacitating
20	02/19/1998	2220	500	TEXAS	UNIVERSITY DR	*missing*	Clear	Non-Injury
21	02/19/1998	2100	500	TEXAS	UNIVERSITY DR	*missing*	Clear	Non-Injury
22	02/23/1998	0856	700	TEXAS	UNIVERSITY DR	South	Clear	Possible Injury
23	02/25/1998	1335	700	TEXAS	LONE STAR DR	South	Clear	Non-Injury
24	02/27/1998	1615	800	TEXAS	LIVE OAK ST-CS	South	Clear	Non-Injury
25	02/28/1998	1430	1080	TEXAS	WALTON DR	*missing*	Clear	Non-Injury
26	02/28/1998	2202	700	TEXAS	LIVE OAK ST-CS	South	Clear	Non-Injury
27	03/03/1998	0752	1400	TEXAS	GEORGE BUSH DR	North	Clear	Non-Injury
28	03/05/1998	2130	800	TEXAS	LIVE OAK ST-CS	*missing*	Clear	Non-Injury
29	03/06/1998	2257	500	TEXAS	UNIVERSITY DR	*missing*	Clear	Incapacitating
30	03/06/1998	1509	0	TEXAS	REDMOND DR	*missing*	Clear	Non-Injury
31	03/07/1998	1629	600	TEXAS	LIVE OAK ST-CS	South	Clear	Possible Injury
32	03/07/1998	2140	1400	TEXAS	GEORGE BUSH DR	South	Clear	Non-Injury
33	03/09/1998	1411	500	TEXAS	UNIVERSITY DR	South	Clear	Non-Incapacitating
34	03/12/1998	2120	700	TEXAS	*missing*	*missing*	Raining	Non-Injury
35	03/12/1998	2158	700	TEXAS	UNIVERSITY DR	South	Raining	Non-Injury
36	03/13/1998	1310	1500	TEXAS	HARVEY	North	Clear	Possible Injury
37	03/14/1998	1320	1000	TEXAS	WALTON DR	South	Raining	Non-Injury
38	03/16/1998	1229	1500	TEXAS	REDMOND DR	South	Clear	Possible Injury
39	03/18/1998	1239	1000	TEXAS	WALTON DR	South	Clear	Possible Injury
40	03/21/1998	0112	1000	TEXAS	WALTON DR	*missing*	Clear	Possible Injury
41	03/24/1998	1135	500	TEXAS	UNIVERSITY DR	*missing*	Clear	Non-Injury
42	03/25/1998	2303	1400	TEXAS	GEORGE BUSH DR	*missing*	Clear	Possible Injury
43	03/25/1998	1705	1500	TEXAS	HARVEY	North	Clear	Non-Injury

Date Printed May 20, 2002 10:09:04 AM

Page 1 of 1

Figure A5: CSPD Crash Data File

CRIS Newsletter

CRIS Project Newsletter

Volume 1, Issue 1

What is CRIS?

Crash Records Information System

The crash records information system (CRIS) project is a joint initiative between the Department of Public Safety (DPS) and the Texas Department of Transportation (TxDOT). The vision of the project is to implement a new crash records information system that will provide enhanced efficiencies to capture, manage and disseminate timely and accurate data to parties who need it to improve the safety of the Texas roadways.

Accident data is used to evaluate the effectiveness of safety programs and obtain funding to support traffic safety. This data is also critical for state and local transportation project planning and prioritization, highway and railroad crossing safety evaluation, identification of target areas for enhanced law enforcement, and for traffic safety studies.

The system in use today was designed in the 1970's using technologies available at that time that do not meet the needs of DPS, TxDOT or other local and state agencies in 2002. The result is a system that is manually intensive and untimely in it's reporting capabilities.

The CRIS Project includes the redesign of the current accident /crash records system resulting in the creation of a new crash records information system. This may include designing linkages to other components of the traffic records system and other systems.

The CRIS project is utilizing the DPS Concurrent Engineering Methodology (CEM) to manage and document the project. A steering committee comprised of DPS and TxDOT stakeholders provide guidance. Sponsors for the project are Frank Elder, Assistant Chief-Driver License Division, Carol Rawson, Deputy Director-Traffic Operations and Bob Burroughs, Major-Traffic Law Enforcement.

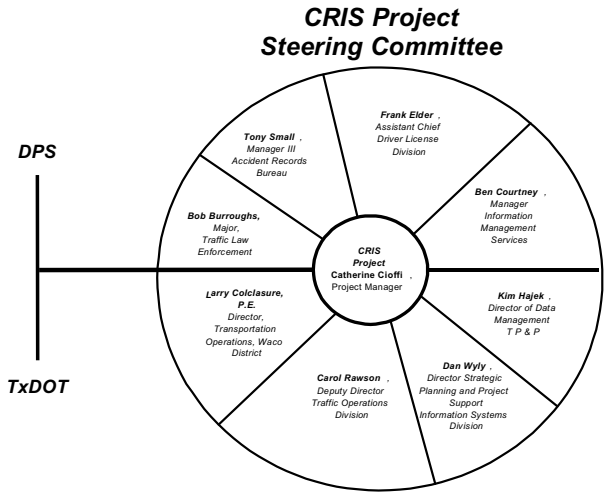
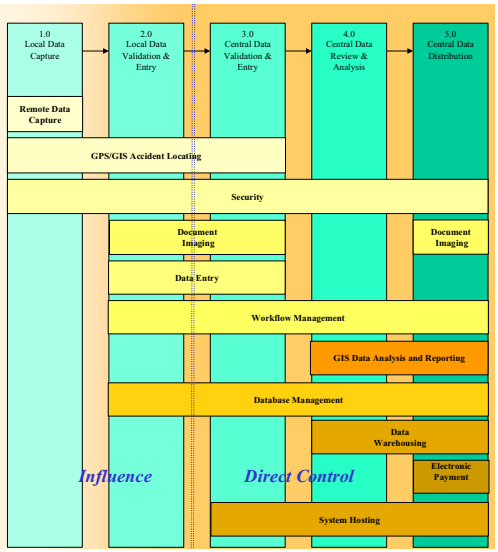


Figure A6: Page 1 of a CRIS Newsletter

Currently, we have a vendor conducting a Study & Recommend assisting the CRIS Steering Committee in making a solution approach decision. The vendor will deliver a Findings and Recommendation Report in August that will include a recommended solution approach and a cost benefit analysis.

The project will take a phased approach to implementation. A Request for Offer (RFO) for the next phase of the project is expected to go out by the end of the 4th quarter 2002. The project is expected to move into the design and implementation phase by the end of the 1st quarter 2003.

The five CRIS process areas and potential technologies are graphically depicted below:



Special thanks to Rick Pearson and Tony Zamarripa of the Accident Records Bureau for their imagination and creativity in the design of the CRIS project logo.

If you are interested in receiving quarterly CRIS updates or have questions about the project, please contact:

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Figure A7 Page 2of a CRIS Newsletter

APPENDIX B: CRASH ANALYSIS DATA

Table B1: Crashes

Time Period	Total Crashes Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	38	8	24	69
1994	56	8	20	82
1998*	13	0	3	19
1999*	18	0	8	28
Before	94	16	44	151
After	31	0	11	47

- Indicates that the timeframe goes from July of that year through June of the next.

Table B2: Vehicles

Time Period	Total Vehicles Involved Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	85	16	49	149
1994	136	16	40	191
1998*	31	0	6	44
1999*	44	0	18	64
Before	221	32	89	340
After	75	0	24	108

- Indicates that the timeframe goes from July of that year through June of the next.

Table B3: People

Time Period	Total People Involved Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	130	18	67	217
1994	276	27	60	352
1998*	34	0	6	47
1999*	54	0	22	78
Before	406	45	127	569
After	88	0	28	125

* Indicates that the timeframe goes from July of that year through June of the next.

Table B4: Injuries

Time Period	Total Injuries Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	25	2	8	36
1994	35	5	8	45
1998*	13	0	3	19
1999*	23	0	7	32
Before	60	7	16	81
After	36	0	10	51

- Indicates that the timeframe goes from July of that year through June of the next.

Table B5: Fatalities

Time Period	Total Fatalities Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	0	0	0	0
1994	0	0	0	0
1998*	0	0	0	0
1999*	0	0	0	0
Before	0	0	0	0
After	0	0	0	0

- Indicates that the timeframe goes from July of that year through June of the next.

Table B6: Non-Injuries

Time Period	Total Non-Injuries Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	105	16	59	181
1994	241	22	52	307
1998*	21	0	3	28
1999*	31	0	15	46
Before	346	38	111	488
After	52	0	18	74

* Indicates that the timeframe goes from July of that year through June of the next.

Table B7: Rear-Ending

Time Period	Total Rear-Ending Crashes Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	34	0	3	41
1994	48	0	1	52
1998*	8	0	0	10
1999*	13	0	1	14
Before	82	0	4	93
After	21	0	1	24

* Indicates that the timeframe goes from July of that year through June of the next.

Table B8: Sideswipe

Time Period	Total Sideswipe Crashes Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	1	0	3	6
1994	1	0	0	2
1998*	0	0	1	2
1999*	0	0	0	0
Before	2	0	3	8
After	0	0	1	2

* Indicates that the timeframe goes from July of that year through June of the next.

Table B9: Right-Angle

Time Period	Total Right-Angle Crashes Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	1	8	18	20
1994	5	8	18	23
1998*	4	0	2	6
1999*	4	0	7	11
Before	6	16	36	43
After	8	0	9	17

* Indicates that the timeframe goes from July of that year through June of the next.

Table B9: Head-On

Time Period	Total Head-On Crashes Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	0	0	0	0
1994	0	0	0	1
1998*	0	0	0	0
1999*	0	0	0	0
Before	0	0	0	1
After	0	0	0	0

* Indicates that the timeframe goes from July of that year through June of the next.

Table B10: Single Vehicle

Time Period	Total Single Vehicle Crashes Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	1	0	0	1
1994	2	0	0	2
1998*	1	0	0	1
1999*	1	0	0	3
Before	3	0	0	3
After	2	0	0	4

* Indicates that the timeframe goes from July of that year through June of the next.

Table B11: Other

Time Period	Total Single Vehicle Crashes Attributed to:			
	Texas & George Bush	Texas & Dominik	MP 6.100 to MP 6.255	Texas Corridor
1993	1	0	0	1
1994	2	0	0	2
1998*	1	0	0	1
1999*	1	0	0	3
Before	3	0	0	3
After	2	0	0	4

* Indicates that the timeframe goes from July of that year through June of the next.

JEFFREY DAVID MILES

Jeffrey David Miles is currently pursuing his Bachelor of Science degree in Civil Engineering at Texas A&M University. He will complete his B.S. in December, 2002, and he has already been accepted by the Civil Engineering Graduate School Program at Texas A&M University. While attending graduate school, he will be a Graduate Research Assistant at the Texas Transportation Institute (TTI).

Mr. Miles has had a diverse academic and professional career. He graduated Advanced with Honors from Richardson High School in 1994, then joined the United States Armed Forces and served 4 years in the U.S. Navy. Three and a half years of his service career were spent aboard the U.S.S. Alaska (SSBN 732B). After completing thirty-seven academic hours at Richland College in Dallas, he left to seek his B.S. in Civil Engineering at Texas A&M University. He has become an active member of the American Society of Civil Engineers (ASCE), the Institute of Transportation Engineers (ITE) and the International Golden Key Honor Society. In addition, he is the newly elected Vice-President for the Texas A&M ITE Student Chapter. He has worked at TTI since February, 2001.

It is his desire to study transportation engineering and to direct his energies towards research in the areas of intelligent transportation systems (ITS), geometric design, access management and signage during his graduate studies and throughout his remaining career.

MICRO-SIMULATION ANALYSIS OF INTERSECTIONS NEAR HIGHWAY-RAILROAD GRADE CROSSINGS

by

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SUMMARY

The purpose of this project was to perform micro-simulation analyses on intersections near Highway-Railroad Grade Crossings to determine if controlling mean train speed and train speed variability would improve safety and reduce delays. The project included two objectives. First, the micro-simulation model of the Wellborn Corridor was completed and checked for errors and accuracy. The second part consisted of train speed sensitivity analyses on mean train speed and train speed variability.

A micro-simulation model of the Wellborn Corridor was created using VISSIM. The simulation was run ten times in each of the nine train speed distributions. Average delay was collected for each of the four intersections. Additionally, the simulation was run with alternate train detection distances and select train speed distributions, and average delay was again collected.

