6-2016

Comparison of Safety and Operational Performances for Three Engineering Countermeasures

Ali Hamzah Hussein Alzuhairi
Western Michigan University

COMPARISON OF SAFETY AND OPERATIONAL PERFORMANCES FOR THREE ENGINEERING COUNTERMEASURES

by

Ali Hamzah Hussein Alzuhairi

A thesis submitted to the Graduate College
in partial fulfillment of the requirements
for the degree of Master of Science
Civil and Construction Engineering
Western Michigan University
June 2016

Thesis Committee:
- Jun-Seok Oh, Ph. D., Chair
- Valerian Kwigizile, Ph. D.
- Ron Van Houten, Ph. D.

COMPARISON OF SAFETY AND OPERATIONAL PERFORMANCES FOR THREE ENGINEERING COUNTERMEASURES

Ali Hamzah Hussein Alzuhairi, M.S.E

Western Michigan University, 2016

While there have been many studies on engineering treatments for reducing traffic crashes or for improving intersection efficiency, few studies have been simultaneously taking both impacts into consideration. This thesis analyzed impacts of engineering countermeasures and determines when these countermeasures are cost effective with respect to the amount of traffic and the number of crashes. Both crash reduction and operational costs were compared for analysis. This study specifically investigated three countermeasures: changing from permitted to protected for a left-turn on minor approaches, leading pedestrian interval (LPI), and exclusive pedestrian phase (Barnes Dance). The general Safety Performance Functions (SPFs) from the Highway Safety Manual (HSM) were used to calculate the average number of crashes for all crash types; these values were set as the base. Crash Modification Factors (CMFs) available in Crash Modification Factor Clearinghouse for these countermeasures were used to calculate the number of crashes reduced. Meantime, traffic operational performances were evaluated through VISSIM microscopic traffic simulation. Both crash reduction and additional delay were compared with varying traffic conditions. There were trade-offs between safety and operational performances. In order to determine cost effective conditions, cost-benefit analyses at different traffic conditions were performed. This thesis provides a general guideline for decision makers to determine if the treatment options are cost-effective in both aspects.

DEDICATION

To my county IRAQ

To my father who taught me how to face obstacles with persistence and strength,

To my mother, the kindest woman in the world,

To my caring and supportive brothers and sisters,

I dedicate this work for you all with respect and love!

ACKNOWLEDGEMENTS

First and foremost, I would like to thank my supervisor Dr. Jun-Seok Oh who supported, advised, and encouraged me a lot during my study at Western Michigan University. I would also like to thank the other members of my Master committee: Dr. Valerian Kwigizile and Dr. Ron Van Houten. Their insight, feedback, and advice were influential and essential throughout my research.

I would also like to thank my best friends Ahmed Abbas Alzubaidi and Hussein Khalaf for their support during my study. I would also like to thank all of my friends and lab coworkers.

A word of thanks should be recorded for the Higher Committee for Education Development in Iraq (HCED) for their financial support. I am really indebted to my home country (Iraq) for granting me this scholarship to pursue my Master's degree at Western Michigan University. No word can express my sincere indebtedness to my homeland, Iraq.

Finally, I would like to thank my family for supporting me during this journey. I wished there was room on my diploma to write the names of my family.

Ali Hamzah Hussein Alzuhairi

TABLE OF CONTENTS

Table of Contents–Continued

LIST OF TABLES

LIST OF FIGURES

List of Figures–Continued

List of Figures–Continued

CHAPTER 1

INTRODUCTION

Over the last several years an increase in crash frequency has been observed on the nation's roadways, especially at the intersections. According to National Highway Traffic Safety Administration (NHTSA), 8,598 fatalities have been observed at intersections during 2013 in the United States. The nation's roadways have become riskier with more congested traffic and an increased number of crashes. Therefore, an effort has been put forth to develop countermeasures that would decrease crash frequency and severity. The majority of the studies concentrated on evaluating countermeasures effectiveness toward crash reduction. However, few studies have taken into consideration the impact of those countermeasures on traffic operational efficiency. According to American Association of State Highway and Transportation Officials (AASHTO), every one hour delay per vehicle costs about $9.10. This could result in a large cost, ranging from several hundred thousand to a few million, being spent on implementing a countermeasure. Additionally, substantial amounts of vehicular emission will be produced due to the stop and go traffic at the intersections. Therefore, it is important to evaluate each countermeasure from both traffic safety and traffic operations perspectives. This thesis analyzed impacts of specific crash countermeasures and provided in which condition these countermeasures are cost-effective by computing both savings from crash reduction and additional costs due to changes in intersection operations.

Research Problem and Motivation

To improve traffic safety for motor vehicles, bicyclists and pedestrians, especially at intersections, many agencies have started implementing countermeasures in a systematic approach. Recommending appropriate countermeasures has been addressed for each certain traffic safety problem. However, few studies have taken into consideration the impact of those countermeasures on traffic operation (e.g. delay, travel time, and vehicular emission) when recommending safety countermeasures. Many countermeasures could have resulted in a negative impact on traffic operation. Consequently, millions of dollars can be saved if both safety and operational efficiency are considered and treated simultaneously. Additionally, health concerns associated with vehicular emission should be considered in the studies. Recent research reveals that human exposure to air pollutants, such as Carbon monoxide (CO), mono-Nitrogen Oxides (NOx), and Particulates (PM_{10} and $PM_{2.5}$), can lead to respiratory diseases, especially among school-age children. This study will consider three countermeasures at different locations in the United States from perspectives of traffic safety, delay, and emission.

Objective

The goal of this study is to increase the understanding of the relationship that exists between traffic safety and operational efficiency. The main purpose of this thesis is to evaluate the engineering countermeasures from both traffic safety (e.g., the number of crashes) and traffic operation perspectives (e.g. delay, travel time, and vehicular emission). Engineering countermeasures that have been implemented to reduce crashes are evaluated economically in term of operational cost, as well as crash cost. Both crash cost and operational cost are compared by quantifying crash reduction and delay increase due to each countermeasure. In addition, the study uses a range of average daily traffic values in order to provide a general guideline to help decision makers when determining cost-effective countermeasures.

Scope of the Study and Thesis Format

This thesis is limited to the evaluation of three countermeasures (left-turn changing phase, leading pedestrian interval, and exclusive pedestrian phase) that have been implemented at different locations in the United States. Safety effectiveness (e.g. provide a Crash Reduction Factor) of these countermeasures were provided by Crash Modification Factor Clearinghouse. Finally, the content will be presented in five chapters: literature review (Chapter 2), methodology (Chapter 3), analysis effectiveness of countermeasures (Chapter 4), and conclusion, recommendation, and limitation (Chapter 5).

CHAPTER 2

LITERATURE REVIEW

Existing literature concentrating on the evaluation and analysis of different safety engineering countermeasures was reviewed. This literature review mainly focuses on operational performance measures, including vehicular emission, and safety performance measures of the engineering countermeasures.

Operational Performance Modeling

Operational efficiency at intersections can be indicated by the level of service (LOS). LOS mainly represents the relations between demand and supply, larger demand than supply leads to traffic congestion (Knoop, 2009). Delay and travel time are the major aspects that determine the LOS and traffic operation effectiveness at intersections (Xi et al., 2015). In other words, delay and travel time are the two variables that can characterize the operational efficiency of an intersection. Therefore, delay is one of the major concerns for professional transportation studies. By converting it to a monetary value, delay can effectively reflect the inconvenience caused by traffic signal timing and other signal characteristics to the road users (Mousa, 2002). Therefore, in order to evaluate the performance of an intersection, average traffic delay should be calculated (Olszewski, 1993). Average traffic delay can be measured by calculating the difference between travel time when a vehicle is unaffected by the controlled intersection and when a vehicle is affected by the controlled intersection (Mousa, 2002), or it can be interpreted as the difference in travel time for a vehicle crossing an intersection before and after a change or treatment at the intersection. This delay calculation is accomplished in two steps. First, a distance between two unaffected points, upstream and downstream from the intersection, is established.

Then the difference between the ideal and actual travel time between that points is calculated to represent the delay time at the intersection. The calculated delay results from deceleration, acceleration, and stop delay time (Mousa, 2002).

Many models are available and can be used to estimate average vehicle delay at intersections. The Highway Capacity Manual (HCM) model is one of the most popular models used to estimate vehicle delay at intersections. The HCM delay model essentially considers a 15 minute time period at under-saturated conditions. For over-saturated conditions, the HCM model predicts higher vehicle delay compared to other models. Therefore, the degree of difference in delay estimation is increase with increasing saturation degree (Akgungor and Bullen, 1999). Akgungor and Bullen (1999) developed time dependent delay models to estimate vehicle delay at signalized intersections. They stated that capacity, traffic volume, green time, degree of saturation, analysis time period, and arrival patterns of vehicles are the parameters that can contribute to the delay in the model. They concluded that the degree of saturation and analysis time period are the most important parameters to estimate the vehicle delay. Dion et al. (2004) clarified five different models used for delay estimation at signalized intersections including deterministic queuing model, shock wave delay model, steady-state stochastic delay model, time-dependent stochastic delay model, in addition to the delays estimated from the microscopic traffic simulation software. Estimated delays by different models were compared over a range of v/c ratios (0.1 to 1.4) to evaluate their consistency. In the end, they proved that different models essentially provide similar delay results for signalized intersections. The results showed that there is a strong consistency between delays calculated by time-dependent stochastic and INTEGRATION microscopic traffic simulation model.

In another study, Ban et al. (2009) estimated delay using a model that used sampled travel times at signalized intersections. Sampled travel times are attained by two observations at two different locations, one upstream and another downstream of a signalized intersection. Therefore, in this model there is no need to know the cycle length and other traffic characteristics. Ghasemlou et al. (2015) demonstrated a comparison between three different delay models including the Nassiri and Nadernejad model, HCM model, and Akçelik model. The three models were applied to five different studies for over-saturated traffic conditions. The results showed that the three models gave approximately the same vehicle delay at signalized intersections.

Tian et al. (2002) examined two microscopic traffic simulation models, including VISSIM, for the delay performance measure at signalized intersections. Essentially, average vehicle delay estimated through simulation models is calculated by comparing the ideal travel time (i.e. without signal control at free flow speed) and actual travel time, which may be at lower speeds than free flow conditions. The results showed that simulation models for delay calculation are affected by the link length and speed, even though those two parameters are not included in the HMC model. The results revealed that shorter links with higher speeds would show lower delays. Additionally, the number of simulation runs can affect the accuracy of the results. In general, completing multiple simulation runs results in a smaller range of error in the estimated delay.

Finally, Xu et al. (2013) calculated the average delay at a signalized intersection in Beijing using VISSIM software. They also compared delay results obtained from VISSIM to results acquired using spot sample method. In VISSIM simulation, a single vehicle was used to define the running behavior and to calculate more precise delay. For running behavior, VISSIM was calibrated for lane change, following behavior, and lateral behavior. Additionally, the study considered multiple runs for more accurate results. On the other hand, spot sample methodology is summarized by "counting the number of vehicles that stop behind the stopping line, the number of vehicles that passed the stopping line after their stops, and the vehicles that do not stop behind the stopping line in average 15s" (Xu et al., 2013). The findings showed that delay results calculated using VISSIM are strongly consistent to those calculated using the spot sample method. Therefore, VISSIM simulation is accurate enough to be used for delay calculation.

Vehicular Emissions Analysis

As the level of congestion and duration of congestion increases, vehicular emissions and concentrations also increase accordingly. This fact is especially observed near congested freeways and arterials. Vehicular emissions, including carbon monoxide (CO), carbon dioxide (CO_2), volatile organic compounds (VOCs) or hydrocarbons (HCs), nitrogen oxides (NOx), particulate matters (PMs), and other pollutants have been attributed as a major source of air pollution (US National Research Council, 2002). Recent research has revealed that long time exposure to air pollutants can cause short-term health problems, such as headaches, nausea, skin and eye irritation, and nose, throat, and lung inflammation, as well as long-term respiratory and cardiovascular health problems, such as asthma and heart disease (Donham et al., 1990). Evidence shows that such health impacts are particularly significant on children (Ries et al., 2010). According to Ries et al. (2010), children are particularly vulnerable to airborne pollution because of their narrower airways and the fact that they breathe more air per pound of body weight than adults, which increases their exposure to air pollutants.

In another study, Abou-Senna et al. (2013) showed that a large amount of vehicular emissions occurs at speeds less than or equal to 20 mph. It has been reported that driver behavior, position in the queue, lane volume, and posted link speed are all factors that can significantly influence emission rates (Hallmark et al., 2002; De Vlieger, 1997; Chu & Meyer, 2009). Frequent acceleration and deceleration on the link were found to have significant impacts on the total emissions (Nesamani et al., 2007). To assess such health impacts, the current practice is to compare the emission concentrations collected by community-wide monitors to the air quality standards, such as EPA's Air Quality Standards.

Safety Analysis (Estimation of Crash Modification Factor CMF)

The literature review in this section addresses safety studies that include an estimation of Crash Modification Factors (CMFs). CMFs are a value generated by estimating the reduction in crashes after implementing a specific countermeasure. In other words, the CMF is "a quantitative statement of the result which a countermeasure is expected to cause when implemented" (Davis, 2000). A CMF is derived from the number of crashes experienced before implementing a treatment and the number of crashes that occurred after its implementation. CMFs can be calculated using multiple methods, e.g. naïve analyses of observational before and after crash data.

The naïve analysis method simply includes a comparison between the number of crashes at an identified hazardous location before and after treatment. However, naïve analysis cannot estimate the actual reduction in crashes due to a treatment; therefore, it cannot estimate the actual effectiveness of the countermeasure. In other words, lower crash values have a tendency of following higher values, which in turn make it difficult to interpret the effectiveness of the countermeasure. This tendency of the lower value to follow the higher values is called the regression to the mean (Campbell and Stanley, 1963). Therefore, many methods have been recommended to correct for the regression to the mean bias. Essentially, crash comparison should be done between the before and after period without implementing the countermeasure.