For each train speed distribution and intersection, delays were compared using the t-test with a 95% confidence interval. Comparisons were made against train speed distributions with either the same mean speeds, the same standard deviations, the base train speed, or the same distribution with a different train detection distance. Furthermore, these comparisons were made for each of the four 10-minute intervals of the simulation.

Significant differences were found only in the second time interval, and this was expected because this was the interval that included the trains. Some significant differences were found when the mean train speeds were altered, and these were more prevalent at the George Bush intersection, where the traffic volumes are the highest. However, the number of statistically different comparisons was still not considered substantial.

Ultimately, it was found that manipulating the train detection distance, the mean train speed, or the train speed standard deviation did not have a significant effect on the average delay for the traffic layout that was modeled for this particular corridor.

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INTRODUCTION

Intersections near Highway-Railroad Grade Crossings require special operating procedures in order to ensure that vehicles and pedestrians are not put at risk when trains are present. This is accomplished by traffic signal preemption. Traffic signal preemption basically entails four steps (1). First, the train is detected and the active railroad warning devices, consisting of gates and flashing lights, are initiated. Next, the right-of-way must be transferred from the current phase to the phase that controls the critical approach (e.g. where vehicles could potentially queue across the railroad tracks). This is defined as the "right-of-way transfer time." Third, this phase must have an adequate green clearance phase such that the tracks may be cleared. This is the "queue clearance time." Finally, there must be a "separation time" which is the amount of time the tracks are clear before the train arrives at the intersection. The main objective of signal preemption is safety. By law, the warning devices must provide a minimum of 20 seconds of warning time (2). Consequently, the sum of the three components listed above ("right-of-way transfer time," "queue clearance time," "separation time") must be less than or equal to 20 seconds during preemption.

Specifically, the current preemption strategies are designed to clear the tracks of all vehicle traffic before the train arrives at the intersection. Although safety is the main concern of preemption, the secondary objective of preemption is to minimize delay for the vehicles and pedestrians at the intersection. The current preemption techniques are designed for the fastest train and the train detector is placed based on the minimum 20 seconds of active warning time. This means that the detector is placed at the distance that the fastest train can travel in 20 seconds. Because the vast majority of trains travel at speeds slower than the fastest train, they will have warning times that are greater than the minimum 20 seconds. This extra warning time can have an adverse effect on the delay of the vehicles and pedestrians at the intersection.

Although current preemption techniques perform well in clearing the tracks of vehicular traffic, some inefficiencies still exist. Particularly, delays could be lowered for the vehicles at the intersections. While there have been a number of approaches for remedying this problem, they have all put the onus on traffic operations and management. In this project, the focus was on controlling the trains directly.

Scope

This project examined controlling certain train characteristics as a possible solution to the delay problem. Specifically, the project looked at the effect of controlling the mean train speed and the variance of train speed on vehicle delay. It should be noted that while the technology exists to control train speed directly, there is no law to allow this. However, this should not preclude examining this option. In fact, one could envision that the law might be changed if this option increases safety, reduces delay, and is cheaper than other alternatives. Ultimately, the hypothesis is that if the mean speed and speed variance of trains can be controlled, the system will be safer and more efficient.

In order to answer the hypothesis, the VISSIM 3.60 micro-simulation package was used to model the corridor. VISSIM was chosen because it can effectively simulate multi-modal systems, which, in this case, includes rail, vehicle, and pedestrian traffic.

The test bed for this project is the "Wellborn Corridor" in College Station, Texas. This corridor includes an urban arterial roadway, Wellborn Rd., which runs parallel to a single Union Pacific two-way railroad line. Both the road and tracks run through the campus of Texas A&M University, and there are a relatively large number of pedestrians and vehicles that cross the rail line at various highway-railroad at-grade crossings. In addition, there are approximately 15-20 trains per day using this corridor (3), and the number of traffic preemption events is relatively high. The section of interest will include four signalized intersections, each near a Highway-Railroad Grade Crossing, and they are (from north to south): Old Main Dr., Joe Routt Blvd., George Bush Dr., and Holleman Dr. See Figure 1 for a map of the corridor.

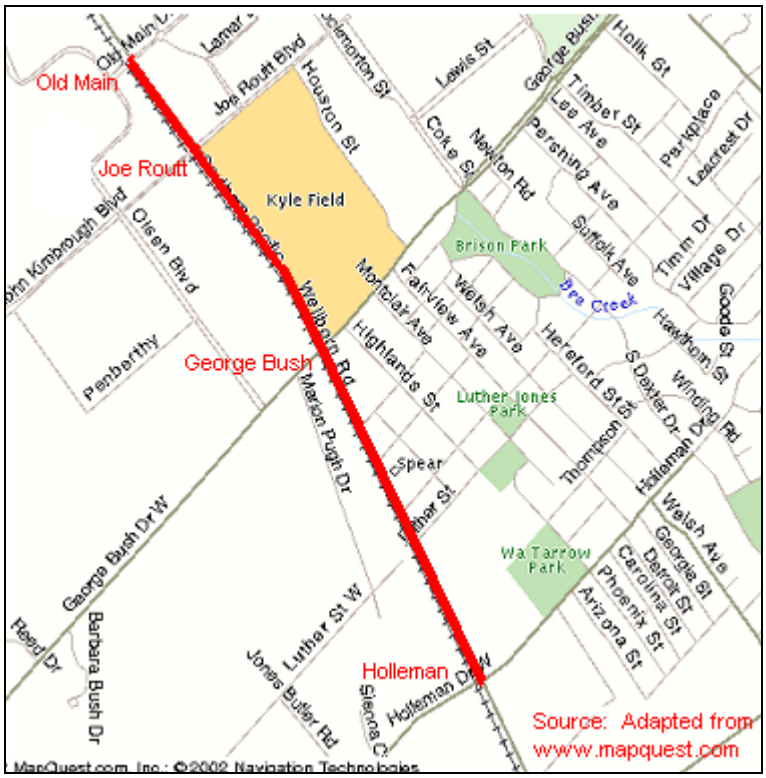


Figure 1. Map of Wellborn Corridor

All work on this project was done using a micro-simulation model. Although some traffic data and other input values were collected, no other field data was collected. None of the results or strategies developed from this project were tested in the field.

Objectives

There were two objectives for this project. The first objective was to create a micro-simulation model of the Wellborn Rd. corridor that considers vehicle traffic, pedestrian traffic, and trains. After the creation of the model, the second objective was initiated. This entailed testing the hypothesis that by controlling train speed and train variance we can increase safety and reduce delays.

METHODOLOGY

In order to create a working model in the VISSIM program, large amounts of field data, which included intersection layout, traffic volumes, and traffic signaling data, needed to be collected and compiled. Then, this data was coded into the VISSIM program to create the model for the corridor. This task included creating the layout for the model, inserting additional field data, and creating the traffic signaling files. The final task entailed running the VISSIM model to collect the traffic simulation data and then make changes to increase system efficiency and/or safety.

Intersection Layout

The intersection layout data was collected through the use of a Distance Measuring Instrument (DMI) in a TTI vehicle and through numerous site visits to the intersections. This data consisted of the following.

- Distance along Wellborn Rd.
- Length across the intersections
- Lane width
- Number of lanes and their configurations at each intersection
- Whether a turning bay was present, and, if present, its length

After this data was collected, some information was still missing. Due to the construction at the Joe Routt intersection, exact layout information was not available from the field at the time. Therefore, it was assumed that the intersection geometry would be as it was before the construction began. This data was found through old photos of the corridor as well as from a student's previous research project (4).

Traffic Volumes

The traffic volume data for this project came from a multitude of sources. Vehicle and pedestrian volumes were obtained for the George Bush and Old Main intersections from a traffic count done in September 2001 by the Texas A&M ITE Student Chapter. Their counts did not include the Holleman intersection, so the vehicle and pedestrian counts were taken for this project at the Holleman intersection on March 5, 2002. Unfortunately, the ITE counts were taken after construction had begun at the Joe Routt intersection, so the counts for this intersection were also lacking. Therefore, the traffic volumes for the Joe Routt intersection were found through the use of some older count data available from the City of College Station. Although the data for the Joe Routt intersection was about two or three years older than the other data, the volumes seemed to be reasonable, and they were deemed acceptable. All of the counts were taken during weekdays when the University classes were in session; therefore, the student population was near its peak. Although all of the counts included both AM and PM-peak volumes, this project looked only at the AM-peak period from 7:15-8:15. A figure of the AM-peak vehicle and pedestrian volumes used in the model can be found in Figure 2.

Traffic Signaling Data

In order to create the model to display the correct signaling, actual signal timings for the intersections would be needed. With the help of TTI researcher, Srinivasa Sunkari, the appropriate signaling data was collected from the City of College Station traffic-signal database. The signal timings were found for AM-peak, noon-peak, and PM-peak operations for each of the 4 intersections of concern; however, only the AM-peak timings were used for this project.

Coding Data into VISSIM

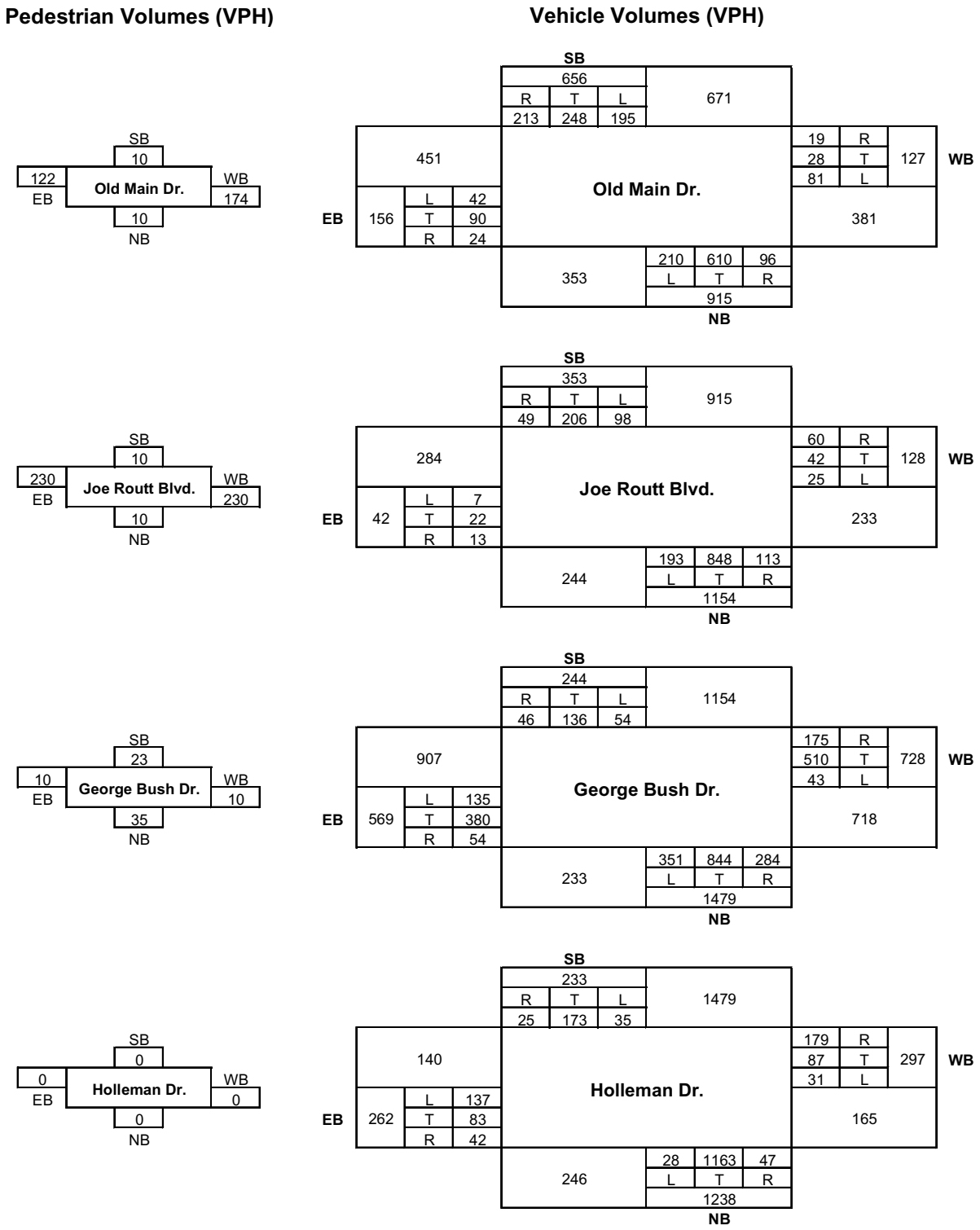
Intersection Layout

The creation of the VISSIM model began with the coding of the intersection layout. First, the lengths of the lanes, or "links" were created, and, then, the sections connecting the links at the intersections, or "connectors," were created as well. Finally, the railroad line was created as one link through the corridor.

During creation of the layout of the model, some generalizations were made about certain geometric aspects. Some basic generalizations about the specific lane geometry were also made. Although the actual lanes of Wellborn Rd. and the intersecting streets are not all of uniform width, every lane (including turning bays) was assumed to be 3.50 meters wide. This allowed for easier model creation and connectivity. Additionally, the actual left-turn bays are essentially continuous along Wellborn Rd., but this would not be feasible in the model. Instead, the lengths of the left-turn bays were shortened corresponding to the amount of use for each one. The lengths for the right-turn bays remained unchanged. The final geometric generalization involved the orientation of Wellborn Rd. with the railroad tracks. Although both of these do not run perfectly straight, it was assumed that Wellborn Rd. ran perfectly straight and that the railroad tracks ran parallel to the road. Since the tracks are offset from the

road by 21.5 m at the Old Main and Joe Routt intersections and by 11.0 m at the George Bush intersection, the tracks curved slightly between the Joe Routt and George Bush intersections to accommodate this change.

Figure 2. AM-peak Vehicle and Pedestrian Volumes



Traffic Volumes

Due to the construction at the Joe Routt intersection and the variety of sources for the volume data, the northbound outputs from the intersections did not match the northbound inputs at the next intersection to the north. This seems trivial because the only input volumes that were entered for Wellborn Rd. were the volumes for northbound vehicles at the Holleman intersection, those for the southbound vehicles at the Old Main intersection, and the side street volumes. Turning volume percentages were used for the northbound and southbound movements on the interior of the model. However, the actual volumes were used for all of the westbound and eastbound volumes.

In addition to the complexity added by the variety of sources for the volumes, some input errors were made with respect to the turning movement percentages. Although these errors were minor, they were not found until after the first set of runs was made, and they should be mentioned. The errors were caused by the fact that the data were collected at different times; therefore, the traffic volumes at adjacent intersections did not balance. The traffic volumes for north and south movements at adjacent intersections in the model were off by 7-9 % when compared to the actual counts, and a slight input error caused them to be off by another 1-2%. These errors are not thought to affect the results in any large amount for two reasons. First, for each of the ten runs made for each train speed distribution, VISSIM produced slightly different traffic volumes generated randomly for the intersections. Second, all of the results from this data are drawn from the comparison of the data generated from this model. Because these input errors were carried through the entire data collection process, the relative comparison of delays should not be adversely affected.

Traffic Signaling

For the traffic signal operation in the model, the vap logic file component of VISSIM was used instead of hardware-in-the-loop technology. The vap file method was chosen because, unlike hardware-in-the-loop, it allows a simulation to be run at various speeds faster than real-time. Because the goal was to run the simulation 10 times for each of the nine train speed distributions, this method would greatly shorten the data collection time for the project. Unfortunately, the creation of the vap files with preemption strategies proved to consume almost as much time as it saved.

Although all four of the intersections under consideration are operated in a coordinated mode in the field, for the purposes of this project, the signals in the model were operated in the actuated, free-running mode (without coordination). It was understood that this could cause the data to be somewhat skewed, but the time allotted for this project did not allow for the development of coordinated signaling logic with the corresponding railroad preemption logic. However, all of the other signaling data, including traffic actuation and pedestrian push button actuation, is the same as in the field.

Additional VISSIM Inputs

The model was run ten times for each of the nine train speed distributions. The distributions were made up of three mean speeds (30, 40, and 50 km/h) each with three standard deviations (5, 10, and 15 km/h). Each run was for 2400 seconds, and the train was sent into the model at 600 seconds for every run. Additionally, the train length used for the model was 1184 meters, an average train length for the Wellborn Corridor.

The mean speeds and the standard deviations used for this model were based on the actual train speeds in the corridor, and the actual train data used for this project is shown in Figure 3. This histogram is from a graduate student's current work with the corridor, and it shows the average train speeds at the George Bush intersection.

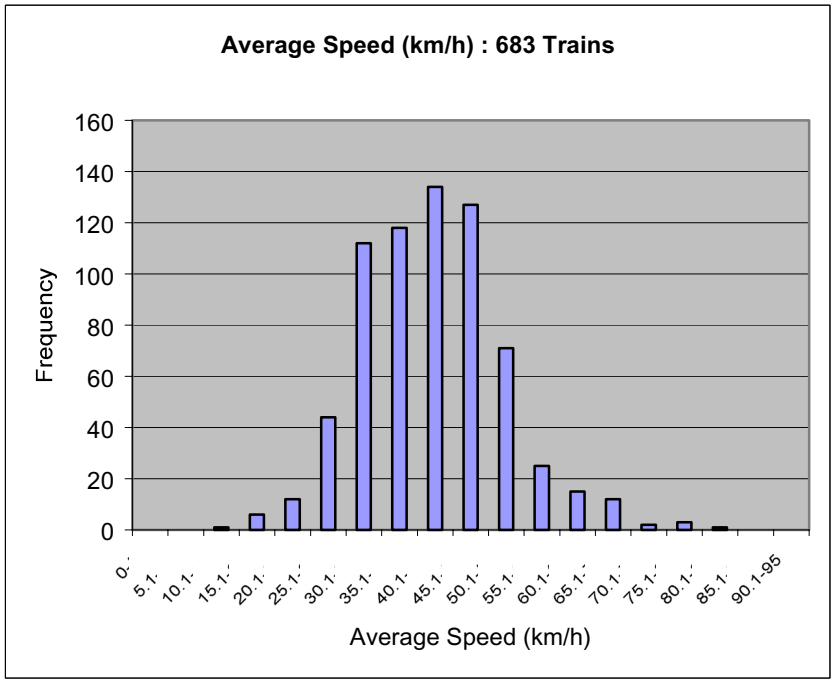


Figure 3. Actual Average Train Speeds at George Bush Dr.

The model was run ten times for each distribution using a train detection distance of 335 meters. This is based on the distance that a train traveling 60 km/h can travel in the required 20 seconds of warning time. Another set of runs were made in which the train detection distances were 280 meters and then 225 meters to detect trains traveling at 50 km/h and 40 km/h, respectively.

Collecting Data from VISSIM

The data collected from the runs included average vehicle delay, d_{ij}, for each movement at each intersection; however, this data was summarized to produce delay per intersection, d_j. The average delay for each intersection was found from Equation 1.

$$d_j = \frac{\sum_{i=1}^{N} V_{ij} d_{ij}}{\sum_{i=1}^{N} V_{ij}} \qquad \textbf{Eq. 1}$$

where

d_j = Average Delay at Intersection j
V_{ij} = Volume of Movement i at Intersection j
d_{ij} = Average Delay for Movement i at Intersection j
N = Number of Movements

Each distribution was run ten times for the 60 km/h train detection, and the distributions are as follows.

- 30 km/h mean with 5 km/h standard deviation
- 30 km/h mean with 10 km/h standard deviation
- 30 km/h mean with 15 km/h standard deviation
- 40 km/h mean with 5 km/h standard deviation

- 40 km/h mean with 10 km/h standard deviation
- 40 km/h mean with 15 km/h standard deviation
- 50 km/h mean with 5 km/h standard deviation
- 50 km/h mean with 10 km/h standard deviation
- 50 km/h mean with 15 km/h standard deviation
- 60 km/h mean with no deviation

Not all of the distributions were run for the other train detection distances. The 50 km/h train detection was run for only the following distributions.

- 30 km/h mean with 5 km/h standard deviation
- 30 km/h mean with 10 km/h standard deviation
- 40 km/h mean with 5 km/h standard deviation
- 50 km/h mean with no deviation

Finally, the 40 km/h train detection was run for only the 30 km/h mean with 5 km/h standard deviation and the base case of 40 km/h with no deviation.

RESULTS

Average delay was used as the main measure of effectiveness for this project. For each of the ten runs, average delay was collected for each intersection for each train speed distribution analyzed. Then, the ten runs were averaged, and five separate average delay times were found for each train speed distribution for the entire 40-minute period (one for each intersection and one average of the intersections). The summarized collected average delay data for the 40-minute period follows in Tables 1-3.

Table 1. Average Intersection Delay Times with 60 km/h Train Detection (seconds)

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	29.3	23.1	41.0	28.6	32.0
(30 - 10)	28.8	24.2	42.4	28.2	32.6
(30 - 15)	30.5	25.4	40.7	29.4	32.8
(40 - 05)	26.5	23.5	38.0	27.4	30.2
(40 - 10)	27.8	22.9	37.2	26.2	29.8
(40 - 15)	27.3	23.0	38.4	28.2	30.6
(50 - 05)	25.0	23.4	35.6	26.3	28.7
(50 - 10)	26.3	22.7	35.0	26.2	28.6
(50 - 15)	26.7	23.3	35.4	26.3	29.0
(60 - 00)	26.6	21.8	37.9	26.5	29.6

Table 2. Average Intersection Delay Times with 50 km/h Train Detection (seconds)

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds)				
	Old Main	**Joe Routt**	**George Bush**	**Holleman**	**Average**
(30 - 05)	29.2	24.0	39.6	27.8	31.5
(30 - 10)	29.6	24.1	40.0	28.2	31.9
(40 - 05)	27.7	23.2	37.8	26.4	30.1
(50 - 00)	24.9	22.4	37.7	25.9	29.3

Table 3. Average Intersection Delay Times with 40 km/h Train Detection (seconds)

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds)				
	Old Main	**Joe Routt**	**George Bush**	**Holleman**	**Average**
(30 - 05)	28.9	23.0	40.1	27.4	31.4
(40 - 00)	26.8	21.9	40.1	27.3	30.6

Because the data showed a large amount of similarity when average delay was taken over the entire 40-minute simulation period, the average delay was then found at 10-minute intervals for each train speed distribution and for each intersection. As expected, it was found that the first and fourth 10-minute intervals showed a large amount of similarity because these were the times when the train was not present in the model. In addition, the third interval showed some similarity, but not as much as the first and fourth. This is because, for some of the runs with slower train speeds, the trains were still present in the early part of the third 10-minute interval.

However, the data from the second 10-minute interval showed some differences due to the fact that this was the interval that included the trains. Although the second interval appeared to show differences, the data still needed to be statistically tested to prove any significant differences. The summarized collected average delay data for the second 10-minute interval follows in Tables 4-6, and the data for each of the 10-minute intervals is found in Appendix A.

Table 4. Average Intersection Delay Times with 60 km/h Train Detection (seconds)
Second 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 660-1200 seconds (Second block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	38.1	29.1	55.1	31.2	40.3
(30 - 10)	39.1	31.8	52.2	29.5	39.7
(30 - 15)	37.0	32.1	46.5	29.2	37.3
(40 - 05)	31.9	31.0	43.1	30.8	35.3
(40 - 10)	33.2	26.7	47.3	30.0	36.1
(40 - 15)	34.0	31.0	47.1	32.9	37.7
(50 - 05)	30.2	26.4	44.2	30.1	34.3
(50 - 10)	30.9	25.5	43.0	30.3	33.8
(50 - 15)	34.0	25.1	43.3	29.0	34.3
(60 - 00)	30.5	23.9	48.8	27.2	34.8

Table 5. Average Intersection Delay Times with 50 km/h Train Detection (seconds)
Second 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 660-1200 seconds (Second block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	38.6	31.9	54.5	32.1	41.0
(30 - 10)	43.1	30.2	51.9	29.5	40.0
(40 - 05)	33.2	26.7	46.6	28.8	35.5
(50 - 00)	27.8	24.1	49.7	30.2	35.4

Table 6. Average Intersection Delay Times with 40 km/h Train Detection (seconds)
Second 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 660-1200 seconds (Second block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	39.8	28.6	55.4	30.5	40.4
(40 - 00)	31.8	26.1	47.2	29.7	35.5

CONCLUSIONS

Statistical Testing

In order to prove if any differences exist in the data, the t-test statistical test was used. First, for each train speed distribution, intersection, and 10-minute interval, the 95% confidence interval of the estimate was found for the corresponding average delay. Using the average delay, $\bar{x}$, and the standard deviation of the 10 runs, s, a range was found for each train speed distribution from Equation 2 (5).

$$\bar{x} \pm \frac{t_{\alpha/2,n-1} s}{\sqrt{n}}$$

Eq. 2

where

$t_{\alpha/2,n-1}$ =	2.26, for 95% Confidence Interval
$\bar{x}$ =	Average Delay for Train Speed Distribution j
s =	Standard Deviation of Delay From 10 runs for Train Speed Distribution j
n =	Number of Simulation Runs, 10 for all cases

The confidence intervals were subsequently compared against each other. If any two ranges overlapped, the two distributions were not statistically different at the 95 % confidence interval. If the ranges did not overlap, the distributions were statistically different at the 95% confidence interval. The distributions are referenced in the following manner. The mean train speed of 30 km/h with a 5 km/h standard deviation for the 60 km/h train detection would be denoted as 60k 30-05, and the other distributions are noted similarly. The results from the statistical analysis are found in Appendix B.