The Empirical Bayesian (EB) approach is a recommended method to estimate CMFs. This method relies on being able to predict crash frequency at an identified site for the period after treatment, prior to the actual treatment occurring. Once the treatment is completed, the predicted crash frequency is compared with the actual after-treatment crash frequency within the countermeasure area. Both the Federal Highway Administration's Interactive Highway Safety Design Model (IHSDM) and the Highway Safety Manual (HSM) provide established guidelines for predicting crash frequencies. These developed models can be used to estimate the crash frequency for the periods before and after the treatment. The models depend on multiple exposure factors, such as traffic volume, and can be developed by combining data from a reference group of untreated sites with pretreatment data from the treated sites (Davis and Aul, 2007). By using this method, it is possible to generalize a linear model that can explain and describe the variation of crash frequency. However, one of the limitations of the EB method is that crash data for the treated and comparison groups is required to be overdispersed; otherwise using the EB method can be problematic. However, it is not possible to know for certain all the factors that might affect the crash frequency after treatment. This in turn results in an estimation of a CMF in which not all the uncertainties have been considered. It is stated that if the estimation of CMFs are processed by a local condition on its own, these restrictions tend to be significant. This may result in a smaller amount of treated locations when compared to the state or nationwide database (Davis and Aul, 2007).

Before-after study with comparison group method is another approach used to estimate the level of safety at treated sites and calculate the CMF. Even though the reference group method needs a large sample of untreated sites, it is currently the most widely used for safety analysis. The reference group method is used to develop a Safety Performance Function (SPF) from crash data at untreated sites, which is then used to calculate the predicted crash frequency at treated sites. Moreover, the comparison group method can consider other factors that might affect crash occurrence that are not related to the implementation of the countermeasure. Therefore, it is important to separate the impact of crashes due to a treatment from those crashes that are due to factors unrelated to the countermeasure. Reference sites should be selected carefully in order to match the treated sites in terms of traffic characteristics and roadway geometry. Another shortcoming of the comparison group method is the need for 'comparability' between the treatment and reference (or comparison) sites. The term comparability refers to crash trends in the comparison group that are substantially homogenous to the treatment groups in both the before and after treatment periods. In other words, an increase in crashes by 5% per year in the treatment group during the before period should also be seen in the comparison group. The sequence of odds ratios can be calculated from historical crash counts to test the comparability between the treatment and comparison groups (Fayish and Gross, 2010).

Michigan U-turn Evaluation

As the importance of traffic crash treatment increases, the resulting traffic congestion and cost of traffic delay along with other operational factors due to the treatment need to be considered. Few studies have been done to investigate the balance between crash saving and operation cost.

Michigan U-turn design was one of the evaluated countermeasures for safety and operational performance. Michigan U-turn design is another name for Superstreet, restricted crossing U-turn (RCUT), and J-turn designs. Inman and Haas (2012) preformed an evaluation of

RCUT design in Maryland. The evaluation of the RCUT intersection on a rural four-lane divided highway was done by field observations. The field data collected included safety performance and traffic mobility after implementing the RCUT. The number of conflict points and weaving behavior were used in evaluating safety measures; mobility measures were represented by travel times and acceleration lane usage. Inman and Hass (2012) found that average travel time of the through movement was 19 second before treatment, and was increased to 83 second after implementing the countermeasure. They also found that the average travel time had increased from 28 seconds to 80 seconds for the left-turn movement. Therefore, the additional 4000 ft. required to complete the movement after treatment will increase the average travel time by about one minute per movement. The study included an observation on the addition of acceleration/deceleration lanes in order to show the extent of the utilization of those lanes by the right turning vehicles from the minor road. The observation found that acceleration and deceleration lanes were used by a majority of turning vehicles. Therefore, Inman and Haas (2012) recommended that acceleration/deceleration lanes be required for future RCUT designs.

Hummer et al. (2010) also evaluated the operational and safety benefits of Superstreet designs in North Carolina. The study involved evaluation of both signalized and un-signalized Superstreet designs; signalized Superstreets were evaluated for only operation performance, while un-signalized Superstreets were evaluated for safety performance. Probe vehicles provided with a GPS were used to measure average vehicle travel time. The probe vehicles were driven several times during a 90 minute period. The study found that changing a conventional signalized intersection to a signalized Superstreet reduces the overall average travel time per vehicle traveling through the intersection. In addition, the study found that there is a significant reduction in crashes due to changing a Stop-control intersection to an un-signalized Superstreet.

The third study was a research project done by Edara et al. (2013) on addressing the effectiveness of the J-turn intersection design in Missouri. Edara et al. evaluated the J-intersection by using field studies, crash analysis, and a public opinion survey. The study used a set of intersections with traditional two-way stop control (TWSC) as a reference group for operational comparison. The operational measures involved travel times, waiting time, and acceleration lane use. The travel times of vehicles turning left from the major road to the minor road were measured for the J-turn and TWSC. The study showed that average travel time at J-turn sites was greater than that at the TWSC sites by approximately one minute. However, the results showed that the waiting time at J-turn sites was half of that at TWSC sites. Once again, the increase in travel time at J-turn sites was due to the additional distance that a vehicle needs to make a U-turn to complete the movement.

CHAPTER 3

METHODOLOGY

This chapter describes the methodology for the two main aspects of this thesis: traffic safety analysis and operational performance analysis. The traffic safety study involves the estimation and analysis of predicted crash frequencies at signalized intersections by using Safety Performance Functions (SPFs) available in the Highway Safety Manual (HSM, 2009). The safety analysis section also describes the approaches used to evaluate the safety effectiveness of each selected countermeasure. In addition, the operational performance section involves conducting before and after treatment analysis. These results are used to obtain the operational performance of each countermeasure that is conveyed in terms of traffic delay, vehicle queue length, and vehicular emission. The results from the traffic safety and operational analyses are then converted to total annual crash benefit (i.e. saving) and operational cost to output the final cost-effective impact of implementing a countermeasure. In other words, annual total crash saving is compared to the total annual operational cost impact related to the safety treatment in order to determine the final cost and benefit of each countermeasure.

Safety Analysis

This section describes the approach used to calculate the predicted crash frequency given a specific location and traffic volume. Three countermeasures have been chosen as a case study and are evaluated in this thesis. The countermeasures being assessed have already been installed at different locations around the United States. The locations were identified based on studies addressed by the Crash Modification Factors Clearinghouse website.

The three selected countermeasures are leading pedestrian interval, exclusive pedestrian phase (i.e. Barnes Dance), and left-turn phasing change on minor approach from permitted to protected phasing. Each selected countermeasure addressed by Crash Modification Factor Clearinghouse website has an associated Crash Modification Factor (CMF) and Crash Reduction Factor (CRF). The CMF Clearinghouse presents multiple categories of studies for each countermeasure considered. The studies present crash analysis and safety effectiveness of the countermeasure. Moreover, the Crash Modification Factor Clearinghouse shows the crash type, crash severity, and area type when the countermeasure was implemented. It also addresses the quality of the studies. Consequently, the three selected countermeasures were chosen with high quality and different CMFs in order to demonstrate the countermeasure performance in terms of safety and operation.

For the present study, the process defined in the Highway Safety Manual (HSM) was used for calculating the predicted crash frequency before and after implementing the countermeasure at the treated intersections. The purpose of choosing the method outlined in the HSM is that it predicts the crash frequency at a variety of traffic volume and, therefore, encompasses more scenarios. This process will be explained in more detail later in this section.

In the HSM, the base condition of Safety Performance Functions (SPFs) for predictive crash frequency at urban and suburban arterial intersections is described (Chapter 12.6). The section includes the SPFs of four types of intersections: three-leg intersections with stop control on the minor-road approach, three-leg signalized intersections, four-leg intersections with stop control on the minor-road approaches, and four-leg signalized intersections. Additionally, each type of intersection has four categories of crash type: multiple-vehicle collisions, single-vehicle collisions, vehicle-pedestrian collisions, and vehicle-bicycle collisions. Each type of collision has its own SPFs with corresponding traffic volumes on major and minor approaches and the associated coefficients.

In this thesis, the locations of the countermeasures obtained from the CMF Clearinghouse were used as a case study; then, the HSM's SPFs are applied for those locations. The SPFs are adjusted for the effect of individual geometric design and traffic control features by using Accident Modification Factors (AMFs). The AMFs involve left-turn lane, intersection left-turn signal phasing, intersection right-turn lane, intersection right-turn on red, lighting condition, and red light cameras at multiple-vehicle collisions and single vehicle collision only and at signalized intersections. Then, a vehicle-pedestrian collisions' model is calibrated for existing bus stops, schools, and alcohol sales establishments.

Additionally, local calibration (C) is taken into consideration at each selected intersection. This calibration considers the varying geometric region, climate, animal population, driver population, crash reporting threshold, and crash reporting practices unique to the location (Highway Safety Manual, 2009). If the calibration factor for the intersection (C_i) is greater than one, the intersection experienced a larger number of crashes than indicated by the SPF; conversely, a value less than one specifies a lower number of crashes than indicated by the SPF. However, no crash data is available at the selected intersections for calibration. Therefore, the value of C_i is taken to be one for all studied intersections.

The adjusted SPFs are then used to predict the crash frequency prior to implementing the countermeasure. This calculation is completed for different crash types including multiple-vehicle collisions, single vehicle collision, vehicle-pedestrian collision, and vehicle-bicycle collision. The calibrated SPF is applied to estimate the total crash frequency for the targeted intersection given the alternative traffic volume projections. To predict crash frequency after the treatment when the countermeasure is applied, a CMF for that countermeasure is applied. As a result, the number of annual crashes reduced due to a treatment can be calculated from the crash frequencies of the before and after treatment periods.

Finally, annual crash saving (i.e. benefit) will be calculated based on the reduction in annual crashes due to a treatment. Because all crash severity levels have occurred at the treated intersections, overall crash cost will be considered in the total crash cost calculations. The overall crash cost for the analyzed crash conditions were extracted from a report that related injury severity to cost in Michigan (Kostyniuk et al., 2011). Table (7) provides a summary of the crash costs for different crash severities in Michigan. From Table (7), weighted average costs for total, fatal and injury (FI), and property damage only (PDO) crashes were used for the analysis.

Table 1: Michigan Crash Costs for KABCO Crashes (Kostyniuk et al., 2011)

Traffic Crash Casualty Severity	**Traffic Crash Casualties**	**Traffic Crash Costs**	**Traffic Crash Costs / Traffic Crash Casualties**
Fatal (K)	871	$3,146,015,418	$3,611,958
Incapacitating Injury (A)	6511	$1,495,225,106	$229,646
Non-Incapacitating Injury (B)	16149	$1,105,092,219	$68,431
Possible Injury (C)	48271	$1,926,495,610	$39,910
Property Damaged Only (O)	382424	$1,411,144,560	$3,690
Weighted Average Cost			
Fatal and Injury (KABC)	$106,860.93		
Total (KABCO)	**$19,998.80**		

Operational Performance Analysis

The following section describes the methodology used for the operational analysis conducted in this thesis. The operational analysis involves total intersection traffic delay, maximum queue length, Carbon monoxide (CO) vehicular emission, mono-Nitrogen Oxides (NOx) vehicular emission, and fuel consumption. Before and after intersection treatment periods were simulated and data was collected in order to complete operational analysis. Traffic signal optimization and characteristics (e.g. cycle length and phase scheduling) were constructed using Synchro software. The signal optimization software Synchro was used to analyze typical signal timing and traffic conditions for the network during the periods before and after implementing the treatment. In addition, the operational analysis also involved calibrating and validating VISSIM models of the three existing countermeasure. Data required in VISSIM was entered including intersection geometry, flow characteristics, signal timing, and turning characteristics for the two periods and the corresponding scenarios. Flow characteristics include the vehicle and pedestrian volume, approach speed, and traffic composition. Multiple runs are considered in each scenario in order to reach the most reliable results. Again, each countermeasure was applied at a specific intersection; then, that intersection was used as a case study (i.e. base scenario) for operational analysis. Eventually, the net value of each operational factor between before and after treatment is calculated for the cost-benefit analysis.

For the cost-benefit analysis and calculation, traffic delay and vehicular emission costs are considered for each evaluated countermeasure. The American Association of State Highway and Transportation Officials (AASHTO) considered the average vehicle delay cost to be $9.10 per hour. Vehicular emission, including CO and NOx, costs were defined by the US Environmental Protection Agency (EPA). It was found that the cost of CO is $37 per ton and the cost of NOx is $550 per ton. Table (8) provides a summary of costs for all factors considered during the economic calculation.

Table 2: Cost of Safety and Operation Factors

Factor	**Cost**
Crash	19999 ($/crash)
Delay	9.10 ($/hour)
CO	37 ($/ton)
NOx	550 ($/ton)

Generally, the methodology used for determining crash benefit (i.e. saving) and operational cost was as follows:

- Total Crash Saving per Year: multiply the number of crashes reduced by the overall crash cost.

  Total Crash Saving = Number of Crashes Reduced * Overall Crash Cost

- Total Delay Cost per Year: subtract the vehicle delay before treatment from after treatment delay, take this net value change and multiply it by the total hourly delay per year and multiplied that by the delay cost per hour.

  Total Delay Cost = (Net Value of Delay in hours * AADT * 365 days)*(Delay Cost)

- Total CO and NOx Cost per Year: subtract the emission before treatment from the after treatment emission value, then multiply the net value change by the total tons per year and multiply that by emission cost per ton.

  Total Emission Cost = (Net Value of Emission * 24 hours * 365 days)*(Emission Cost)

This process is repeated for seven scenarios, considering the base scenario with actual traffic volume and then six other scenarios that considered varying traffic volume. The base scenario uses the actual Annual Average Traffic Volume (AADT); other scenarios are considered that use varying volumes calculated by multiplying the actual AADT by 0.7, 0.8, 0.9, 1.1, 1.2, and 1.3 for each of the scenarios, respectively. These scenarios are then evaluated for before and after treatment for both safety and operational performance. The study uses a range of average daily traffic values in order to provide a general guideline to help decision makers when determining cost-effective countermeasures.

Also, the general SPF, acquired from the HSM, was used to calculate the average number of crashes for all crash types; these values were set as the base. The CMF developed for this countermeasure was used to calculate the amount of savings resulting from a reduction in crashes. By applying factors to the base number of crashes (average crashes reduced), the cost and benefit were observed for varying amounts of crashes, which were crash factors multiplied by the base number of crashes as shown in Table 3.