Comparisons were made with distributions having the following characteristics.

- Same mean speed with same train detection distance Ex. (60k 30-05 with 60k 30-10)
- Same standard deviation with same train detection distance Ex. (60k 30-05 with 60k 40-05)
- Base mean speed (with no standard deviation) for the same train detection distance Ex. (60k 30-05 with 60k 60-00)
- Same mean speed and standard deviation with a different train detection distance Ex. (60k 30-05 with 50k 30-05)

Each distribution was compared to all other distributions with the same train detection distance and either the same mean speed, same standard deviation, or the base speed (with no standard deviation). Each distribution was also compared to the same distribution with a different train detection distance, if that distribution was used. Each comparison was done for all four intersections and for the average of the intersections, and the comparisons were divided into the four 10-minute intervals of the simulation.

As expected, no statistically different delays were found for the first 10-minute interval, and only three similarities were found in the third and fourth intervals (out of 630 comparisons). Therefore, it was hypothesized that in these time frames, the effect of the trains on traffic was negligible. This was confirmed visually.

For the second 10-minute interval, differences were found with the base mean speed; although, the differences were expected to be found in more than just 5 out of 65 comparisons.

Also, no differences were found for the same mean speed with different standard deviations. In general the delays rose with standard deviation as expected; however, there was no statistical difference. Since this was out of 120 comparisons, it was hypothesized that for this particular traffic setup, altering the standard deviations between 5, 10, and 15 km/h had no significant effect on the average delay.

Similarly, no cases showed a statistical difference for the same distribution with a different train detection distance. Again, the delay was expected to shorten as the train detection distance was lowered, but no significant difference was found.

The only meaningful differences appeared when the same standard deviations were used with different mean speeds at the same train detection distance. Comparing the data for each intersection and the average of the intersections, exactly 100 comparisons were made, and only 12 statistically different cases were found. However, if this data is compared for only the George Bush intersection, the result is 6 statistically different cases out of only 20 comparisons. This seems more substantial because the George Bush intersection is where the most traffic occurs; therefore, this is where one would expect the most significant impact on delay. Generally, delay decreased when the mean train speed increased, and the delays were shown to be statistically different in some of the comparisons.

Even though some statistically different cases were found for some cases involving different mean train speeds, the percentage of these cases was not high enough to be considered as showing a significant difference. Ultimately, it was found that manipulating the train detection distance, the mean train speed, or the train speed standard deviation did not have a significant effect on the average delay for the traffic layout that was modeled for this particular corridor.

Recommendations

One recommendation for future testing of this model would be to consider the pedestrian phasing during railroad preemption. During this project the number of times that pedestrian phases were truncated was not counted. Collecting this data would show to what degree pedestrian safety is compromised. Furthermore, this data could be compared to the delay data to possibly show that a tradeoff between the two exists.

Future testing of the model could also include using various larger and smaller traffic volumes to see the effect on the average delay. Sensitivity analyses could be performed to explore different train detection techniques as well as the number and length of the trains in the model. Another future test for the model could use emission data as a measure of effectiveness. Finally, the signal phasing could be updated to include coordinated signaling to again see the effect on average delay.

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APPENDIX A – SUMMARIZED DATA FROM SIMULATIONS

Average Intersection Delay Times with 60 km/h Train Detection (seconds)
First 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 0-600 seconds (First block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	26.4	19.8	27.7	23.6	25.0
(30 - 10)	27.2	20.1	28.1	24.0	25.5
(30 - 15)	26.4	19.8	27.7	23.6	25.0
(40 - 05)	25.5	19.2	26.9	23.1	24.2
(40 - 10)	25.2	19.4	27.7	23.6	24.7
(40 - 15)	26.7	19.0	26.2	22.6	24.1
(50 - 05)	26.4	19.8	27.7	23.6	25.0
(50 - 10)	26.4	19.8	27.7	23.6	25.0
(50 - 15)	26.5	20.7	27.5	23.3	25.0
(60 - 00)	25.6	20.3	28.9	24.9	25.7

Average Intersection Delay Times with 50 km/h Train Detection (seconds)
First 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 0-600 seconds (First block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	26.4	19.8	27.7	23.6	25.0
(30 - 10)	26.6	20.0	28.3	24.2	25.4
(40 - 05)	24.5	20.7	28.0	24.4	25.1
(50 - 00)	24.9	20.7	27.7	24.1	24.9

Average Intersection Delay Times with 40 km/h Train Detection (seconds)
First 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 0-600 seconds (First block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	26.3	19.4	28.5	23.9	25.3
(40 - 00)	26.8	21.0	28.0	23.9	25.5

Average Intersection Delay Times with 60 km/h Train Detection (seconds)
Second 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 660-1200 seconds (Second block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	38.1	29.1	55.1	31.2	40.3
(30 - 10)	39.1	31.8	52.2	29.5	39.7
(30 - 15)	37.0	32.1	46.5	29.2	37.3
(40 - 05)	31.9	31.0	43.1	30.8	35.3
(40 - 10)	33.2	26.7	47.3	30.0	36.1
(40 - 15)	34.0	31.0	47.1	32.9	37.7
(50 - 05)	30.2	26.4	44.2	30.1	34.3
(50 - 10)	30.9	25.5	43.0	30.3	33.8
(50 - 15)	34.0	25.1	43.3	29.0	34.3
(60 - 00)	30.5	23.9	48.8	27.2	34.8

Average Intersection Delay Times with 50 km/h Train Detection (seconds)
Second 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 660-1200 seconds (Second block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	38.6	31.9	54.5	32.1	41.0
(30 - 10)	43.1	30.2	51.9	29.5	40.0
(40 - 05)	33.2	26.7	46.6	28.8	35.5
(50 - 00)	27.8	24.1	49.7	30.2	35.4

Average Intersection Delay Times with 40 km/h Train Detection (seconds)
Second 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 660-1200 seconds (Second block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	39.8	28.6	55.4	30.5	40.4
(40 - 00)	31.8	26.1	47.2	29.7	35.5

Average Intersection Delay Times with 60 km/h Train Detection (seconds)

Third 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 1260-1800 seconds (Third block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	25.6	21.7	45.9	29.2	32.9
(30 - 10)	25.3	23.8	46.2	31.6	33.9
(30 - 15)	34.4	27.1	42.3	29.0	34.4
(40 - 05)	25.8	20.9	42.9	27.2	31.2
(40 - 10)	25.0	23.1	37.0	25.8	29.1
(40 - 15)	23.3	19.6	39.2	29.3	29.6
(50 - 05)	22.6	22.9	34.4	25.8	27.6
(50 - 10)	23.9	22.6	35.2	25.3	28.0
(50 - 15)	23.7	23.4	35.2	25.7	28.1
(60 - 00)	24.1	21.3	37.3	27.8	29.1

Average Intersection Delay Times with 50 km/h Train Detection (seconds)
Third 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 1260-1800 seconds (Third block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	25.8	22.1	39.9	28.2	30.7
(30 - 10)	25.6	21.6	44.0	32.5	33.1
(40 - 05)	26.4	21.2	38.5	26.5	29.7
(50 - 00)	23.5	21.3	36.0	24.3	27.8

Average Intersection Delay Times with 40 km/h Train Detection (seconds)
Third 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 1260-1800 seconds (Third block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	25.2	22.3	40.2	28.3	30.8
(40 - 00)	23.1	21.4	41.5	26.4	29.9

Average Intersection Delay Times with 60 km/h Train Detection (seconds)
Fourth 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 1860-2400 seconds (Fourth block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	27.4	22.2	35.2	29.5	29.6
(30 - 10)	24.3	20.9	41.9	27.2	30.6
(30 - 15)	24.0	22.0	44.4	34.8	33.5
(40 - 05)	22.9	21.9	37.5	27.7	29.0
(40 - 10)	27.4	21.7	35.8	25.1	28.8
(40 - 15)	25.8	22.4	39.1	27.3	30.2
(50 - 05)	21.9	23.7	35.1	25.3	27.7
(50 - 10)	24.4	22.2	33.2	25.0	27.1
(50 - 15)	23.0	23.5	34.4	26.6	28.0
(60 - 00)	26.0	21.3	35.0	25.7	28.2

Average Intersection Delay Times with 50 km/h Train Detection (seconds)
Fourth 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 1860-2400 seconds (Fourth block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	26.1	22.3	36.3	26.8	29.1
(30 - 10)	24.1	24.2	35.3	25.8	28.5
(40 - 05)	26.2	23.9	36.9	25.7	29.4
(50 - 00)	23.6	23.3	36.0	24.7	28.2

Average Intersection Delay Times with 40 km/h Train Detection (seconds)
Fourth 10-minute Interval

Train Speed (Mean – Std Dev) (km/h)	Average Intersection Delay (seconds) for 1860-2400 seconds (Fourth block)				
	Old Main	Joe Routt	George Bush	Holleman	Average
(30 - 05)	24.9	21.6	36.4	26.2	28.7
(40 - 00)	25.6	19.5	41.8	28.7	30.9

APPENDIX B – STATISTICAL ANALYSIS

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 30-05*** Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 30-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	60k 50-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 30-05	0	0	0	0	0
	40k 30-05	0	0	0	0	0
660-1200 seconds	60k 30-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-05	0	0	**1**	0	**1**
	60k 50-05	0	0	**1**	0	**1**
	60k 60-00	0	0	0	0	0
	50k 30-05	0	0	0	0	0
	40k 30-05	0	0	0	0	0
1260-1800 seconds	60k 30-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	60k 50-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 30-05	0	0	0	0	0
	40k 30-05	0	0	0	0	0
1860-2400 seconds	60k 30-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	60k 50-05	**1**	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 30-05	0	0	0	0	0
	40k 30-05	0	0	0	0	0

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 30-10*** Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 30-05	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 30-10	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	60k 30-05	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 50-10	0	0	0	0	**1**
	60k 60-00	0	**1**	0	0	0
	50k 30-10	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	60k 30-05	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 30-10	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	60k 30-05	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 30-10	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX

* Note: A train speed distribution of 60k **30**-*05* means that for the 60 km/h train detction, the trains had a mean speed of **30** km/h and a standard deviation of *5* km/h.

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 30-15*** Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 30-05	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	60k 30-05	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	60k 30-05	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	60k 30-05	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 40-05*** Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 40-10	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	60k 50-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 40-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	60k 40-10	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 30-05	0	0	**1**	0	0
	60k 50-05	0	0	0	0	0
	60k 60-00	0	**1**	0	0	0
	50k 40-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	60k 40-10	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	60k 50-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 40-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	60k 40-10	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	60k 50-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	50k 40-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 40-10*** Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 40-05	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	60k 40-05	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	60k 40-05	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	60k 40-05	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 40-15*** Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 40-05	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	60k 40-05	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	60k 40-05	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	60k 40-05	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 50-05*** Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 50-10	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	60k 50-10	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 30-05	0	0	**1**	0	**1**
	60k 40-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	60k 50-10	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	60k 50-10	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 50-10*** Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 50-05	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	60k 50-05	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 30-10	0	0	0	0	**1**
	60k 40-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	60k 50-05	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	60k 50-05	0	0	0	0	0
	60k 50-15	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	60k 40-10	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **60k 50-15***				
		Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	60k 50-05	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	60k 50-05	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	60k 50-05	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	60k 50-05	0	0	0	0	0
	60k 50-10	0	0	0	0	0
	60k 30-15	0	0	0	0	0
	60k 40-15	0	0	0	0	0
	60k 60-00	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **50k 30-05***				
		Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	50k 30-10	0	0	0	0	0
	50k 40-05	0	0	0	0	0
	50k 50-00	0	0	0	0	0
	40k 30-05	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	50k 30-10	0	0	0	0	0
	50k 40-05	0	0	**1**	0	**1**
	50k 50-00	0	0	0	0	0
	40k 30-05	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	50k 30-10	0	0	0	0	0
	50k 40-05	0	0	0	0	0
	50k 50-00	0	0	0	0	0
	40k 30-05	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	50k 30-10	0	0	0	0	0
	50k 40-05	0	0	0	0	0
	50k 50-00	0	0	0	0	0
	40k 30-05	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **50k 30-10***				
		Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	50k 30-05	0	0	0	0	0
	50k 50-00	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	50k 30-05	0	0	0	0	0
	50k 50-00	**1**	0	0	0	0
	60k 30-10	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	50k 30-05	0	0	0	0	0
	50k 50-00	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	50k 30-05	0	0	0	0	0
	50k 50-00	0	0	0	0	0
	60k 30-10	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **50k 40-05***				
		Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	50k 30-05	0	0	0	0	0
	50k 50-00	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	50k 30-05	0	0	**1**	0	0
	50k 50-00	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	50k 30-05	0	0	0	0	**1**
	50k 50-00	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	50k 30-05	0	0	0	0	0
	50k 50-00	0	0	0	0	0
	60k 40-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

1=Statistically Different
0=Not Statistically Different

Simulation Time	Train Speed Distribution (For Comparison)	Train Speed Distribution of Concern = **40k 30-05***				
		Old Main	Joe Routt	George Bush	Holleman	Average
0-600 seconds	40k 40-00	0	0	0	0	0
	50k 30-05	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
660-1200 seconds	40k 40-00	0	0	**1**	0	**1**
	50k 30-05	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1260-1800 seconds	40k 40-00	0	0	0	0	0
	50k 30-05	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
1860-2400 seconds	40k 40-00	0	0	0	0	0
	50k 30-05	0	0	0	0	0
	60k 30-05	0	0	0	0	0
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX
	XXX	XXX	XXX	XXX	XXX	XXX

JONATHAN TYDLACKA

Jonathan Tydlacka will receive his Bachelor of Science degree from Texas A&M University in December 2002. As an undergraduate, he worked on transportation research projects for both the University Undergraduate Fellows and the TTI Summer Undergraduate Fellows Programs. He will begin graduate school at Texas A&M in the spring of 2003 with the goal of pursuing a Master of Science Degree in Civil Engineering. Some of his interests include traffic operations, micro-simulation modeling, and ITS.

COSTS AND BENEFITS ASSOCIATED WITH ELECTRONIC TOLL COLLECTION AND VARIABLE TOLL RATES

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SUMMARY

Electronic toll collection (ETC) made its debut in Dallas in 1989 and has become increasingly popular in the thirteen years since. Many agencies are realizing the benefits of electronic toll collection and are converting their existing toll plazas to incorporate it. ETC has been shown to reduce congestion and improve traffic flow on America's toll roads. By using vehicle-mounted transponders to collect tolls, ETC can greatly reduce queue length and average delay at toll plazas. Use of ETC has also shown significant reductions in harmful emissions and fuel consumption.

By adding variable toll rates to an ETC-equipped toll system, additional benefits can be gained. By increasing the toll during peak periods, drivers are encouraged to shift their drive times from congested traffic periods into relatively uncongested periods. By doing this, average delay is reduced, subsequently increasing travel time savings, fuel savings, and emissions savings.

While much literature exists on the operational benefits of ETC, there have been few cost-benefit analyses performed on the implementation of this type of system. This paper seeks to quantify the costs and benefits of implementing ETC with regards to users, agencies, and the general public. It will quantify the costs and benefits associated with a flat toll rate ETC-equipped plaza versus a traditional plaza with only manual or automatic coin machine (ACM) toll collection. Then, the additional costs and benefits associated with using ETC with a variable toll rate will be examined.

This research followed standard practices for cost-benefit analysis. Costs and benefits were quantified using both actual and assumed monetary values. Actual values were used wherever possible, and assumed values were derived using the latest research available. The values were estimated for the evaluation period of ten years, and they were then reduced to a net present value. The net present values of costs and benefits were compared to find the net cost-benefit ratio of implementing the ETC system.

A net benefit of $7.64 million was calculated. The majority of the calculated benefits came from travel time savings, comprising more than 90% of the total. The savings from fuel reduction (8% of total benefits) was small compared to travel time savings but was significant nonetheless. Emissions reduction, however, was very small in comparison to other benefits. The savings from emissions reduction was approximately 1% of the total benefits. The benefit-cost ratio was calculated to be 1.40 for this analysis, with the benefits outweighing the costs by 40%.

An additional benefit of $74,908 was gained from the implementation of variable toll rates, a small amount in comparison to the benefits gained from ETC. However, this analysis assumed uncongested conditions. Variable toll rates would have a larger impact on a more congested toll plaza.

While in this particular study the net benefit was found to be positive, this will not necessarily be true in all cases. This research is merely a framework for cost-benefit analysis. A toll agency interested in evaluating the potential costs and benefits of implementing ETC will need to make assumptions based on the specifications of its toll facilities. It is also important to realize that while a positive net benefit may result, not every party involved will be affected positively. In this analysis, the majority of the costs were paid by the agency while the users gained most of the benefits. However, almost all ETC deployments are done by government agencies, which will most likely be concerned with the overall net benefit, not simply the benefit to the agency.

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INTRODUCTION

Electronic toll collection (ETC) made its debut in Dallas in 1989 and has become increasingly popular in the thirteen years since. Many agencies are realizing the benefits of electronic toll collection and are converting their existing toll plazas to incorporate it (*1*). Based on data from the Electronic Toll and Traffic Management (ETTM) website, over 40 toll authorities in 19 states are now using ETC (*2*) (see Figure 1). ETC has been shown to reduce congestion and improve traffic flow on America's toll roads (*3*). By using vehicle-mounted transponders to collect tolls, ETC can greatly reduce queue length and average delay at toll plazas (*4*). Use of ETC has also shown significant reductions in harmful emissions and fuel consumption (*5,6,7*).

Another method designed to increase the efficiency of roadway use is variable pricing. Variable pricing attempts to reduce congestion by giving drivers an incentive to drive during off-peak hours. Agencies set toll rates that vary with the congestion level of the toll road to reduce demand during peak times. This demand shifts to off-peak periods with excess capacity, thereby improving traffic flow.

While much literature exists on the operational benefits of ETC, there have been few cost-benefit analyses performed on the implementation of this type of system. This paper seeks to quantify the costs and benefits of implementing ETC with regards to users, agencies, and the general public. It will quantify the costs and benefits associated with a flat toll rate ETC-equipped plaza versus a traditional plaza with only manual or automatic coin machine (ACM) toll collection. Then, the additional costs and benefits associated with using ETC with a variable toll rate will be examined.

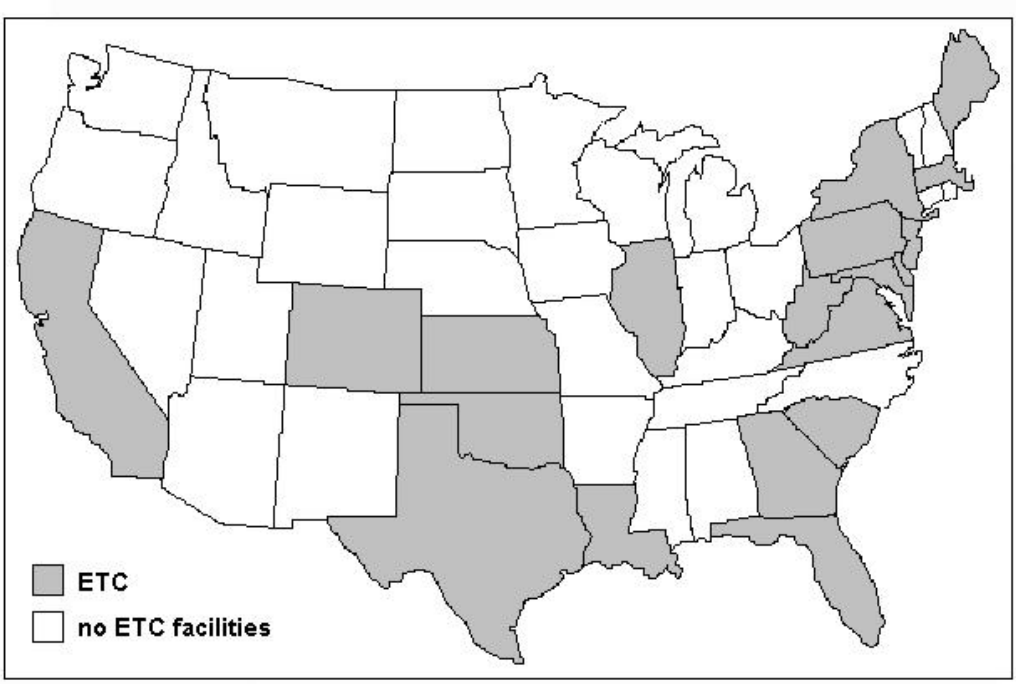

Figure 1. States Currently Using ETC

Electronic Toll Collection

ETC is made possible because of a technology known as automatic vehicle identification (AVI). AVI allows toll customers to mount small transponders to their vehicles and drive through toll plazas without

stopping. When an AVI-equipped vehicle passes through an ETC lane at a toll plaza, an antenna reads the unique transponder ID and deducts the toll amount from the customer's account. This appeals to drivers because it eliminates the need for cash payments at toll facilities. Typically, toll patrons obtain a transponder from the toll agency and are required to open an ETC account from which the tolls will be deducted. This can usually be done in cash or with a credit card.

ETC increases user benefits by lowering the average variable cost (AVC) of driving (see Figure 2). Benefits are defined as the area above the price and below the demand curve. The point at which the AVC intersects the demand curve determines the price and volume of a system. Tolls, fuel costs, and travel time costs are the main components of AVC. Tolls and fuel costs remain relatively constant with greater volumes of traffic, but travel time costs increase substantially. The implementation of ETC can lower travel time costs because of its reduced queuing and non-stop toll transactions. With travel time costs lowered, the AVC is lowered, reducing price and increasing benefits.

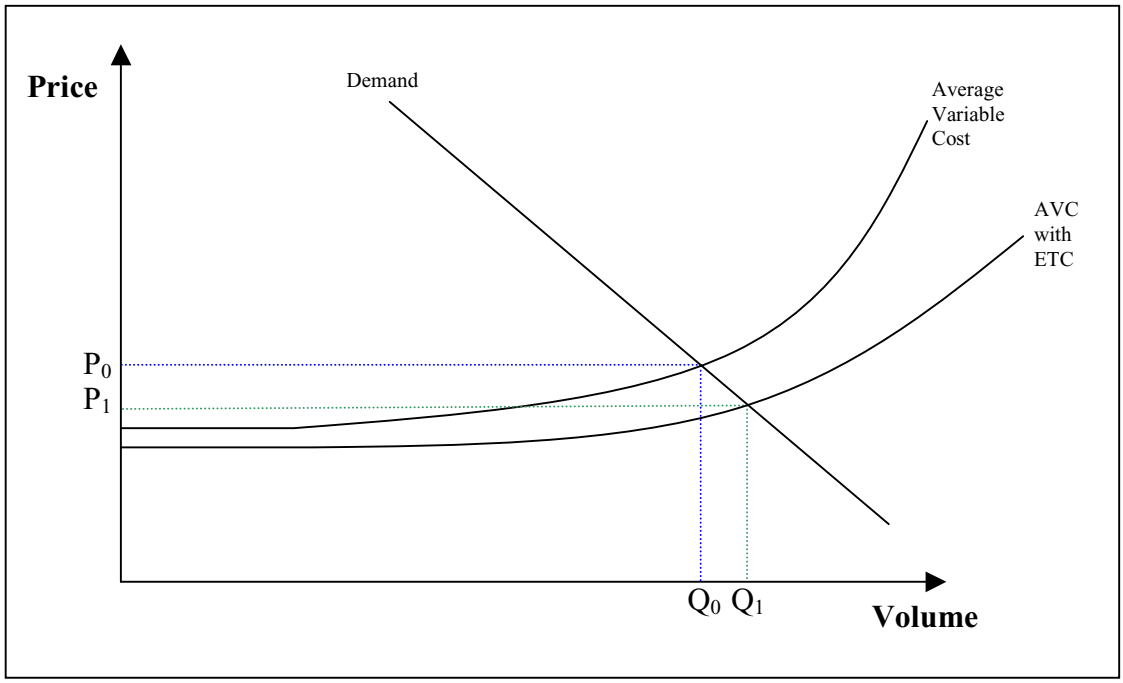

Figure 2. Change in AVC due to ETC implementation.