Table 3: Countermeasure's Scenarios with Different AADT and Crash Factors

Scenario	Volume Used	Crashes Factors
4	70% * AADT	Factor * Average Crashes Reduced
3	80% * AADT	Factor * Average Crashes Reduced
2	90% * AADT	Factor * Average Crashes Reduced
1	100% * AADT	Factor * Average Crashes Reduced
5	110% * AADT	Factor * Average Crashes Reduced
6	120% * AADT	Factor * Average Crashes Reduced
7	130% * AADT	Factor * Average Crashes Reduced

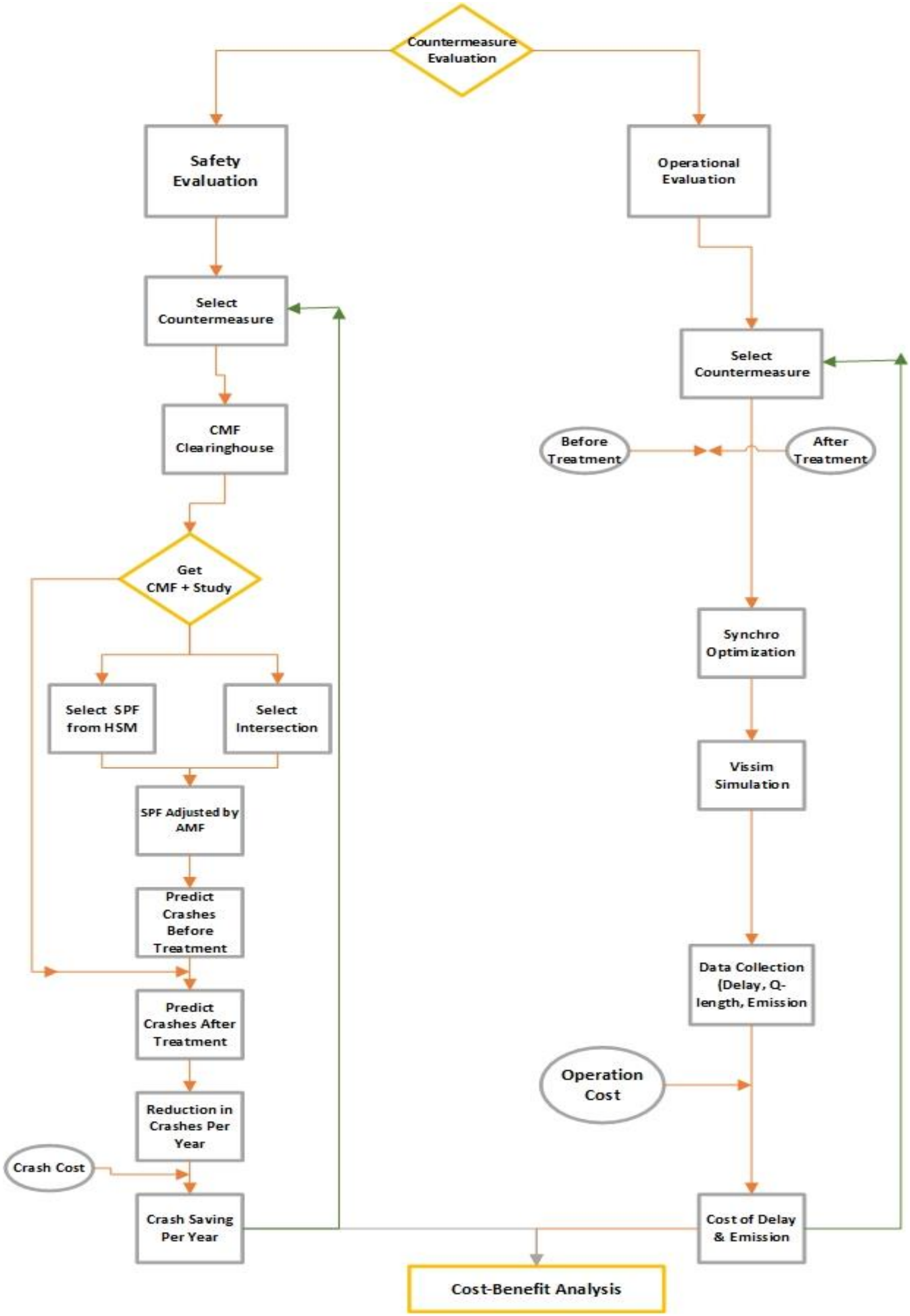


Figure 1: Countermeasure's Evaluation Procedure

CHAPTER 4

ANALYSIS EFFECTIVNESS OF COUNTERMEASURES (CASE STUDIES)

Countermeasure 1: Minor Approach Left-Turn Phase-Change from Permitted-to-Protected

This section describes the approach used to complete and evaluate, including safety evaluation and operational evaluation, the effectiveness of a minor approach left-turn phase-change from permitted-to-protected in Twin Cities Metro District in Minnesota State.

Study Site

A set of treatments have been done in Minnesota's Twin Cities Metro District in order to reduce the frequency of crashes, especially angle crashes, at intersections. One of those treatments involved changing the left turn phase of minor approach from permitted-to-protected. Davis and Aul (2007) mentioned in their study that the treatment group was selected according to a crash data file requested from MnDOT. MNTH 13 & Eagle Creek Ave SE intersection in Twin Cities Metro District MN was one of sites recommended for treatment. MNTH 13 and Eagle Creek Ave SE intersection is a four leg signalized intersection. Eagle Creek Ave SE represents the minor approaches of the intersection with an average daily traffic volume of 10,150 veh/day. Eagle Creek Ave SE has two lanes in each direction; both left and right movements have a shared lane with the through movement. MNTH 13 represents the major approaches to the intersection with an average daily traffic volume of 13,500 veh/day. Each approach of MNTH 13 has one lane for the through movement and two exclusive lanes for left and right turn movements (see figure 2). Intersection signal phasing and cycle length are obtained using Synchro software, with consideration being given to before and after treatment phasing.

Figure 2: MNTH 13 & Eagle Creek Ave SE Intersection

Safety Analysis

This section demonstrates the selection of the Safety Performance Functions (SPFs) for the base condition of the treated intersection as conducted in the Highway Safety Manual. Note that the base condition is represented by the selected location, as described in the previous section, with the recent Annual Average Daily Traffic (AADT).

Safety Performance Function (SPF) Selection and Calibration

MNTH 13 & Eagle Creek Ave SE intersection is a four-leg signalized intersection. As mentioned in Davis and Aul's study, the intersection has experienced a number of angle crashes type; the treatment has been implemented due to this vehicle to vehicle collision type. The SPF value will be calculated for multiple-vehicle collision only due to the occurrence of angle crash type at the treated intersection. As long as all crash severities of the targeted intersection have been recorded, all severity levels will be considered. The calculations for total multiple-vehicle collisions are presented below.

Multiple-Vehicle Collisions (N_{bimv})

Once again, Eagle Creek Ave and MNTH 13 intersection is a four leg signalized intersection. The general SPF for multiple-vehicle collisions can be calculated as follow:

$$N_{bimv} = exp\ (a + b * ln\ (AADT_{maj}) + c * ln\ (AADT_{min}))$$

Where:

AADT at the major approaches (MNTH 13) = 13,500 veh/day

AADT minor approaches (Eagle Creek Ave) = 10,150 veh/day

a, b, and c are regression coefficients that can be found in the following table:

Table 4: SPF Coefficients for Multiple-Vehicle Collisions (HSM, 2009)

Intersection Type	a	b	c
3ST	-13.36	1.11	0.41
3SG	-12.13	1.11	0.26
4ST	-8.90	0.82	0.25
4SG	-10.99	1.07	0.23

Because the treated intersection is a 4-leg signalized intersection, the value of a, b, and c can be determined using the table (4) as -10.99, 1.07, and 0.23, respectively. The predicted crash frequency (N_{bimv}) is:

$$N_{bimv} = exp\ (-10.99 + 1.07 * ln\ (13{,}500) + 0.23 * ln\ (10{,}150)) = 3.699\ crash/year$$

Accident Modification Factors (AMF)

In order to estimate the average crash frequency, the selected SPFs for multiple-vehicle collisions should be adjusted for individual geometry design and traffic control features. The higher AMF gives a higher crash frequency, and vice versa. The accident modification factors for the targeted intersection are calculated as follow.

Intersection Left-turn lane AMF (AMF_1)

The absence of left-turn lanes at the intersection approaches is considered as the base condition (HSM, 2009). Exclusive left-turn lanes are only presented on the major approaches (MNTH13), and not on the minor approaches (Eagle Creek). AMF_1 is applied for the multiple-vehicle collisions and single vehicle collisions only. Table (5) shows intersection left-turn lane accident modification factor (AMF_1) at signalized intersections.

Table 5: Accident Modification Factor of Left-Turn Lane (HSM, 2009)

Intersection Type	**Number of approaches with left-turn lanes**			
	One Approach	**Two Approach**	**Three Approach**	**Four Approach**
3SG	0.93	0.86	0.80	-
4SG	0.90	0.81	0.73	0.66

As long as the targeted intersection is a 4-leg signalized intersection with a left-turn lane on the major approach, then the value of AMF_1 is 0.81.

Intersection Left-Turn Signal Phasing AMF (AMF_2)

The absence of protected and permissive/protected or protected/permissive left-turn phasing is considered as the base condition (HSM, 2009). Prior to the treatment, the intersection had exclusive left-turn phasing on the major approaches, but not on the minor approaches. AMF_2 is applied for multiple-vehicle collisions and single vehicle collisions only. Table (6) shows the accident modification factor for left-turn lanes (AMF_2) at signalized intersections.

Table 6: Accident Modification Factor of Left-Turn phasing (HSM, 2009)

Type of left-turn signal phasing	AMF_2
permissive/protected or protected/permissive	0.99
Protected	0.94

As long as the targeted intersection is a 4-leg signalized intersection with protected left-turn signal phasing on the two major approaches, then the value of AMF_2 is 0.94.

Intersection Right-Turn Lane AMF (AMF_3)

The absence of right-turn lanes on the intersection approaches is considered as the base condition (HSM, 2009). The treated intersection has an exclusive right-turn lane on the major approaches, but not on the minor approaches. AMF_3 is applied for multiple-vehicle collisions and single vehicle collisions only. Table (7) shows intersection right-turn lane accident modification factor (AMF_3) at signalized intersections.

Table 7: Accident Modification Factor of Right-Turn Lane (HSM, 2009)

Intersection Type	Number of approaches with right-turn lanes			
	One Approach	Two Approach	Three Approach	Four Approach
3SG	0.96	0.92	-	-
4SG	0.96	0.92	0.88	0.85

As long as the targeted intersection is a 4-leg signalized intersection with a right-turn lane on the two major approaches, then the value of AMF_3 is 0.92.

Intersection Right-Turn on Red AMF (AMF_4)

Permitted right-turn on red, for all approaches, is considered as the base condition (HSM, 2009). The targeted intersection (Eagle Creek Ave & MNTH 13 Intersection) has no signs prohibiting right-turn on red. Therefore, the value of AMF_4 is 1. AMF_4 is applied to multiple-vehicle collisions and single vehicle collisions only at signalized intersections.

Lighting AMF (AMF_5)

The absence of intersection lighting is considered as the base condition (HSM, 2009). As long as the treated intersection has lighting, the formula below is applied to find AMF_5.

$$AMF_5 = 1 - 0.38 \times P_n$$

Where, P_n is the proportion of total crashes that occurred at night, at unlighted intersections. According to HSM, the P_n for a signalized intersection is 0.235. Therefore, AMF_5 can be calculated as follow:

$$AMF_5 = 1 - 0.38 * 0.235 = 0.91$$

Red Light Cameras AMF (AMF_6)

Red light cameras are installed for enforcement of red signal violation at signalized intersections (HSM, 2009). There are no red light cameras observed at the treated intersection. Therefore, the value of AMF_6 is equal to 1.

Predicted Average Crash Frequency before Implementing the Countermeasure

The predicted crash frequency at the MNTH 13 & Eagle Creek Ave SE intersection for multiple-vehicle collisions (N_{bimv}) after calibration is N_b. N_b can be calculated as follows:

$$N_b = N_{bimv} (AMF_1 * AMF_2 * AMF_3 * AMF_4 * AMF_5 * AMF_6)$$

$$N_b = 3.699 (0.81 * 0.94 * 0.92 * 1 * 0.91 * 1) = 2.358 \text{ crash/year}$$

The total average crash frequency of the treated intersection can be found as follows:

$$N_{predicted\ int.} = C_i (N_b)$$

$$N_{predicted\ int.} = 1 (2.358) = 2.358 \text{ crash/year}$$

Next, in order to calculate the predicted crash frequency, by type of collision, the HSM constructs a table of proportions to separate multiple-vehicle collisions into collision types. Angle crash type has been observed at the targeted intersection with all levels of crash severity. Therefore the proportion of angle crash type is 0.591, and the predicted crash frequency for the intersection is:

$$N_{predicted\ int.} = 2.358 * 0.591 = 1.393 \text{ crash/year}$$

Predicted Average Crash Frequency after Implementing the Countermeasure

The countermeasure presented involves changing the left-turn phase on the minor approaches from permitted-to-protected. The Crash Modification Factor Clearinghouse provides a CMF of 0.01 for permitted-to-protected change. This means that the crash frequency after implementing the countermeasure is expected to be 0.014 crashes per year; or in other words a reduction of 1.379 crashes per year can be seen.

Operational Performance Analysis

This section applies the methodology used for operational performance analysis of changing left-turn phase from permitted-to-protected on minor approaches. Field data from the Davis and Aul study and Google earth is used for the analyses of signal timing. Data is collected from VISSIM simulation for the two before and after treatment periods for operational measures, including approach delay, queue length, and vehicular emission within that area. The computations of these measures are discussed next.

Synchro Simulation and Results

Optimized signal timing for the Eagle Creek Ave & MNTH 13 intersection was obtained using Synchro. The input data required for this included intersection geometry and traffic volume for each approach. Prior to the treatment the major approaches have protected left-turn phasing, while the minor approaches have permitted left turn. After treatment left-turn phasing on the minor approaches were changed from permitted to protected phase.

Synchro results show that the signal has a 60 second cycle length and three phases for the before period. MNTH 13 left turn is represented by phase 1 signal group number 1 and 5, MNTH 13 through movement is represented by phase 2 signal group number 2 and 6, and phase 3 signal group 4 and 8 which includes Eagle Creek Ave through and turning movements (see figure 4a).