Variable Pricing

A standard demand graph is used to illustrate the costs and benefits of variable pricing (*8*) (see Figure 3). A driver's average variable cost (AVC) includes vehicle operating costs, the present toll, and travel time costs. It is primarily these travel time costs that cause the AVC to curve upward as volume increases. Without variable pricing, a volume of Q_0 can be expected at a price of P_0. Under these circumstances, the net societal benefit of travel is defined as the difference between the area $a+b+c+d$ and the triangular area h. The area h is referred to as welfare loss, and it occurs because drivers are not paying the marginal cost of using the toll road. Marginal cost is defined as the total additional cost of travel with the addition of one vehicle. Under congested conditions, the addition of a single vehicle can increase the travel time of all vehicles, yielding a high marginal cost.

Increasing the amount of the toll during congested conditions reduces demand for travel. In the case of marginal cost pricing, the toll is raised to be equal to the marginal cost of travel (P_1-P_0). This reduces

demand to Q_1 and eliminates welfare loss (*h*). At demand Q_1 user benefits are reduced to (*a*), while previous user benefits of *b*+*c* are transferred to the toll authority in the form of increased toll revenue. The benefits (*d*) are lost as some users (Q_0-Q_1) forego the trip on the toll road. Therefore, the change in net societal benefit is equal to *h*-*d*. Clearly, as congestion worsens, the marginal cost and AVC curves diverge and the potential benefit of marginal cost pricing increases.

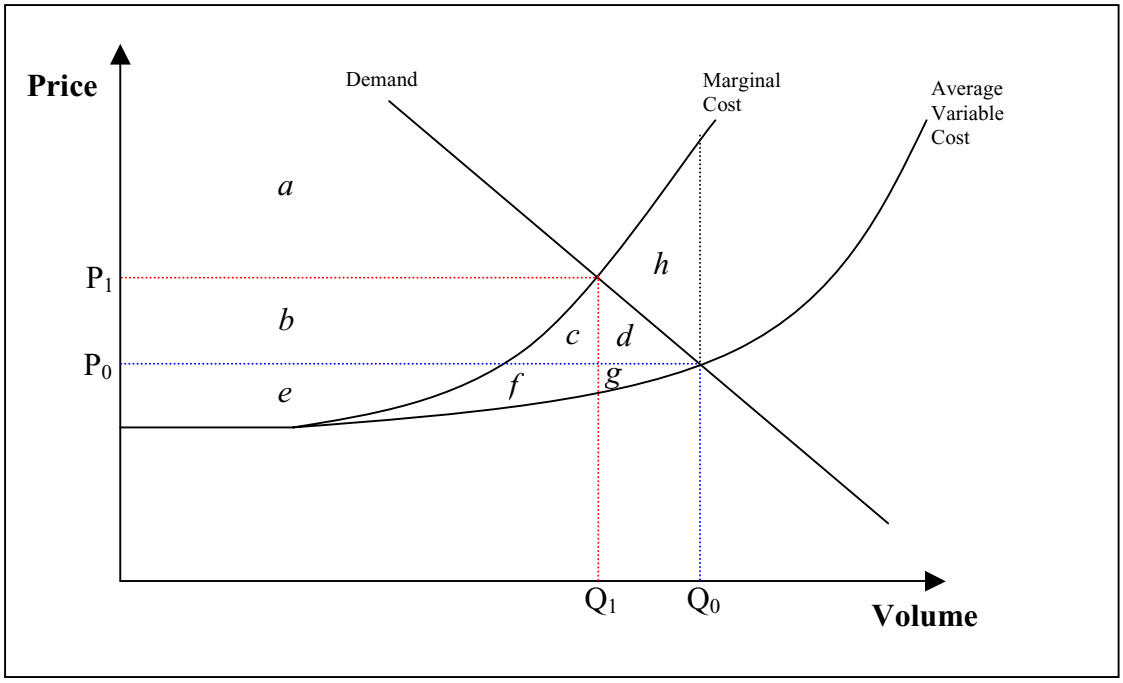

Figure 3. Impact of Marginal Cost on Benefits

METHODOLOGY

This research followed standard practices for cost-benefit analysis (*9*). Costs and benefits were quantified using both actual and assumed monetary values. Actual values were used wherever possible, and assumed values were derived using the latest research available. The values were estimated for the evaluation period, and they were then reduced to a net present value. The net present values of costs and benefits were compared to find the net benefit of implementing the ETC system. The benefit cost ratio for the alternatives examined was found using Equation 1.

$$BCR = \frac{net\ benefits}{net\ costs} \tag{1}$$

A benefit-cost ratio greater than one indicates that the benefits of installing the system outweigh the costs over the evaluation period. In this research, a ten-year evaluation period was used based on the expected life of the ETC technology (*10*). The equipment was assumed to have no salvage value. The present value of all annual costs was found using Equation 2, with a real discount rate of 3.1% (*11*). The real discount rate is defined as the difference between the nominal discount rate and the inflation rate.

$$PV = \sum_{n=1}^{10} A * \left[\frac{1}{(1+i)^{n-1}} \right] \tag{2}$$

where:

PV = present (year 1) value
A = annual cost
i = interest rate (3.1%)
n = year of evaluation period (1-10)

There were three costs that were not included in this analysis: interest on toll deposits, the cost of violation processing, and the cost of increased accident rates. Users are required to open and maintain an account with the toll agency with a minimum balance in the account at all times. Drivers lose the interest that could be earned if this money were invested rather than just placed into an ETC account. However, while this is a cost to the user, it becomes a benefit to the agency. Since this research deals with the net benefit, this value was not included in the results. The cost of violation processing was also not included in this research. It was assumed that the cost of prosecuting violators and collecting fines was equal to the revenue gained from the fines. Finally, the possibility of an increased number of accidents due to ETC implementation was not included in the analysis. Abdelwahab and Abdel-Aty found the use of ETC to increase the probability of accidents at toll plazas (*12*). However, a literature review found no other examples of increased accidents. Certain research found no significant correlation between ETC implementation and crash rates (*7*). Further research is necessary to adequately analyze the relationship between ETC and accidents.

For this research, a model toll plaza system was created based on typical toll plaza sizes across the country. The model system consisted of three toll plazas on a small toll highway. In this manner certain costs (such as transponders and the customer service center) were distributed across all three plazas. Each plaza was assumed to have twelve lanes, with eight automated coin machine (ACM) lanes and four manual lanes. Another model was then created of the toll plaza system after ETC implementation. The second model was also used for the cost-benefit analysis of ETC with variable pricing, because the implementation of variable pricing does not involve a change in toll plaza configuration. This model had a distribution of four dedicated ETC lanes, four ACM/ETC lanes, and four manual/ETC lanes in each toll plaza, with ETC equipment being installed in all lanes. It is common for toll plazas to have ETC equipment installed in all lanes, giving drivers further options (*13*). However, the additional equipment in the ACM/ETC and manual/ETC lanes was only considered during the cost estimates of this report. The additional benefits that result from the presence of this equipment were ignored in order to gain a more conservative estimate.

Table 1. Toll Plaza System Prior to ETC Implementation

	Each plaza	Entire system
ACM Lanes	8	24
Manual Lanes	4	12
TOTAL	12	36

Table 2. Toll Plaza System After ETC Implementation

	Each plaza	Entire system
ETC Only Lanes	4	12
ACM/ETC Lanes	4	12
Manual/ETC Lanes	4	12
TOTAL	12	36

QUANTIFYING COSTS AND BENEFITS

Baseline Costs

The baseline costs are the costs that would be incurred by maintaining the current toll plaza system for the evaluation period. It was assumed that these costs would only consist of operation and maintenance costs. Cost-benefit analysis is concerned with net costs. The net costs are found by subtracting the baseline costs from those associated with installing the ETC system:

$$Net\ Costs = Costs_{ETC} - Costs_{Baseline} \quad (3)$$

Research found approximate operation and maintenance costs of $176,000 per year for a manual lane and $15,800 for an ETC lane (*14*). It was assumed that the operation and maintenance cost of an ACM lane was approximately the same as that of an ETC lane. Using this information, a baseline cost of $21,797,817 was calculated for the evaluation period.

Costs of ETC

All costs of the new system will fall into one of six categories: installation, lane equipment, computer equipment, transponders, customer service and operation and maintenance. All costs were found in per lane values with a few exceptions. Only one plaza computer is needed at each plaza. Also, the host computer system and the customer service center serve the entire toll plaza system, and only one of each is needed. A summary of all costs is shown in Table 3 (see Appendix A for complete cost data and references).

Installation

All installation costs associated with adding ETC equipment to a lane are included in this category. Setting up ETC typically involves the installation of AVI hardware, automatic vehicle classification equipment, a video enforcement system, and all computer hardware and software. Installation may be performed by the agency itself or contracted out to another party. In most cases, the ETC equipment vendor handles installation. In this research, an approximate installation cost of $2,000 per lane was found (*15*). Multiplying this by the 36 lanes to be equipped with ETC systems yields a total installation cost of $72,000.

Table 3. Costs of Installing ETC

Category	Cost
Installation	$72,000
Lane Equipment	$2,160,000
Computer Equipment	$1,250,000
Transponders	$1,500,000
Customer Service Center	$1,000,000
Customer Service Staffing	$1,100,000 per year
Operations and Maintenance	$379,200 per year

Lane Equipment

The equipment necessary for ETC in a toll lane includes automatic vehicle identification (AVI) equipment, automatic vehicle classification (AVC) equipment, video enforcement equipment, and other items such as display signs. The AVI equipment in an ETC lane will typically include an antenna and a reader. The antenna receives the signal from the vehicle transponder. The reader identifies the driver by the unique transponder identification number. AVC equipment allows the toll lane to determine the type of vehicle in order to charge to correct toll. Most toll plazas use a post-classification system that identifies the vehicle after the transponder has been read. A video enforcement system (VES) is necessary to prosecute violators. A VES usually consists of two video cameras that capture photographs of each vehicle's license plates. Cost data obtained during this research found an average lane cost of $30,000. However, this estimate includes information for AVI equipment, AVC equipment, and signage, but not a VES. Alternate data found an average VES cost of $30,000. Therefore, total lane equipment costs were estimated to be $60,000 per lane. This gives a cost of $2,160,000 for the entire system.

Computer Equipment

The core of any ETC system is the host computer. Each toll plaza system will require a host computer and all necessary software. Additional hardware will also be necessary in each ETC lane. The system will also need additional data storage due to the large amount of information that must be kept regarding the toll users. Also included in the cost of computer equipment are auxiliary items such as printers, modems, cables, etc. The cost of computer equipment can vary depending on the quality of existing equipment. Those plazas with newer ACM systems may not need to invest as much capital into computer equipment as those with less state-of-the-art technologies. Research found average costs of $500,000 and $250,000 for host and plaza computer systems, respectively. The model toll plaza system required one host computer and three plaza computers for a total cost of $1,250,000.

Transponders

Each vehicle taking advantage of the ETC system will require a transponder. There are currently three types of transponders available. Type I transponders are read-only, Type II can store information and be written to, and Type III transponders can communicate with the driver through some type of interface. The majority of transponders used in the U.S. are Type I, and those in this research were assumed to be as

well. An important decision to be made in the implementation of ETC is where to place the cost of transponders. Agencies can purchase a large number of transponders and distribute them to users, or they can simply require that toll patrons purchase their own. Polling of toll authorities found that agencies generally chose to pay this cost themselves. Therefore, this analysis assumed the cost of the transponders to be paid by the agency. Polling of agencies also found an average transponder cost of $30. In order to determine the number of transponders that needed to be purchased, data from Lee County, Florida was analyzed. It was found that the number of transponders in circulation was about twice the average daily volume of ETC transactions. Volume and market share assumptions (to be discussed later in the report) found an average daily ETC volume of 25,000, requiring 50,000 transponders and a total cost of $1,500,000.