After changing the left-turn phase from permitted-to-protected on the minor approach (after treatment), the signal would have four phases with a 90 second cycle length. In this case the east movement of Eagle Creek Ave, including turning movements, will be represented by phase 3 signal group number 4, and west movement, including turning movements, will be represented by phase 4 signal group number 8 (see figure 4b).

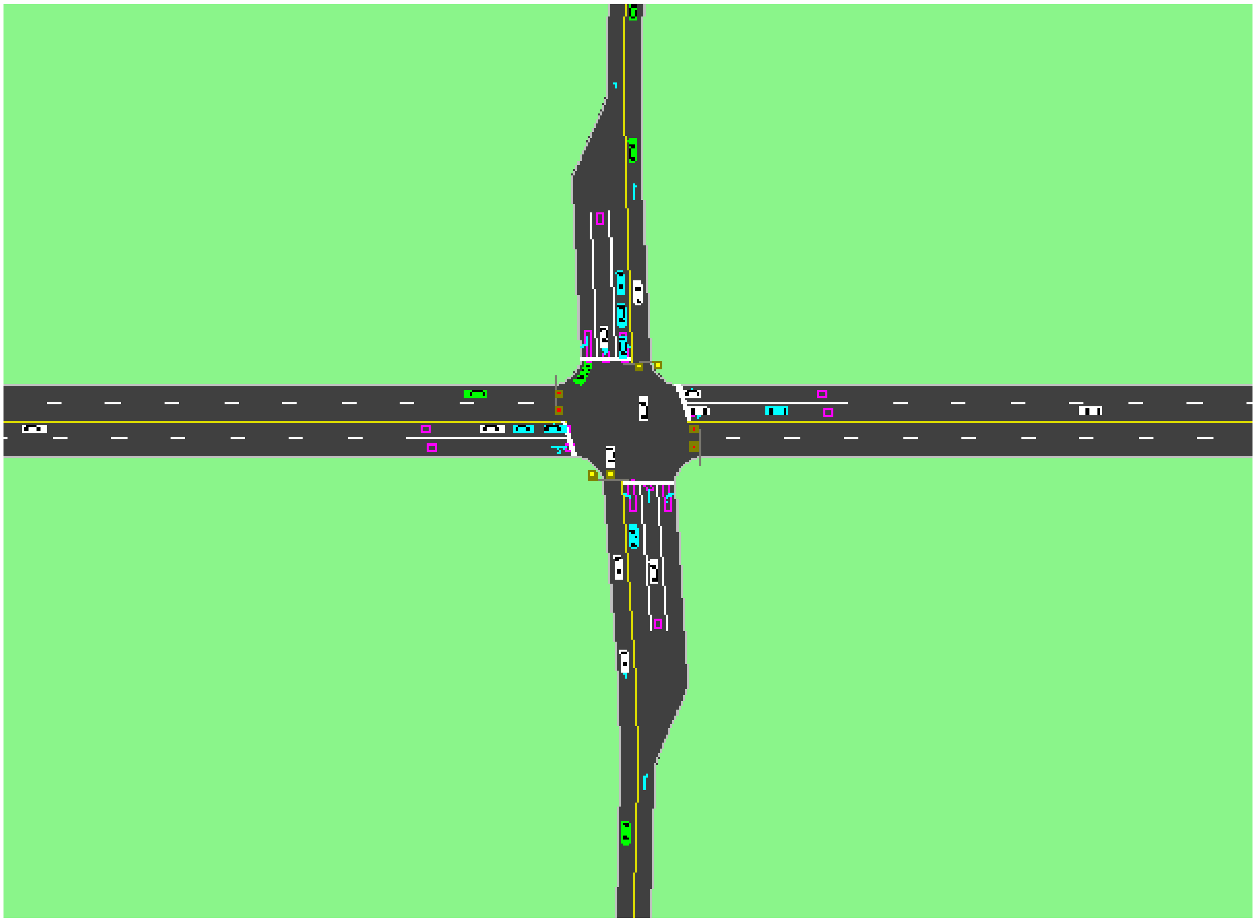
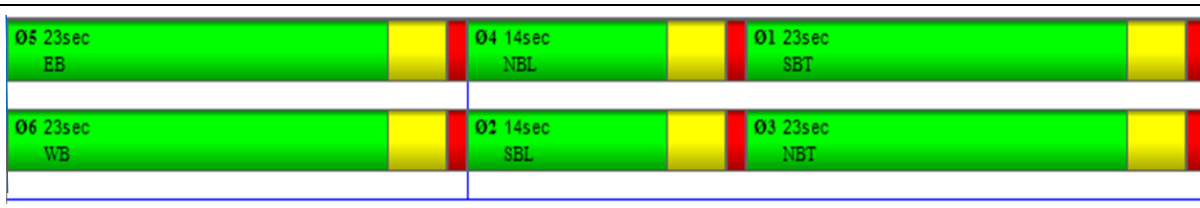
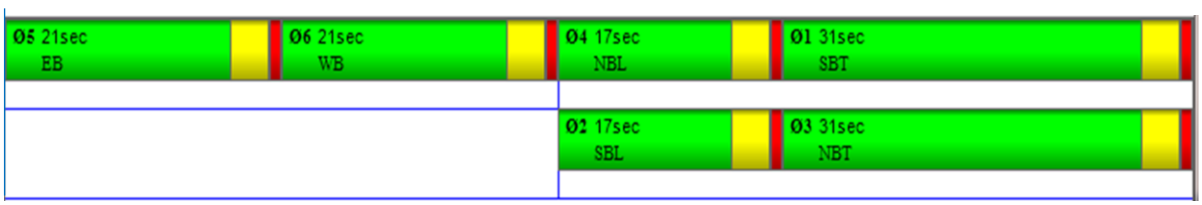


Figure 3: Synchro Simulation of Eagle Creek Ave & MNTH 13 Intersection



a. Signal Timing of the before treatment period



b. Signal Timing of the after treatment period

Figure 4: Signal Timing and Cycle Length of Eagle Creek Ave & MNTH 13 Intersection

Results and Discussion

The intent of this section is to cover the details of the findings for the first countermeasure. This thesis analyzes the impacts of changing left-turn phase from permitted-to-protected on safety and operation and provides in which condition this countermeasure is cost-effective. Different locations were simulated by varying the AADT, base values and geometries were taken from case studies developed by the Crash Modification Factor Clearinghouse. Table (8 and 9) below show the results of both safety and operational analysis per intersection for the before and after treatment periods, respectively. The safety measures consisted of the number of angle crashes per year before and after treatment. The operational measures consisted of change in traffic delay, queue length, vehicular emission and fuel consumption for the periods before and after treatment.

Table 8: Results of Before Treatment Period for the Treated Intersection

Before Treatment							
Scenarios	**Predicted Crashes (Crash/year)**	**Total Number of Vehicle**	**Delay (sec/veh)**	**Q-Length (ft.)**	**CO Emission (g)**	**NOx Emission (g)**	**Fuel Consumption (gal)**
70%*AADT	0.876	1820	16	190	2579	502	36.89
80%*AADT	1.043	2075	17	210	3050	593	43.64
90%*AADT	1.215	2366	18	203	3564	693	50.99
100%*AADT	1.393	2648	20	220	4127	803	59.04
110%*AADT	1.577	2883	23	400	4961	965	70.98
120%*AADT	1.766	3133	26	534	5899	1148	84.39
130%*AADT	1.960	3381	36	1322	7628	1484	109.13

Table 9: Results of After Treatment Period for the Treated Intersection

After Treatment						
Scenarios	**Predicted Crashes (Crash/year)**	**Delay (sec/veh)**	**Maximum Q-Length (ft.)**	**CO Emission (g)**	**NOx Emission (g)**	**Fuel Consumption (gal)**
70%*AADT	0.009	23.84	206.5	3052.273	593.861	43.666
80%*AADT	0.010	26.03	271.66	3699.246	719.739	52.922
90%*AADT	0.012	27.98	463.44	4376.581	851.524	62.612
100%*AADT	0.014	30.32	512.63	5141.856	1000.418	73.56
110%*AADT	0.016	33.47	560.53	5893.382	1146.638	84.312
120%*AADT	0.018	50.04	1322.19	7842.368	1525.84	112.194
130%*AADT	0.020	76.78	1340.23	10748.919	2091.349	153.776

A Crash Modification Factor (CMF), for left turn changing phase from permitted-to-protected, was developed to determine the reduction in multiple-vehicle collisions. A CMF value of 0.01 was determined, indicating that after implementing the countermeasure a 99 percent reduction in crashes of all severity would be seen. While this decrease in crashes occurs, an operation analysis reviled that a negative influence on traffic delay, CO and NOx emissions, as well as fuel consumption would occur. A total of seven scenarios were considered with varying average daily traffic volumes (AADT) in order to demonstrate the impact of implementing the countermeasure on safety and operation. Figures (5) through (11) show the before and after treatment crash and operation results.

The results of the first scenario, which consider the actual AADT (case study), are shown in figure (5). As can be seen, the implementation of protected left-turn on minor approaches reduced the crash frequency by 1.375 crashes per year. However, a 10 second increase in the average traffic delay per vehicle occurs as a result of the treatment. Additionally, a significant increase in CO and NOx emission and fuel consumption can be seen. The after treatment increases in CO and NOx emission and fuel consumption are 1015 grams per hour, 198 grams per hour, and 14.5 gallons per hour, respectively.

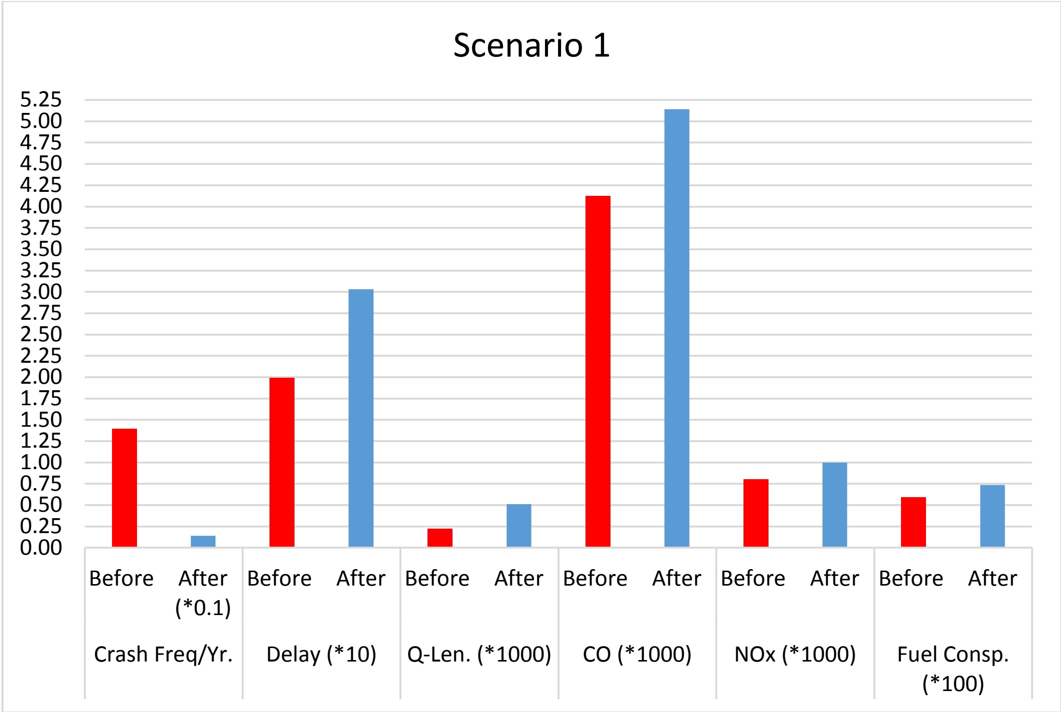


Figure 5: Before and After Differences at Eagle Creek Ave & MNTH 13 Intersection with Actual AADT

The results of the second and third scenarios, were a reduction of 10 and 20 percent of actual AADT was considered, is shown in figures (6) and (7). As was expected, a decrease in traffic volume resulted in a lower crash frequency reduction. A reduction of 1.2 and 1.03 crashes per year was seen as a result of treating the intersection, considering the second and third scenarios, respectively. Simultaneously, the 10 and 9 second increase in overall average vehicle delay, as experienced by the first scenario, affected both the second and third scenarios, respectively. While the same increase in delay was experienced, slightly lower increases in emissions and fuel consumption were seen. The increases in emissions were 813 and 649 grams per hour of CO and 158 and 126 grams per hour of NOx, for the second and third scenarios respectively.

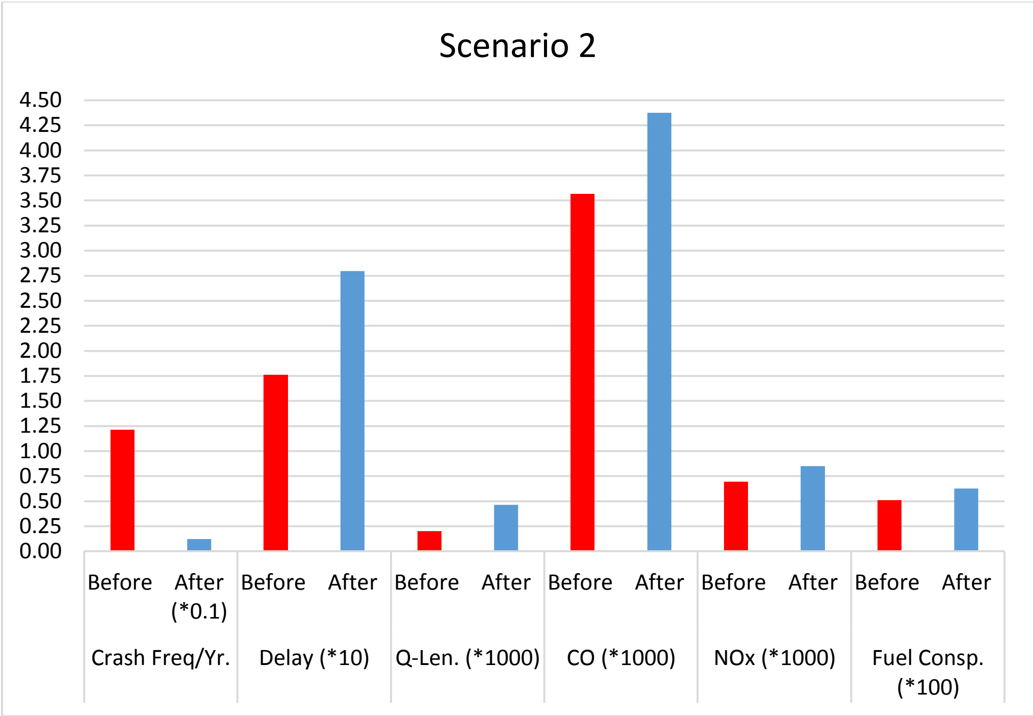


Figure 6: Before and After Differences at Eagle Creek Ave & MNTH 13 Intersection with 90% AADT

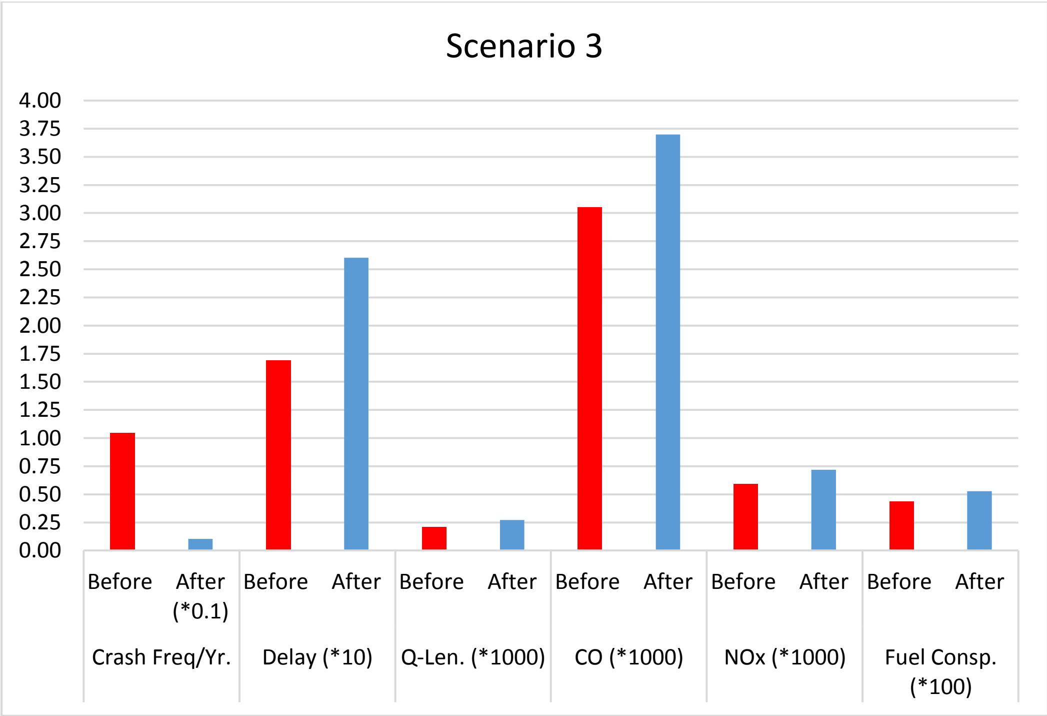


Figure 7: Before and After Differences at Eagle Creek Ave & MNTH 13 Intersection with 80% AADT

The results of scenario 4, which considers a reduction in AADT by 30 percent, are shown in figure (8). As can be seen, implementing a protected left-turn on minor approaches at the selected intersection, with AADT reduction, will reduce crash frequency by 0.868 crashes per year. Once again the average traffic delay per vehicle will be increased by 8 seconds; however, the maximum queue length will remain the same. Additionally treatment will have little to no effect on the NOx emission and fuel consumption. A slight increase of 474 grams per hour for CO emissions will be seen after implementing the counter measure.

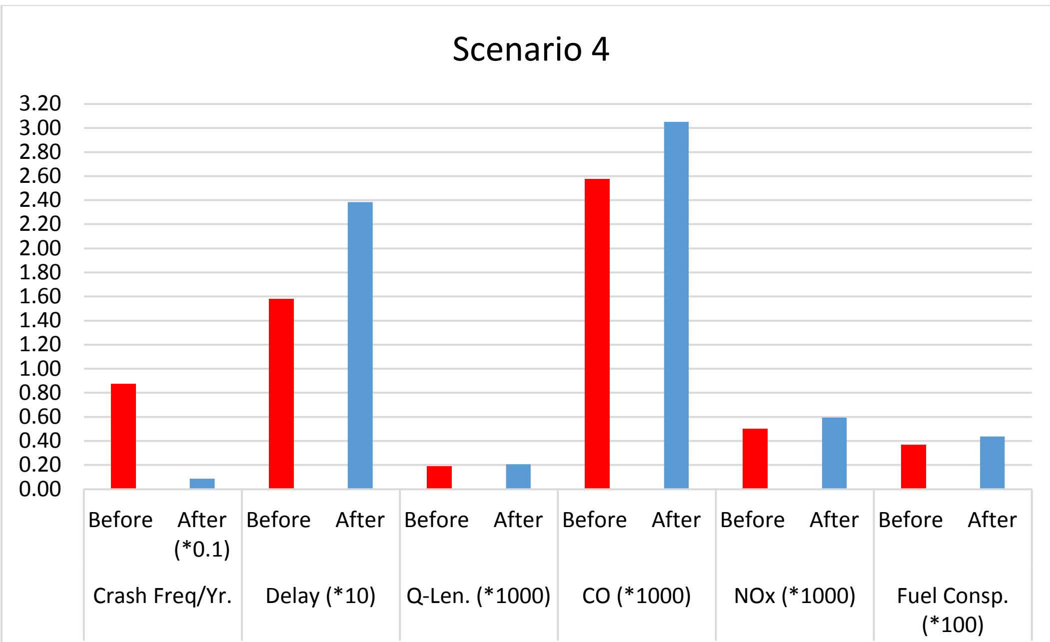


Figure 8: Before and After Differences at Eagle Creek Ave & MNTH 13 Intersection with 70% AADT

Figure (9), on the other hand, shows the results of the scenario that considered a 10 percent increase in AADT. This uptick in traffic resulted in an after treatment increase of vehicle delay by 11 seconds. Additionally, vehicular emission and fuel consumption both drastically increased after treatment. CO emission increased by 932 grams per hour, NOx emissions increased by 181 grams per hour, and fuel consumption increased by 13.3 gallons per hour. While an increase in delay and emissions was seen, a simultaneous reduction in total crash frequency occurred, resulting in 1.56 crashes per year.

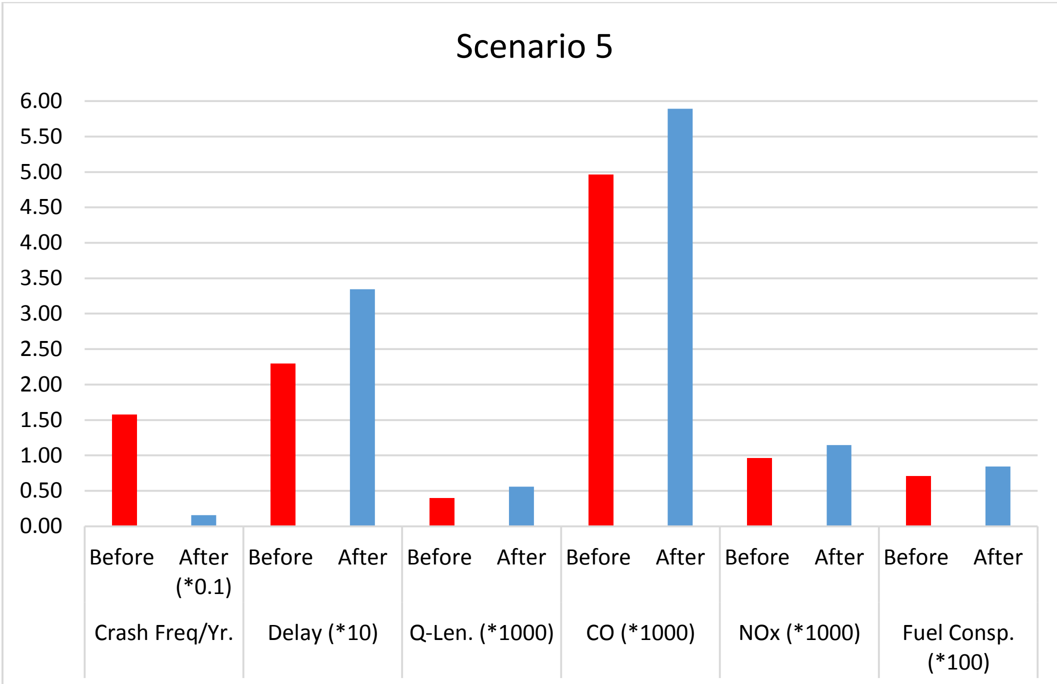


Figure 9: Before and After Differences at Eagle Creek Ave & MNTH 13 Intersection with 110% AADT

As evident in figures (10) and (11), representing scenarios 6 and 7 were a traffic volume increase of 20 and 30 present occurred, a drastic increase in traffic delay will occur when traffic volume in increase beyond 120% actual AADT. It can be observed that scenario 6 increased after treatment delay by 24 second and scenario 7 by 41 seconds. This means implementing the countermeasure at sites with traffic volume higher than 110% actual AADT can cause high traffic congestion and vehicular emission. Moreover, the comparison between before and after treatment periods showed an increase in CO emission by 1943 and 3121 grams per hour, NOx emission increases of 378 and 607 grams per hour, and fuel consumption by 27.8 and 44.6 gallons per hour for scenarios 6 and 7, respectively. Both scenarios showed an increased crash reduction over the other scenarios of 1.75 and 1.94 crashes per year.

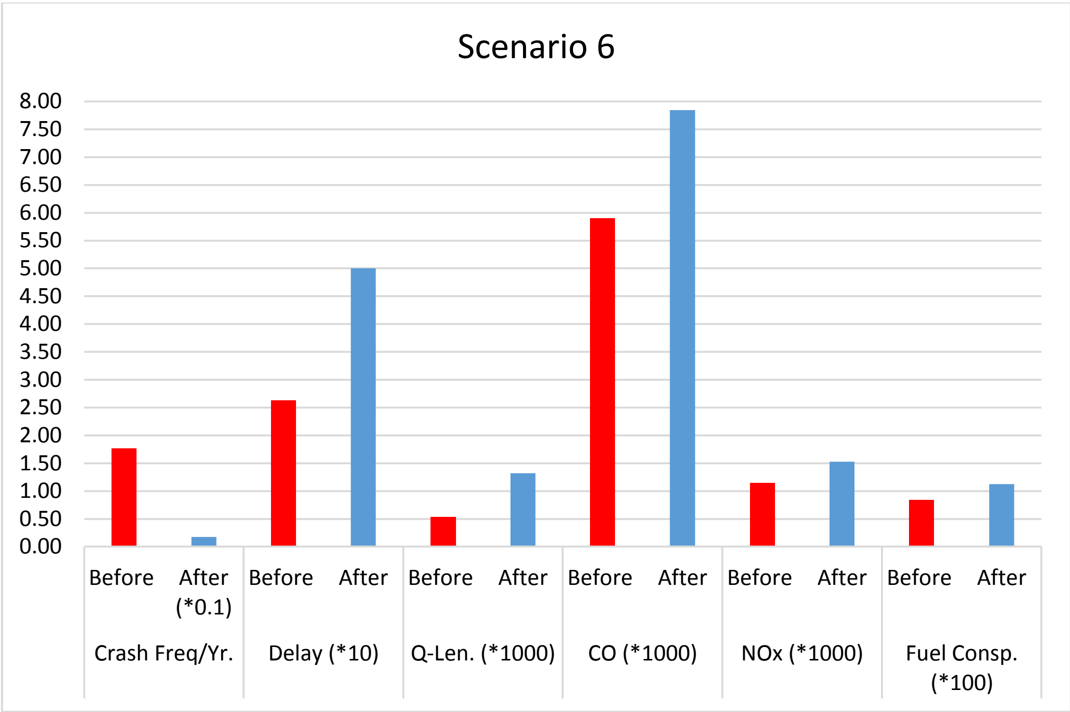


Figure 10: Before and After Differences at Eagle Creek Ave & MNTH 13 Intersection with 120% AADT

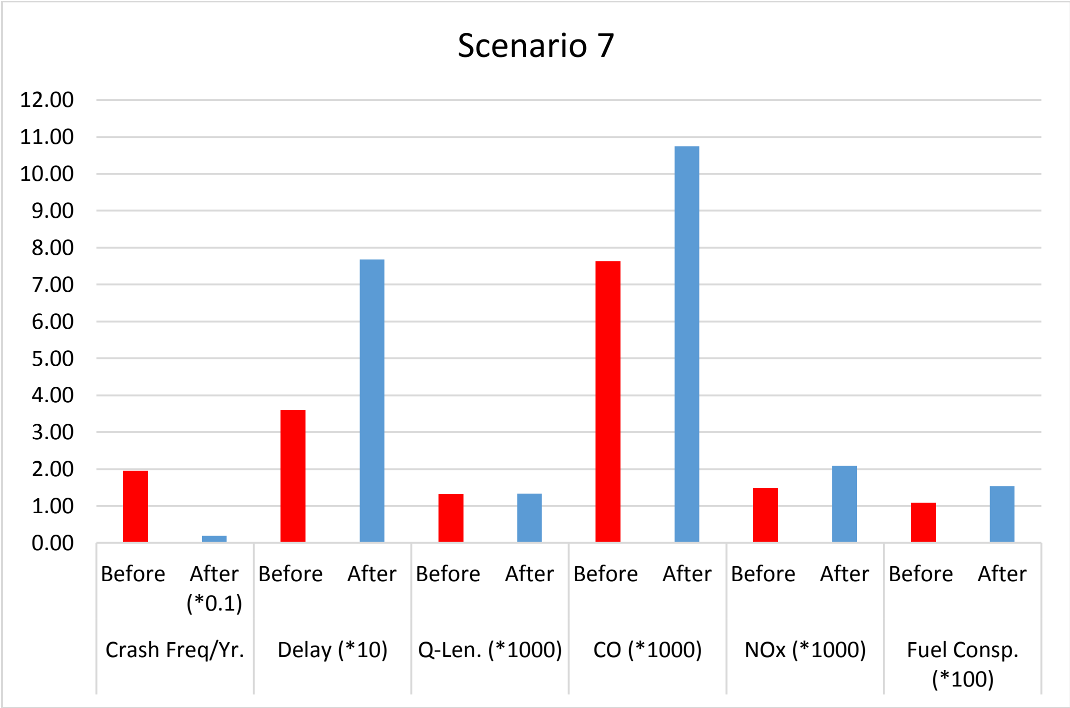


Figure 11: Before and After Differences at Eagle Creek Ave & MNTH 13 Intersection with 130% AADT

Cost-Benefit Analysis

Understanding the operational costs and safety benefits of a countermeasure is an important aspect to consider before implementing any changes. The intent of the following section is to discuss the economic analysis of changing left-turn phase on minor approach from permitted-to-protected. Chapter 3 of this thesis described the methodology and approaches used to determine crash and operational costs for the cost-benefit calculations. Cost-benefit analysis at varying traffic volumes and number of crashes observed will help States and local agencies determining if implementing the left-turn phase-change from permitted-to-protected will be economically beneficial or not.