Customer Service

A large portion of the costs incurred will be due to the customer service center. This center will handle all ETC accounts as well as customer relations and the distribution of transponders. Implementation of the customer service center involves a large capital cost of construction, plus annual staffing costs. However, some of the staffing costs may be offset. ETC lanes, like ACM lanes, do not have to be manned by a toll employee. Implementation of ETC may reduce the need for tollbooth workers due to the reduction in manual lanes. However, additional staff will be required to handle users' toll accounts allowing the possibility of reassignment as an alternative to staff layoffs. The Oklahoma Turnpike Authority successfully accomplished this (*14*). Lee County, Florida was also able to forego eliminating toll workers (*16*). However, because this report assumed no manual lanes to be converted to dedicated ETC lanes, none of the service center staffing costs was assumed to be offset. Research found the average cost of opening a customer service center to be approximately $1,000,000, with annual staffing costs of $1,100,000.

Operation and Maintenance

The operation and maintenance costs of the toll plaza will change due to the installation of ETC. It was assumed that the operation and maintenance costs of an ACM lane are approximately the same as those of an ETC lane, due to their similarities. Therefore, the operation and maintenance costs of the four ACM lanes being converted to ETC were assumed to remain constant. The additional operation and maintenance costs incurred by implementing ETC will be those added by installing ETC hardware into the four remaining ACM lanes and the four manual lanes. Using the values defined previously, an operation and maintenance cost of $25,115,789 was calculated for the ETC-equipped plaza system for the evaluation period. However, cost-benefit analysis is concerned with net costs. Subtracting the baseline costs from this value gives an additional operation and maintenance cost of $3,317,972 for the ETC-equipped plaza.

After all costs had been calculated, the total was divided by the estimated number of ETC transactions over the evaluation period to get an average transaction cost. Dividing the total cost of $19 million by the estimated 270 million transactions yielded an approximate transaction cost of $0.07. Additionally, a model developed by Levinson was used to find the total operating costs (*17*). An approximate cost of $0.07 per transaction was found by dividing the total operating costs by the total number of transactions. The model is described with the equation:

$$Ln(TOC) = 1.087 + 0.477*(Ln(L)) + 0.490*(Ln(AT)) - 0.125*ETC \quad (4)$$

where:
TOC = Total operating costs (in thousands of dollars)
L = number of lanes in toll plaza
AT = annual number of transactions (in thousands of transactions)
ETC = average daily ETC market share

These values correspond well with other research, which showed transaction costs to range between $0.05 and $0.15 with an average value of $0.10 (*18,19,20,21*).

Benefits of ETC

The implementation of electronic toll collection yields many benefits (*4,5,6,7*), but the primary ones are travel time savings, fuel reduction, and emissions reduction.

Calculation of Travel Time Savings

Time savings are usually the largest contributor to benefits (*22*). The implementation of ETC impacts travel time savings in two different ways. The first is by shortening queues at toll plazas. The ETC lanes have higher service rates than manual and ACM lanes and can therefore process vehicles more efficiently. Implementing electronic toll collection also reduces travel times by reducing the time spent decelerating and accelerating. Toll patrons using ETC lanes have to decelerate to a moderate speed while those using manual and automatic lanes must come to a complete stop. Polling of toll agencies found ETC lane speeds ranging from 5 mph to 45 mph. The average ETC lane speed was approximately 30 miles per hour, and that value was used in this analysis. Because ETC users spend less time decelerating and accelerating than those using manual or automatic lanes, a driver using ETC lanes will cover a distance faster than one using manual or ACM, even when there are no queues in any lanes.

Using stochastic queuing analysis, the annual time spent in the toll system was calculated for the initial toll plazas and for the ETC-equipped plazas. Annual travel time savings were calculated as the difference between these two values. Values were calculated using two different daily volumes: those for work days and those for non-work days. A distribution of 250 work days and 115 non-work days per year was assumed. By assuming typical peaking characteristics on urban roadways, daily volumes were also divided into three time periods: peak, off-peak, and low. Tables 4 and 5 show the distribution of these time periods in a typical work day and non-work day. Due to the consistent nature of traffic flow on non-work days, only two hourly volumes were used for the analysis on these days: off-peak and low. Examination of several toll plazas found that market penetration of ETC is generally greater during peak periods than in other periods. Average market share of ETC was found to range from 19 to 76 percent, but in all cases was greater in the peak times. Therefore, this research assumed the plaza to have its largest ETC market share in the peak period (see Table 6). It is important to note that all of the assumed volumes describe the traffic at one toll plaza in one direction. All calculated benefits were then doubled to find the benefit of the entire plaza and then tripled to find the benefits of the entire toll system.

Table 4. Daily Volume Distribution for Work Days

Time period	Hourly volume	Hours per day
Peak	2250	3
Off-Peak	1425	13
Low	250	8

Table 5. Daily Volume Distribution for Non-Work Days

Time period	Hourly volume	Hours per day
Off-Peak	1200	14
Low	400	10

Table 6. ETC Market Share by Period

Time period	Work Day	Non-Work Day
Peak	65%	N/A
Off-Peak/Low	45%	45%

The following standard stochastic queuing formulas, which assume a negative exponential distribution for arrival and service rates, were used:

$$\rho = \frac{\lambda}{m\mu} \tag{5}$$

$$P_0 = \frac{1}{\sum_{n=0}^{m-1}\left(\frac{(\lambda/\mu)^n}{n!}\right) + \frac{(\lambda/\mu)^m}{m!(1-\rho)}} \tag{6}$$

$$Q = \frac{P_0(\lambda/\mu)^m \rho}{m!(1-\rho)^2} \tag{7}$$

$$W = \frac{Q}{\lambda} \tag{8}$$

$$t = W + \frac{1}{\mu} \tag{9}$$

$$AT = t * ((ADV_{Work} * 250) + (ADV_{Non\text{-}Work} * 115)) \tag{10}$$

$$ATS = AT_{Baseline} - AT_{ETC} \tag{11}$$

where:
λ = arrival rate (vehicles/hour)
μ = service rate (vehicles/hour)
m = number of lanes
ρ = traffic intensity
P_0 = probability of an empty system
Q = average queue length
W = average delay
t = average time spent in system
ADV_{Work} = average daily volume on work days
$ADV_{Non\text{-}Work}$ = average daily volume on non-work days
AT = annual time spent in system
ATS = annual time savings

To find the time savings gained from the change in lane speeds, two vehicles were compared over an arbitrary distance longer than that needed to completely decelerate and accelerate. Drivers were assumed to drive at highway speed and then rapidly decelerate as they approached the toll plaza. They then accelerated rapidly back to highway speed after paying the toll. Calculations were performed using deceleration and acceleration rates from *A Policy on Geometric Design of Highways and Streets* (*23*). Annual time savings was calculated by subtracting these two values and multiplying by the total number of annual ETC users.

Deceleration Time Savings:

$$t_1 = (v_{toll} - v_0)/a \tag{12}$$

$$D_{decel} = v_0 t_1 + (1/2)at_1^2 \tag{13}$$

$$t_2 = (D_{eval} - D_{decel})/v_0 \tag{14}$$

$$T_{decel} = t_1 + t_2 \tag{15}$$

$$TS = T_{Manual/Automatic} - T_{ETC} \tag{16}$$

$$ATS = TS * (ADV_{ETC\ Work} * 250 + ADV_{ETC\ Non\text{-}Work} * 115) \tag{17}$$

Acceleration Time Savings:

$$t_1 = (v_0 - v_{toll})/a \tag{18}$$

$$D_{accel} = v_{toll} t_1 + (1/2)at_1^2 \tag{19}$$

$$t_2 = (D_{eval} - D_{accel})/v_0 \tag{20}$$

$$T_{accel} = t_1 + t_2 \tag{21}$$

$$TS = T_{Manual/Automatic} - T_{ETC} \tag{22}$$

$$ATS = TS * (ADV_{ETC\ Work} * 250 + ADV_{ETC\ Non\text{-}Work} * 115) \quad (23)$$

where:
v_0 = velocity of toll road (65 mph)
v_{toll} = speed of toll lane (0 or 30 mph)
a = average deceleration/acceleration
t_1 = time spent decelerating/accelerating
t_2 = time spent traveling remainder of evaluation distance
D_{decel} *and* D_{accel} = deceleration or acceleration distance
D_{eval} = evaluation distance (1000 ft in advance of plaza, 3000 ft downstream of plaza)
T = time spent traveling evaluation distance
TS = time savings = 20,320 hours/year
$ADV_{ETC\ Work}$ = average daily volume of ETC lanes on work days
$ADV_{ETC\ Non\text{-}Work}$ = average daily volume of ETC lanes on non-work days
ATS = total annual time savings

To convert the total travel time savings into a monetary value, a measure of the average person's time value was needed. Research has shown the value of a driver's time to be about fifty percent of the national average wage (*24*). The average wage was found to be $15.46 per hour based on a forty-hour work week (*25*). However, the average occupancy of vehicles is also a factor of total time value. For this analysis, average passenger time was valued at thirty percent of the national average wage (*26*). To find the average number of passengers an average vehicle occupancy was needed. This was calculated using highway statistics from the FHWA (*27*). An average occupancy of 1.6 was found by dividing the total number of person-miles traveled by the total number of vehicle-miles traveled. The total time value was calculated to be $10.51 per vehicle-hour. This value was inflated according to an average wage index to calculate values for the entire evaluation period. Total time savings were found with the following equations:

$$T = (0.5*NAW) + (0.3*NAW*(1-AVO)) \quad (24)$$

$$ATSV = (ATS_{Queue} + ATS_{Decel} + ATS_{Accel}) * T \quad (25)$$

where:
T = time value
NAW = national average wage
AVO = average vehicle occupancy
$ATSV$ = annual time savings value
ATS_{Queue} = annual time savings from queuing reduction
ATS_{Decel} = annual time savings from shorter decelerations
ATS_{Accel} = annual time savings from shorter accelerations

Computer Models

Two computer models were used in the calculation of benefits due to fuel and emissions reduction. The models used were Comprehensive Modal Emissions Model (CMEM) and a model developed by Ahn, Rakha, Trani, and Van Aerde (*28*). The models were used to calculate fuel and emissions reductions due to the shortened deceleration and acceleration times of ETC. The behavior of a vehicle traveling from a point 1000 feet before the toll plaza to a point 3000 feet after it was analyzed. Calculations were made on a second-by-second basis for a vehicle traveling through both an ETC lane and a manual lane. The second-by-second data was inputted into the two models and the results were recorded. Fuel and

emission reductions due to ETC were calculated using these outputs. The average values of the two results were used in the final calculations.

Calculation of Fuel Reduction

Electronic toll collection can decrease fuel consumption by reducing the time spent idling in queue and the amount of decelerating and accelerating. The rate of idling fuel consumption was found through research and the outputs of the two models. Fuel reduction from shortened deceleration and acceleration was calculated from the outputs of the two models. To give a monetary value to the reduction, an average gasoline price was found. The price was then inflated using an average fuel index to calculate values for the entire evaluation period. The values used for the calculations are found in Table 7. Total fuel reduction is found using these equations:

$$AFS_{Queue} = ATS_{Queue} * FC_{Idle} \tag{26}$$

$$AFS_{Decel/Accel} = AT_{ETC} * FS_{Decel/Accel} \tag{27}$$

$$AFS = (AFS_{Queue} + AFS_{Decel/Accel}) * G \tag{28}$$

where:

AFS_{Queue} = annual fuel savings from queuing reduction (gallons)
ATS_{Queue} = annual time savings from queuing reduction (hours)
FC_{Idle} = average fuel consumption when idling (gallons/hour)
$AFS_{Decel/Accel}$ = annual fuel savings from acceleration reduction (gallons)
AT_{ETC} = annual number of ETC transactions
$FS_{Decel/Accel}$ = fuel savings of one vehicle using ETC (gallons/transaction)
AFS = annual fuel savings ($)
G = average price of fuel ($/gallon)

Table 7. Fuel Reduction Values

Fuel Consumption (idling)	0.440 gallons/hour [1,2,3]
Average Fuel Savings from using ETC	0.00444 gallons/transaction [2,3]
Average Fuel Price	$1.50/gallon [4]

Sources: [1]Biggs and Akcelik (*29*), [2]Comprehensive Modal Emissions Model, [3]Ahn et al. (*28*), [4]American Automobile Association (*30*)