Operation data were analyzed for seven different traffic volume scenarios. Data was collected, considering both before and after implementing the countermeasure, in order to determine the impact of the treatment on operational performance. In terms of safety, The CMF developed for this countermeasure was used to calculate the amount of savings resulting from a reduction in crashes. By applying factors to the base number of crashes, the costs and benefits were observed for varying amounts of crashes, which were crash factors multiplied by the base number of crashes. Operational cost and crash saving due to the implementation of left-turn changing phase for the seven scenarios were combined in order to show the cost and benefit trends after the treatment.

Figures (12) and (13) show the comparison between crash saving, delay cost, and overall operational cost, considering an increase in the number of observed crashes. Minor approaches left-turn changing phase implemented at the intersection of MNTH 13 & Eagle Creek Ave. reduced crash frequency by 1.38 crashes per year at the base condition (actual AADT). This resulted in a saving of $27,588, per year. The total delay cost at the intersection, however, was

$226,932 per year and $228,097per year for operation factors, including delay, and CO & NOx emissions.

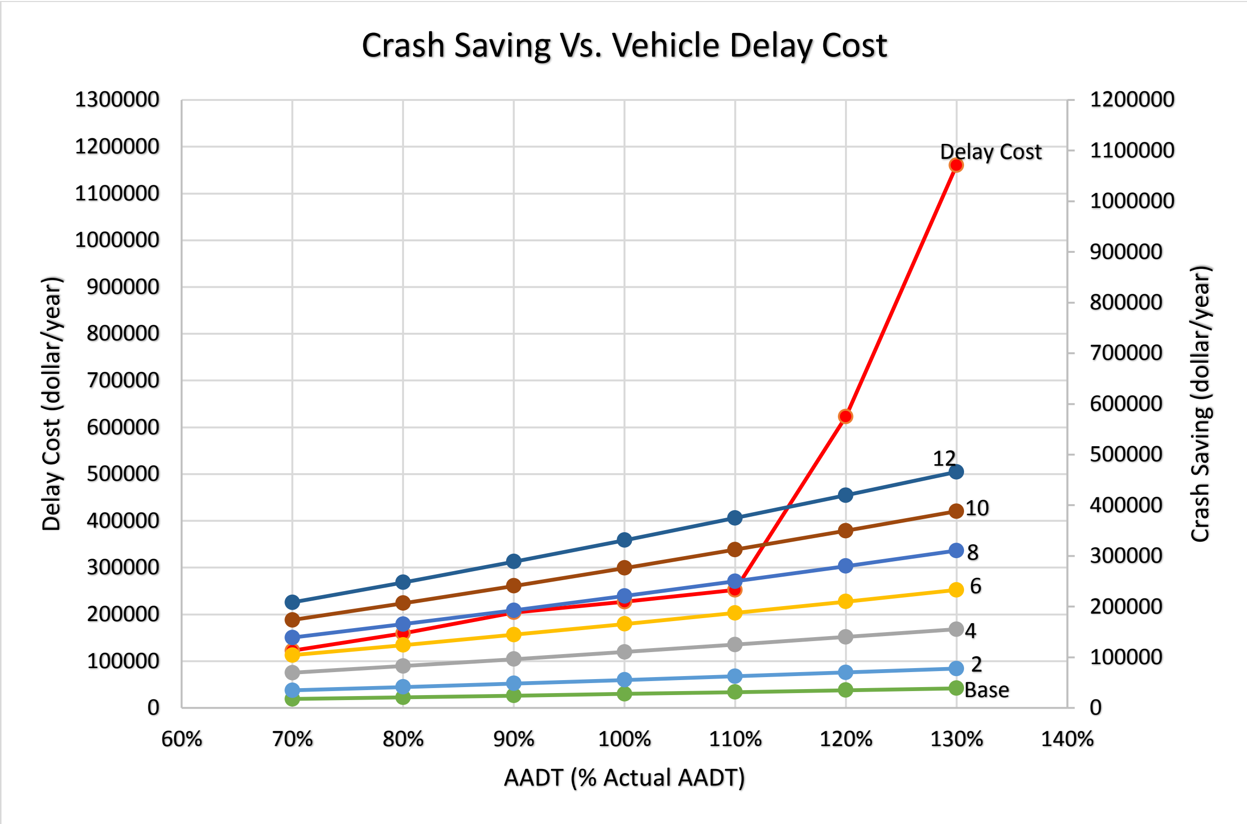


Figure 12: Crash Saving and Delay Cost Comparison for Eagle Creek Ave & MNTH 13 Intersection

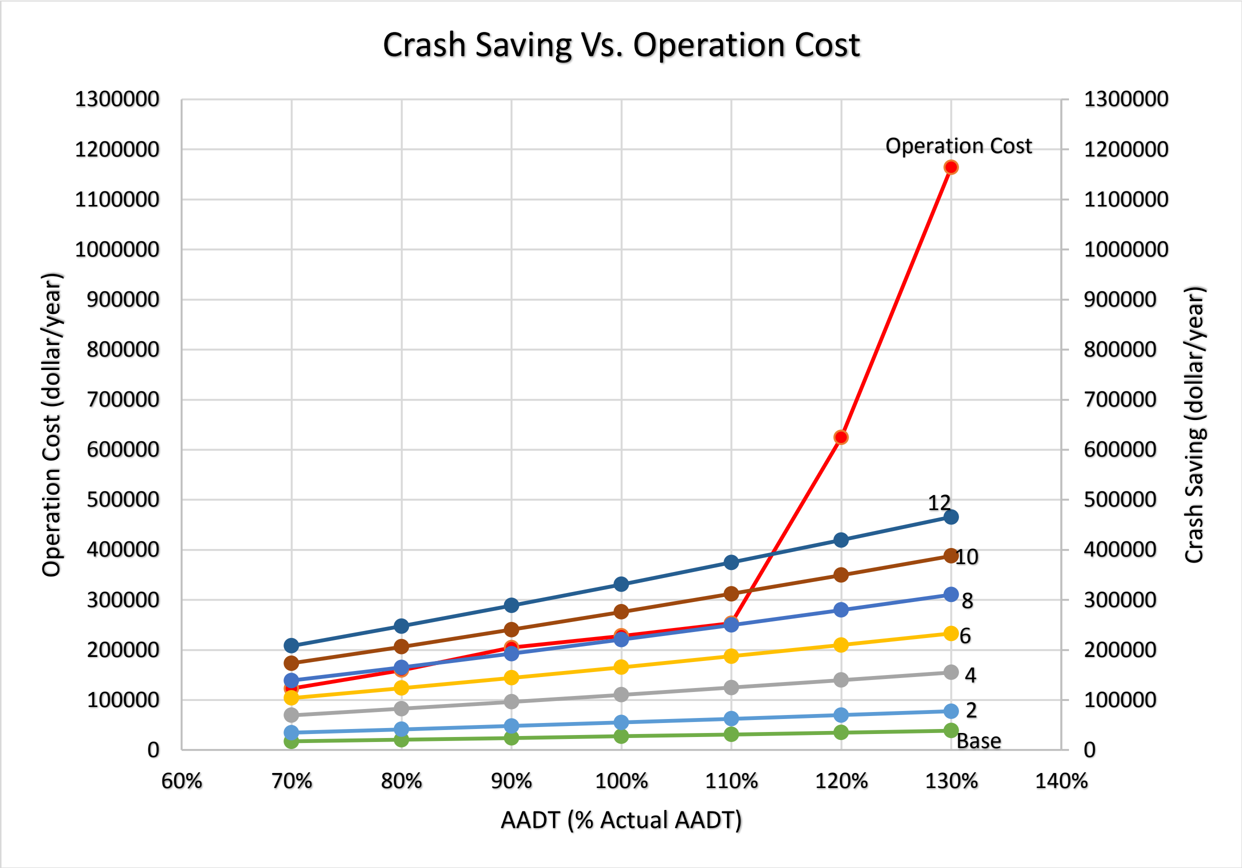


Figure 13: Crash Saving and Operational Cost Comparison for Eagle Creek Ave & MNTH 13 Intersection

The treatment, therefore, resulted in a cost of approximately $200,508 per year at the treated intersection within average crash observation (Base) due to the treatment. Additionally, the other scenarios considered showed the same cost-benefit trend. A decrease in AADT by 30 or 20 percent could lead to a significant benefit if the treated intersection experienced more than 7 crashes per year. An increase in AADT by 30 percent led to a significant increase in cost of $1,125,252 per year. The highest benefit ($121,390 per year) can be obtained when the treatment is implemented at intersections having 110 % of actual AADT with number of observed crashes 18.93 crash per year. In summary, the results show that the cost of implementing left-turn changing phase from protected-to-permitted is higher than the benefit at the base condition. Table (10) summarizes cost-benefit analysis for all scenarios at different crash observations.

Table 10: Safety Benefit (saving) and Operational Cost of all Scenarios for Eagle Creek Ave & MNTH 13 Intersection

Scenarios		Crash Factor						
		Base (1)	2	4	6	8	10	12
70% * AADT	No. of Crashes	0.88	1.75	3.51	5.26	7.01	8.76	10.52
	Benefit/Cost	-105538	-88186	-53482	-18778	15926	50629	85333
80% * AADT	Crashes	1.04	2.09	4.17	6.26	8.34	10.43	12.51
	Benefit/Cost	-139130	-118489	-77206	-35924	5359	46642	87925
90% * AADT	No. of Crashes	1.22	2.43	4.86	7.29	9.72	12.15	14.58
	Benefit/Cost	-180722	-156665	-108551	-60438	-12325	35789	83902
100% * AADT	No. of Crashes	1.39	2.79	5.57	8.36	11.15	13.93	16.72
	Benefit/Cost	-200508	-172920	-117744	-62568	-7392	47784	102960
110% * AADT	No. of Crashes	1.58	3.15	6.31	9.46	12.62	15.77	18.93
	Benefit/Cost	-222108	-190880	-128426	-65972	-3518	58936	121390
120% * AADT	No. of Crashes	1.77	3.53	7.06	10.60	14.13	17.66	21.19
	Benefit/Cost	-589929	-554962	-485029	-415095	-345161	-275227	-205294
130% * AADT	No. of Crashes	1.96	3.92	7.84	11.76	15.68	19.60	23.52
	Benefit/Cost	-1125252	-1086451	-1008848	-931245	-853642	-776040	-698437

Countermeasure 2: Leading Pedestrian Interval (LPI)

This section describes the approach used to complete and evaluate, including safety evaluation and operational evaluation, the effectiveness of implementing Leading Pedestrian Interval (LPI) in down town State College in Pennsylvania.

Study Site

During 2005, ten signalized intersections in down town State College in Pennsylvania were treated with Leading Pedestrian Intervals (LPI). The treatment sites are located along two urban principal arterial highways (State Route 26 and College and Beaver Avenues), which form a one-way couplet in the central business district. Each arterial street has two through lanes, with the average daily traffic values being approximately 13,500 and 12,000 for College and Beaver Avenues, respectively. All major and minor approaches at the treatment sites have speed limits of 25 mph. Pedestrians crossing the major street would cross two travel lanes. Due to the close proximity of the Pennsylvania State University, downtown businesses, apartments, and offices, treated intersections experienced a range of pedestrian volume between 100 to 1,000 pedestrians per hour during the peak periods. The 10 treatment sites are signalized intersections with pedestrian walk–don't walk signal heads. The length of the LPI at each treated site was 3 seconds. Countdown pedestrian signals were added to two of the 10 treated sites at approximately the same time as the LPIs (Fayish and Gross, 2010).

At this stage, one intersection out of the ten treated sites will be evaluated for safety and operation performance after the treatment. The selected intersection is E. College Ave. (one way street) & Shortlidge Rd/S Garner St. E Collage Ave. has two travel lanes with on-street parking on the two sides, while the Shortlidge Rd/S Garner St have one lane in each direction, with exclusive left turn lane at S Garner Ave. The intersection is signalized for both vehicles and pedestrians. The phasing and cycle length of the intersection are obtained using Synchro software, with consideration being given to before and after treatment phasing.

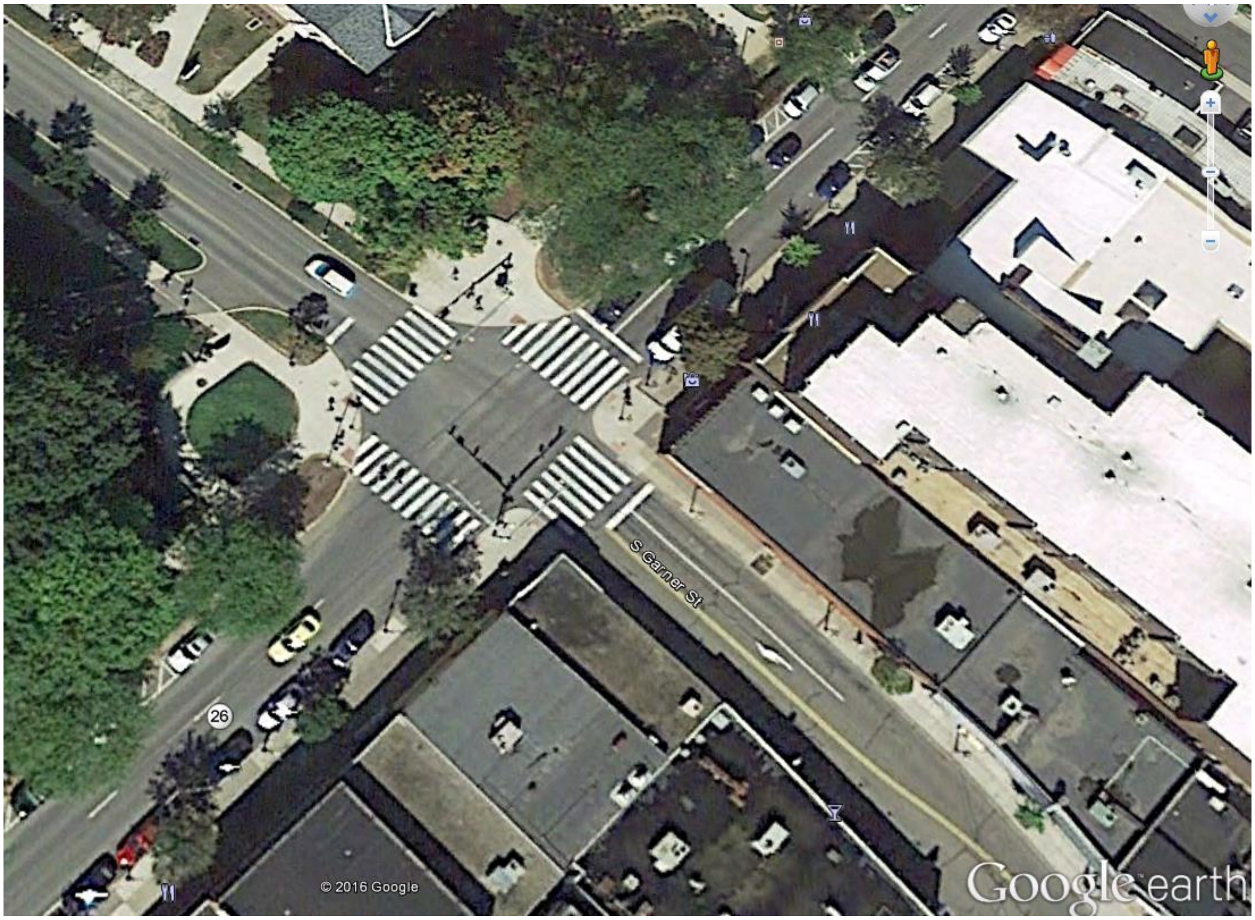


Figure 14: E. College Ave. & Shortlidge Rd/S. Garner St. Intersection

Safety Analysis

This section demonstrates the selection of the Safety Performance Functions (SPFs) for the base condition of the treated intersection as conducted in the Highway Safety Manual. Note that the base condition is represented by the selected location, as described in the previous section, with the recent Annual Average Daily Traffic (AADT).

Safety Performance Function (SPF) Selection and Calibration

E. College Ave. & Shortlidge Rd/S. Garner St. intersection is a four-leg signalized intersection. As mentioned in Fayish and Gross's study, the intersection experienced vehicle-pedestrian and vehicle-bicycle collision type; the treatment has been implemented due to this crash type. The SPF value will be calculated for vehicle-pedestrian and vehicle-bicycle collisions only at the treated intersection. As long as all crash severities of the targeted case study intersection have been recorded, all severity levels will be considered.

Multiple-Vehicle Collisions

Again, E. College Ave & Shortlidge Rd./S. Garner St. intersection is a four-leg signalized intersection. The general SPF for multiple-vehicle collisions can be calculated as follow:

$$N_{bimv} = exp\ (a + b * ln\ (AADT_{maj}) + c * ln\ (AADT_{min}))$$

Where:

AADT on major approaches = 13,500 veh/day

AADT on minor approaches = 12,000 veh/day

a, b, c are regression coefficients that can be found in the following table:

Table 11: SPF Coefficients for Multiple-Vehicle Collisions (HSM, 2009)

Intersection Type	**a**	**b**	**c**
3ST	-13.36	1.11	0.41
3SG	-12.13	1.11	0.26
4ST	-8.90	0.82	0.25
4SG	-10.99	1.07	0.23

Because the treated intersection is a 4-leg signalized intersection, the value of a, b, c can be determined using table (11) as -10.99, 1.07, 0.23, respectively. The predicted crash frequency (N_{bimv}) is:

$$N_{bimv} = exp\ (-10.99 + 1.07 * ln\ (13500) + 0.23 * ln\ (12000)) = 3.844\ crash/year$$

Single-Vehicle Collisions

The general SPF of for single-vehicle collisions can be calculated as follow:

$$N_{bisv} = exp\ (a + b * ln\ (AADT_{maj}) + c * ln\ (AADT_{min}))$$

Where:

AADT on major approaches = 13,500 veh/day

AADT on minor approaches = 12,000 veh/day

a, b, and c are regression coefficients that can be found in the following table:

Table 12: SPF Coefficients for Single-Vehicle Collisions (HSM, 2009)

Intersection Type	a	b	c
3ST	-6.81	0.16	0.51
3SG	-9.02	0.42	0.40
4ST	-5.33	0.33	0.12
4SG	-10.21	0.68	0.27

Because the treated intersection is a 4-leg signalized intersection, the value of a, b, and c can be determined using the table (12) as -10.21, 0.68, 0.27, respectively. Then the number of predicted crash frequency (N_{bisv}) is:

$$N_{bisv} = exp\ (-10.21 + 0.68 * ln\ (13500) + 0.27 * ln\ (12500)) = 0.299\ crash/year$$

Accident Modification Factors (AMF)

In order to estimate the average crash frequency, the selected SPFs for multiple-vehicle collisions, single-vehicle collisions, and vehicle-pedestrian collisions should be adjusted for individual geometry design and traffic control features. The higher AMF gives a higher crash frequency, and vice versa. The accident modification factors for the targeted intersection are calculated as follow.

Intersection Left-turn lane AMF (AMF_1)

The absence of left-turn lanes on the intersection approaches is considered as the base condition (HSM, 2009). Exclusive left-turn lane is only presented on one minor approach (S Grand St.). AMF_1 is applied for the multiple-vehicle collisions and single vehicle collisions only. Table (13) shows intersection left-turn lane accident modification factor (AMF_1) at signalized intersections.

Table 13: Accident Modification Factor of Left-Turn Lanes (HSM, 2009)

Intersection Type	**Number of approaches with left-turn lanes**			
	One Approach	**Two Approach**	**Three Approach**	**Four Approach**
3SG	0.93	0.86	0.80	-
4SG	0.90	0.81	0.73	0.66

As long as the targeted intersection is a 4-leg signalized intersection with a left-turn lane on the major approach, then the value of AMF1 is 0.90.

Intersection Left-Turn Signal Phasing AMF (AMF_2)

The absence of protected and permissive/protected or protected/permissive left-turn phasing is considered as the base condition (HSM, 2009). Prior to the treatment, the intersection had permitted left-turn phasing on all approaches. Therefore, AMF_2 is equal to 1. AMF_2 is applied for multiple-vehicle collisions and single vehicle collisions only.

Intersection Right-turn lane AMF (AMF_3)

The absence of the right-turn lanes on the intersection approaches is considered as the base condition (HSM, 2009). No exclusive right-turn lanes are presented for the targeted intersection. Therefore, the value of AMF_3 is equal to 1.

Intersection Right-Turn on Red AMF (AMF_4)

Permitted right-turn on red, for all approaches, is considered as the base condition (HSM, 2009). The “No Turn on Red” signs are presented for two approaches at the targeted intersection. Therefore, the value of AMF_4 is calculated as follow:

$$AMF_4 = 0.98^n$$

Where, n is the number of signalized intersection approaches for which right turn on red is prohibited

$$AMF_4 = 0.98^2 = 0.9604$$

AMF_4 is applied to multiple-vehicle collisions and single vehicle collisions only at signalized intersections.

Lighting AMF (AMF_5)

The absence of intersection lighting is considered as the base condition (HSM, 2009). As long as the treated intersection has lighting, the formula below is applied to find AMF_5.

$$AMF_5 = 1 - 0.38 \times P_n$$

Where, P_n is the proportion of total crashes that occurred at night, at unlighted intersections. According to HSM, the P_n for a signalized intersection is 0.235. Therefore, AMF_5 can be calculated as follow:

$$AMF_5 = 1 - 0.38 * 0.235 = 0.9107$$

Red Light Cameras AMF (AMF_6)

Red light cameras are installed for enforcement of red signal violation at signalized intersections (HSM, 2009). There are no red light cameras observed at treated intersection. Therefore, the value of AMF_6 is equal to 1.

Bus Stop AMF (AMF_7)

The absence of bus stops near the intersection (no bus stop within 1000 ft. of the center of the intersection) is the base condition (HSM, 2009). Three bus stops were presented within 1000 ft. of the treated intersection. Therefore, the value of AMF_1 is equal to 4.15 (see table 14).AMF_7 is applied for the total vehicle-pedestrian collision only.

Table 14: Accident Modification Factor for Number of Bus Stops (HSM, 2009)

Number of bus stops within 1,000 ft of the intersection	AMF_7
1 or 2	2.78
3	4.15

School AMF (AMF_8)

The absence of schools near the intersection (no school within 1000 ft. of the center of the intersection) is the base condition (HSM, 2009). Due to the present of school within the intersection zone, the value of AMF_8 is equal to 1.35 (see table 15).AMF_8 is applied for the total vehicle-pedestrian collision only.

Table 15: Accident Modification Factor for Number of Schools (HSM, 2009)

Presence of schools within 1,000 ft of the intersection	AMF_8
No School present	1
School present	1.35

Alcohol Sales Establishments (AMF_9)

The absence of alcohol sales establishments near the intersection (no alcohol sales establishments within 1000 ft. of the center of the intersection) is the base condition (HSM, 2009). No alcohol sales establishments are presented within 1000 ft. of the treated intersection; therefore, the value of AMF_9 in this case is 1.

Vehicle-Pedestrian Collisions

The general SPF for vehicle-pedestrian collisions at the targeted intersection can be calculated as follow:

$$N_{ped} = exp\ (a + b * ln\ (AADT_{tot}) + c * ln\ (\frac{AADT_{min}}{AADT_{maj}}) + d * ln\ (pedvol) + e * N_{lanesx}$$

Where;

AADT on major approaches = 13,500 veh/day

AADT on minor approaches = 12,000 veh/day

Total intersection AADT ($AADT_{tot}$) = 25,500 veh/day

Daily pedestrian volume (pedvol) = 1,000 ped/day

Maximum number of traffic lanes crossed by a pedestrian (N_{lanesx}) = 3 lanes

a, b, c, d, and e are the regression coefficients that can be found in the following table:

Table 16: SPFs for Vehicle-Pedestrian Collisions at Signalized Intersections (HSM, 2009)

Intersection Type	a	b	c	d	e
3SG	–6.60	0.05	0.24	0.41	0.09
4SG	–9.53	0.40	0.26	0.45	0.04

Because the targeted intersection is a 4-leg signalized intersection, then the value of a, b, c, d, e can be determined using table (16) as –9.53, 0.40, 0.26, 0.45, 0.04, respectively. The predicted crash frequency (N_{ped}) is:

$$N_{ped} = exp\ (-9.53 + 0.40 * ln\ (25500) + 0.26 * ln\ (\frac{12000}{13,500}) + 0.45 * ln\ (1000) + 0.04 * 3)$$

$$N_{ped} = 0.103\ crash/year$$

However, the crash frequency should be calibrated for individual geometry design and traffic control features using the Accident Modification Factors (CMF_7, CMF_8, and CMF_9).

$$N_{pedc} = N_{ped} * CMF_1 * CMF_2 * CMF_3$$

$$N_{pedc} = 0.103 * 4.15 * 1.35 * 1 = 0.577\ crash/year$$

Vehicle-Bicycle Collisions

The general SPF for vehicle-bicycle collisions at the treated intersection can be calculated as follow:

$$N_{bike} = N_b * f_{bike}$$

Where;

$$N_b = N_{spf} *AMF_1 * AMF_2 * AMF_3 * AMF_4 * AMF_5 * AMF_6$$

$$N_{spf} = N_{bimv} + N_{bisv}$$

Bicycle accident adjustment factor (f_{bike}) can be found in the following table:

Table 17: Bicycle Accident Adjustment Factor (HSM, 2009)

Intersection Type	**(f_{bike})**
3ST	0.016
3SG	0.011
4ST	0.018
4SG	0.015

As long as the targeted intersection is a 4-leg signalized intersection, the value of f_{bike} is 0.015. The number of vehicle-bicycle predicted crash frequency (N_{bike}) is:

$$N_b = (4.143)*0.9 *1 *1 *0.9604 *0.9107 *1 = 3.26 \text{ crash/year}$$

$$N_{bike} = 3.26 * 0.015 = 0.049 \text{ crash/year}$$

Predicted Average Crash Frequency before Implementing the Countermeasure

The predicted crash frequency at the E. College Ave. & Shortlidge Rd./S. Garner St intersection is calculated for vehicle-pedestrian and vehicle-bicycle crash type. The total average crash frequency can be found as follows:

$$N_{\text{predicted int.}} = C_i (N_{pedc} + N_{bike})$$

$$N_{\text{predicted int.}} = 1 (0.577 + 0.049) = 0.626 \text{ crash/year}$$

Predicted Average Crash Frequency after Implementing the Countermeasure

The countermeasure installed was comprised of adding a 3 second leading pedestrian interval at 10 intersections. The Crash Modification Factor Clearinghouse provides a CMF of 0.63 for LPI treatments. This means the crash frequency after implementing the countermeasure is expected to be 0.34 crashes per year; or in other words a reduction of 0.2 crashes per year can be seen.

Operational Performance Analysis

This section applies the methodology used for operational performance analysis of implementing Leading Pedestrian Interval (LPI). Field data from Fayish and Gross' study and Google earth is used for the analyses of signal timing. Data is collected from VISSIM simulation for the two before and after treatment periods for operational measures, including approach delay, queue length, and vehicular emission within that area. The computations of these measures are discussed next.