Calculation of Emissions Reduction

Electronic toll collection can also reduce emissions due to the travel time savings gained from shorter queues and reduced decelerating and accelerating. Emissions reduction was measured for three types of pollutants: nitrous oxides (NO_x), volatile organic compounds (VOC), and carbon monoxide (CO). This research deals with only these three pollutants because much of the literature on emissions reduction due to ETC (*5,6,7*) focused on these three pollutants. Also, due to limitations of the computer models used, these were the only three that could be evaluated. This is a very unfortunate shortcoming of the computer

software, particularly because emissions of particulate matter cannot be modeled. Research has shown the health effects of particulate matter emission to be much greater than that of the three pollutants analyzed in this report (*31*). Emissions reduction due to shorter queue times were found using idling emission rates (see Table 10). Emissions reduction from the reduced acceleration and deceleration was found using average values of the outputs from the two computer models. The outputs of the models can be seen in Table 8. The values used in the final calculations are displayed in Table 9. Using the outputs of the two models, the emissions reduction was found for one car using ETC rather than a manual or ACM lane. This value was then multiplied by the annual number of ETC transactions to find the annual emissions reduction. To give the reductions a monetary value, the costs associated with these pollutants were obtained from research done by Delucchi (*31*). Delucchi's values are based on the health cost of motor-vehicle pollution. The values were inflated according to an average health care index to calculate values for the entire evaluation period (*32*). The following equations were used:

$$AER_{Work} = ADV_{ETC\ Work} * (E_{60\text{-}0\text{-}65} - E_{65\text{-}30\text{-}65}) * 250 \tag{29}$$

$$AER_{Non\text{-}Work} = ADV_{ETC\ Non\text{-}Work} * (E_{60\text{-}0\text{-}65} - E_{65\text{-}30\text{-}65}) * 115 \tag{30}$$

$$AER_{Idle} = ATS_{Queue} * E_{Idle} \tag{31}$$

$$AER = AER_{Work} + AER_{Non\text{-}Work} + AER_{Idle} \tag{32}$$

$$AES = AER * EC \tag{33}$$

where:
AER_{Work} = annual emissions reduction from work days due to less deceleration and acceleration
$ADV_{ETC\ Work}$ = average daily volume of vehicles paying by ETC
$E_{0\text{-}65}$ = emissions of one 65-0-65 deceleration/acceleration
$E_{30\text{-}65}$ = emissions of one 65-30-65 deceleration/acceleration
$AER_{Non\text{-}Work}$ = annual emissions reduction from non-work days due to less acceleration
$ADV_{ETC\ Non\text{-}Work}$ = average daily volume of ETC lanes on non-work days
AER_{Idle} = annual emissions reduction due to shorter queues
ATS_{Queue} = annual time savings from queuing reduction
E_{Idle} = idling emission rate
AER = total annual emissions reduction
AES = annual emissions savings ($)
EC = monetary value of emissions reduction ($/kg)

Table 8. Emissions Outputs (per vehicle)

	NO_x		VOC		CO	
	CMEM	**Ahn et al.**	**CMEM**	**Ahn et al.**	**CMEM**	**Ahn et al.**
Vehicle using manual/ACM lane	0.274 g	0.852 g	0.0428 g	0.321 g	1.38 g	8.62 g
Vehicle using ETC lane	0.242 g	0.761 g	0.0395 g	0.309 g	1.27 g	8.63 g
Reduction from using ETC	0.032 g	0.091 g	0.0033 g	0.012 g	0.11 g	-0.01 g

Table 9. Reduction Values Used in Calculations

NO_x	0.062 grams/transaction
VOC	0.0077 grams/transaction
CO	0.05 grams/transaction

Table 10. Average Idling Emission Rates (grams/hour)

Pollutant	Summer	Winter	Average
NO_x	4.72	6.16	5.44
VOC	16.1	21.1	18.6
CO	229	371	300

Source: Environmental Protection Agency (*33*)

Table 11. Values of Emissions ($/kg)

Pollutant	Low	High	Average
NO_x	$1.59	$23.34	$12.47
VOC	$0.13	$1.45	$0.79
CO	$0.01	$0.10	$0.06

Source: Delucchi (*31*)

Benefits of Variable Pricing

By adding variable toll rates to the ETC-equipped toll system, additional benefits can be gained. By increasing the toll during peak periods, drivers are encouraged to shift their drive times from congested traffic periods into relatively uncongested periods. By doing this, average delay is reduced, subsequently increasing travel time savings, fuel savings, and emissions savings.

With the implementation of variable pricing, new volumes were calculated for the peak and off-peak periods. The toll was assumed to be raised from $1.00 to $1.50 during the peak period, an increase of 50%. To determine the number of cars that would move their driving time from the peak period to the off-peak period, an elasticity of demand was needed. Price elasticity of demand describes the effect a change in price has on demand. For this analysis, a price elasticity of demand of –0.2 was assumed, a conservative estimate based on variable pricing projects currently in place. To find the volume reduction the peak period, the following elasticity equation was used:

$$E = \frac{(q_2 - q_1)/q_1}{(p_2 - p_1)/p_1} \tag{34}$$

where:
E = price elasticity of demand
q = volume
p = price (toll)

When determining the volume transfer from the peak period to the off-peak period, numerous assumptions were made:

- The amount of traffic was constant
- Drivers only switched if they were previously driving within one hour of off-peak time
- Elasticity of demand of –0.2
- Volume transferred to off-peak hours on either side of the peak period

Because of the toll increase, two additional traffic volumes were created to represent the periods involving the drivers that switched their times. Hourly volumes after the implementation of the variable pricing can be seen in Table 12, with the time periods affected by the variable pricing system highlighted in bold.

Table 12. Daily Volume Distribution for Work Days with Variable Pricing in Place

Time period	Hourly volume	Hours per day
Peak 1	2250	1
Peak 2	**2025**	**2**
Off-Peak (Shoulder)	**1650**	**2**
Off-Peak	1425	11
Low	250	8

RESULTS

A summary of the calculated costs and benefits can be seen in Table 13. The creation of a customer service center and its subsequent staffing amounted to the largest portion of total costs, accounting for approximately 56% of the entire amount. It is important to note that because of the large costs involved with implementing a customer service center, the number of toll plazas served by that center is very significant. If a service center is implemented, for instance, to service a single-plaza toll facility, the benefits gained from ETC implementation will probably not outweigh the costs. This research, however, assumed a three-plaza system, and a net benefit of $7.64 million was calculated. The majority of the calculated benefits came from travel time savings, comprising more than 90% of the total. The savings from fuel reduction (8% of total benefits) was small compared to travel time savings but was significant nonetheless. Emissions reduction, however, was very small in comparison to other benefits. The savings from emissions reduction was approximately 1% of the total benefits. The benefit-cost ratio was calculated to be 1.40 for this analysis, with the benefits outweighing the costs by 40%.

Table 13. Summary of Costs and Benefits

COSTS	
Installation	- $72,000
Lane Equipment	- $2,160,000
Computer Equipment	- $1,250,000
Transponders	- $1,500,000
Customer Service Center	- $1,000,000
Service Center Staffing	- $9,624,919
Operation and Maintenance	- $3,317,972
BENEFITS	
Travel Time Savings	+ $24,037,970
Fuel Savings	+ $2,225,413
Emissions Savings	+ $297,374
NET BENEFIT	**+ $7,635,866**

Additional benefits were calculated for the implementation of a variable pricing system during the peak periods. The additional benefits gained totaled $74,908, which is small in comparison to the total benefits of ETC. However, this research assumed relatively uncongested conditions at the toll plazas. Under this low level of congestion, the transfer of vehicles from the peak periods to off-peak times was less effective. With a higher level of congestion, the impact of variable pricing would be much greater.

Table 14. Additional Benefits from Variable Pricing

	Flat Toll ETC	Variable Toll ETC
Travel Time Savings	$24,037,970	$24,107,856
Fuel Savings	$2,225,413	$2,229,773
Emissions Savings	$297,374	$298,036
TOTAL	$26,560,757	$26,635,665
Additional Benefit	**$74,908**	

CONCLUSION

In this research, the costs and benefits of electronic toll collection were analyzed. A net benefit of $7.64 million was calculated for the implementation of ETC over a ten-year evaluation period. Additional benefits of almost $75,000 were found for the addition of variable toll rates, a small value in comparison to the benefit gained from ETC. However, this analysis assumed uncongested conditions. Under more congested conditions, the impact of variable pricing would be greater.

While in this particular study the net benefit was found to be positive, this will not necessarily be true in all cases. This research is merely a framework for cost-benefit analysis. A toll agency interested in evaluating the potential costs and benefits of implementing ETC will need to make assumptions based on the specifications of its toll facilities. It is also important to realize that while a positive net benefit may result, not every party involved will be affected positively. In this analysis, the majority of the costs were paid by the agency while the users gained most of the benefits. However, almost all ETC deployments are done by government agencies, which will most likely be concerned with the overall net benefit, not simply the benefit to the agency.

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APPENDIX A: COST DATA USED IN ESTIMATIONS

Source	Cost of ETC installation (per lane)
Kansas Turnpike Authority	$16,909 (low)
Kansas Turnpike Authority	$32,405 (high)
Gillen, et al.	$92,772
NCHRP Synthesis 194	$15,400
North Texas Tollway Authority	$62,800

Item	Cost	Source
AVI Equipment	$8,568 per lane	Kansas Turnpike Authority
AVI Equipment	$8,664 per lane	Transcore iServer
AVI Equipment	$10,839 per lane	Lee County, Florida
AVI Equipment	$11,000 per lane	North Texas Tollway Authority
Variable Message Sign	$2,600	Mark IV
Vehicle Classification System	$8,000 per lane	North Texas Tollway Authority
VES (w/ 3 cameras)	$40,000 per lane	North Texas Tollway Authority
Violation Camera (average)	$7,500	ITS Unit Cost Database
Computer Equipment/Software	$6,954	Kansas Turnpike Authority
Host Computer Equipment	$100,000	North Texas Tollway Authority
Software Development	$2,000,000	North Texas Tollway Authority
ETC Software (low)	$5,000	ITS Unit Cost Database
ETC Software (high)	$10,000	ITS Unit Cost Database
Service Center	$1,050,444	Lee County, Florida
Service Center Staffing	$1,081,029 per yr.	Lee County, Florida
Plaza Computer Subsystems	$25,887 per lane	Gillen, et al.
Host Computer Subsystem	$588,724	Gillen, et al.
Computer Equipment/Software	$128,300	NCHRP Synthesis 194
Host Computer Equipment	$307,400	NCHRP Synthesis 194
Operating Cost of ETC Lane	$15,800 per year	Oklahoma Turnpike Authority
Operating Cost of Manual Lane	$176,000 per year	Oklahoma Turnpike Authority

Source	**Cost per ETC transaction**
Smith, 2002	$0.05 (low)
Smith, 2002	$0.10 (high)
Miriam Daughtry, Virginia DOT	$0.10
Peter Samuel, Editor, Toll Roads Newsletter	$0.10
Lee County, Florida	$0.15
AVERAGE	$0.10

APPENDIX B: DETAILED SUMMARY OF COSTS AND BENEFITS

Item	Year										Total
	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	
COSTS											
Installation	72,000	-	-	-	-	-	-	-	-	-	72,000
Lane Equipment	2,160,000	-	-	-	-	-	-	-	-	-	2,160,000
Computer Equipment	1,250,000	-	-	-	-	-	-	-	-	-	1,250,000
Transponders	1,500,000	-	-	-	-	-	-	-	-	-	1,500,000
Customer Service Center	1,000,000	-	-	-	-	-	-	-	-	-	1,000,000
Service Center Staffing	1,100,000	1,066,925	1,034,845	1,003,730	973,549	944,277	915,884	888,346	861,635	835,728	9,624,919
Operation and Maintenance	379,200	367,798	356,739	346,013	335,609	325,518	315,730	306,237	297,029	288,098	3,317,972
BENEFITS											
Travel Time Savings	2,414,107	2,411,810	2,409,515	2,407,223	2,404,932	2,402,644	2,400,358	2,398,074	2,395,793	2,393,513	24,037,970
Fuel Savings	250,540	243,865	237,368	231,045	224,889	218,898	213,066	207,390	201,865	196,487	2,225,413
Emissions Savings	31,698	31,245	30,799	30,360	29,926	29,499	29,078	28,663	28,254	27,851	297,374
NET BENEFIT											**7,635,866**

*all values are in 2000 dollars

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Justin is currently pursuing a Bachelor of Science Degree in Civil Engineering at Texas A&M University and plans to graduate in December 2003. He worked for the Texas Department of Transportation during the summer of 2001 and is currently employed with the Texas Transportation Institute. He is a member of the Texas A&M University Student Chapter of the Institute of Transportation Engineers as well as the Texas A&M Chapter of Chi Epsilon Civil Engineering Honor Society where he currently serves as Associate Editor of the Transit. He also performs regularly with Percussion Studio, a student percussion ensemble at Texas A&M.