Synchro Simulation and Results

Optimized signal timing for the E. College Ave & Shortlidge Rd./S. Garner St. intersection was obtained using Synchro. The input data required for this included intersection geometry and hourly traffic volume for each approach. After the treatment traffic signal was increased by 3 seconds for Leading Pedestrian Interval.

Synchro results show that the signal has a 50 second cycle length and two phases for the before period. S. Garner St. and Shortlidge Rd. are represented by phase 1 signal group number 2 and 6, respectively, with 25 seconds total split. E. College Ave is represented by phase 2 signal group number 8 with 25 seconds total split. All turning movements are permitted (see figure 16a).

After treatment, the signal would have a 60 second cycle length with four phases, including the 3 second LPI. Phase 1 represents the leading pedestrian interval for south/north pedestrian movements. S. Garner St. and Shortlidge Rd. are represented by phase 2 signal group numbers 2 and 6, respectively. Phase 3 represents leading pedestrian interval for east/west pedestrian movements. Phase (4) represents E. College Ave movement signal group 8. Once again, all turning movements are permitted (see figure 16b).

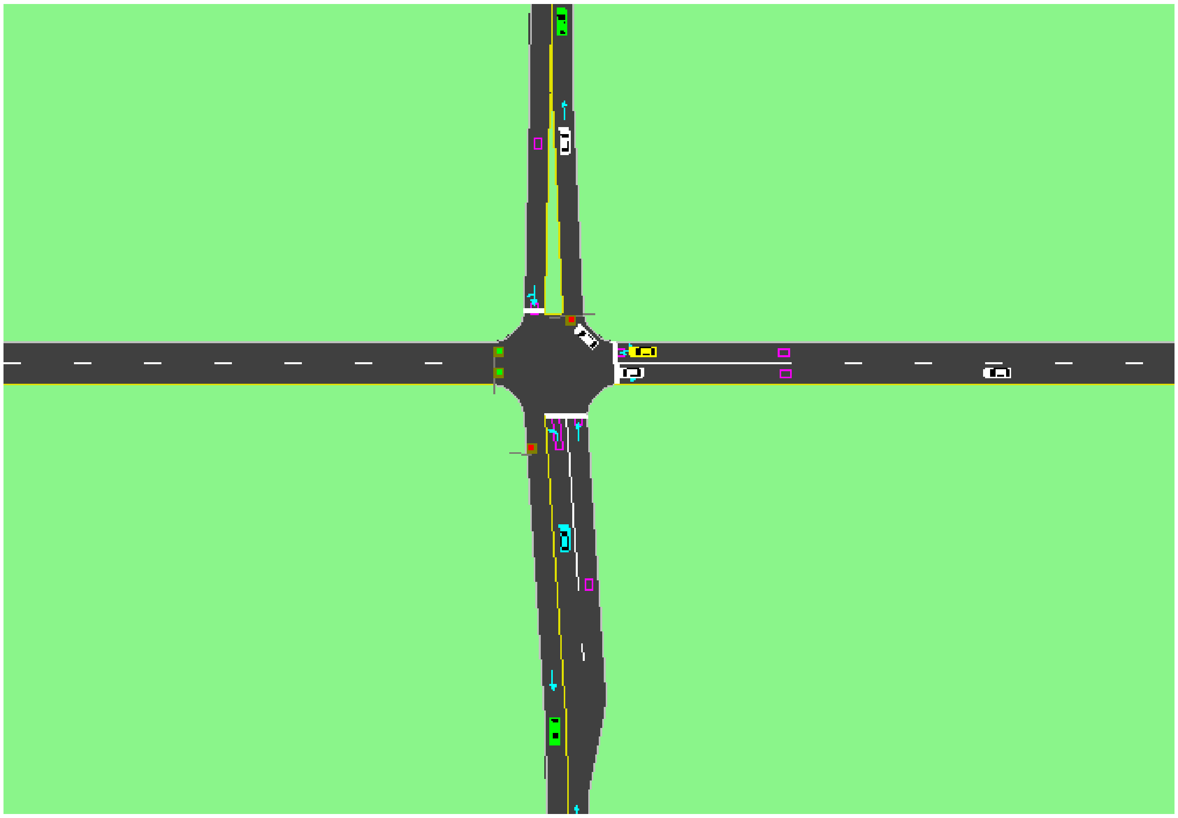
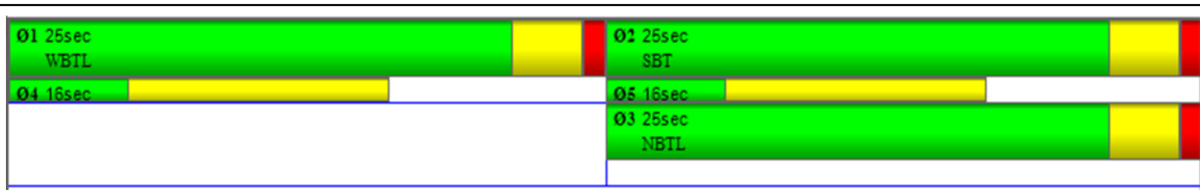


Figure 15: Synchro Simulation of E. College Ave & Shortlidge Rd/S Garner St Intersection



a. Signal Timing of the before treatment period

b. Signal Timing of the after treatment period

Figure 16: Signal Timing and Cycle Length for E. College Ave & Shortlidge Rd/S Garner St Intersection

Results and Discussion

The intent of this section is to cover the details of the findings for the second countermeasure. This thesis analyzes the impacts of LPI on safety and operation and provides in which condition this countermeasure is cost-effective. Different locations were simulated by varying the AADT, base values and geometries were taken from case studies developed by the Crash Modification Factor Clearinghouse. Table (18 and 19) below show the results of both safety and operational analysis per intersection for the before and after treatment periods, respectively. Safety measures and operational measures were analyzed in this study. The safety measures consisted of the number of vehicle-pedestrian and vehicle-bicycle crashes per year for before and after treatment. The operational measures consisted of change in traffic delay, queue length, vehicular emission and fuel consumption for the periods before and after treatment.

Table 18: Results of Before for E. College Ave & Shortlidge Rd/S Garner St Intersection

Before Treatment							
Scenarios	**Predicted Crashes (Crash/year)**	**Total Number of Vehicle**	**Delay (sec/veh)**	**Q-Length (ft.)**	**CO Emission (g)**	**NOx Emission (g)**	**Fuel Consumption (gal)**
70%*AADT	0.531	1413	10	69	1301	253	18.6
80%*AADT	0.564	1612	10	75	1481	288	21.2
90%*AADT	0.596	1829	12	94	1786	347	25.5
100%*AADT	0.626	2034	13	101	1856	361	26.6
110%*AADT	0.654	2229	14	102	2293	446	32.8
120%*AADT	0.682	2425	17	139	2661	518	38.1
130%*AADT	0.709	2612	19	195	3148	613	45.0

Table 19: Results of After for E. College Ave & Shortlidge Rd/S Garner St Intersection

After Treatment						
Scenarios	**Predicted Crashes (Crash/year)**	**Delay (sec/veh)**	**Maximum Q-Length (ft.)**	**CO Emission (g)**	**NOx Emission (g)**	**Fuel Consumption (gal)**
70%*AADT	0.335	14	65	1494	291	21.4
80%*AADT	0.356	14	71	1679	327	24.0
90%*AADT	0.375	16	96	1969	383	28.2
100%*AADT	0.394	17	101	2176	423	31.1
110%*AADT	0.412	23	122	2675	521	38.3
120%*AADT	0.430	30	190	3355	653	48.0
130%*AADT	0.447	53	400	4902	954	70.1

A Crash Modification Factor (CMF), for a 3 second leading pedestrian interval, was developed to determine the reduction in vehicle-pedestrian and vehicle-bicycle crashes. A CMF value of 0.63 was determined, indicating that after implementing the countermeasure a 37 percent reduction in crashes of all severity would be seen. While this decrease in crashes occurs, an operation analysis reviled that a negative influence on traffic delay, CO and NOx emissions, as well as fuel consumption would occur. A total of seven scenarios were considered with varying Annual Average Daily Traffic (AADT) in order to demonstrate the impact of implementing the countermeasure on safety and operation. Figures (17) through (23) show the before and after treatment crash and operation results.

The results of the first scenario, which consider the actual AADT (case study), are shown in figure (17). As can be seen, the implementation of the leading pedestrian interval countermeasure reduces the crash frequency by 0.231 crashes per year. However, a 4 second increase in the average traffic delay per vehicle occurs as a result of the treatment. Additionally, a significant increase in CO and NOx emission and fuel consumption can be seen. The after treatment increases in CO and NOx emission and fuel consumption are 230 grams per hour, 62 grams per hour, and 4.5 gallons per hour, respectively.

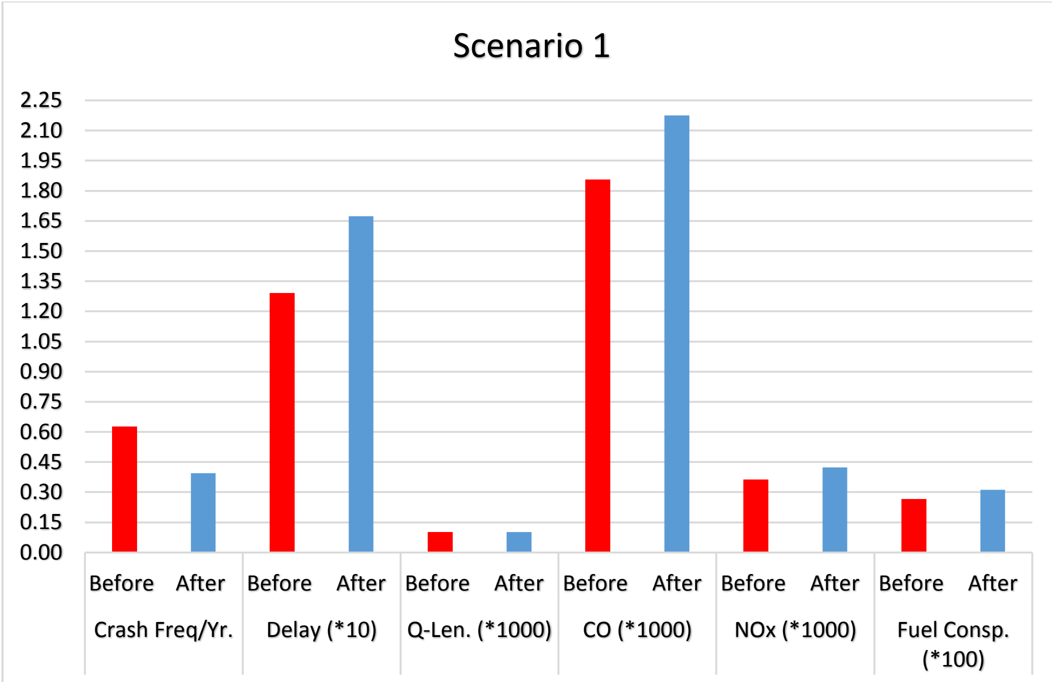


Figure 17: Before and After Differences at E. College Ave & Shortlidge Rd Intersection with Actual AADT

The results of the second and third scenarios, were a reduction of 10 and 20 percent of actual AADT was considered, is shown in figures (18) and (19). As was expected, a decrease in traffic volume resulted in a lower crash frequency reduction. A reduction of 0.22 and 0.21crashes per year was seen as a result of treating the intersection, considering the second and third scenarios, respectively. Simultaneously, the same 4 second increase in overall average vehicle delay, as experienced by the first scenario, affected both the second and third scenarios. While the same increase in delay was experienced, slightly lower increases in emissions and fuel consumption were seen. The increases in emissions were 184 and 198 grams per hour of CO and 36 and 39 grams per hour of NO_x, for the second and third scenarios respectively.

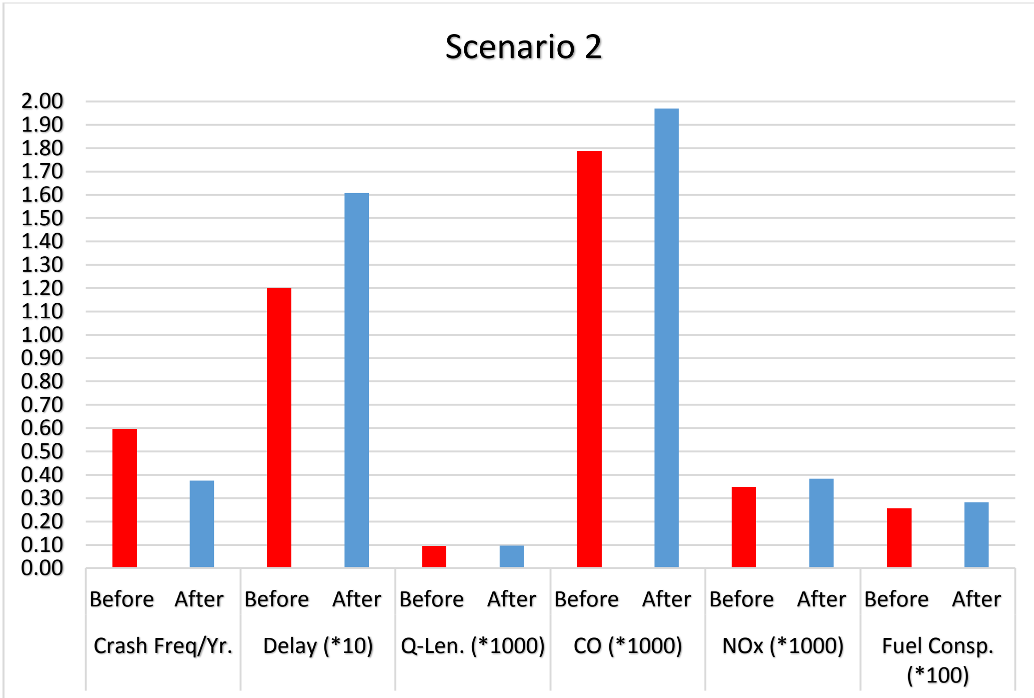


Figure 18: Before and After Differences at E. College Ave & Shortlidge Rd Intersection with 90% AADT

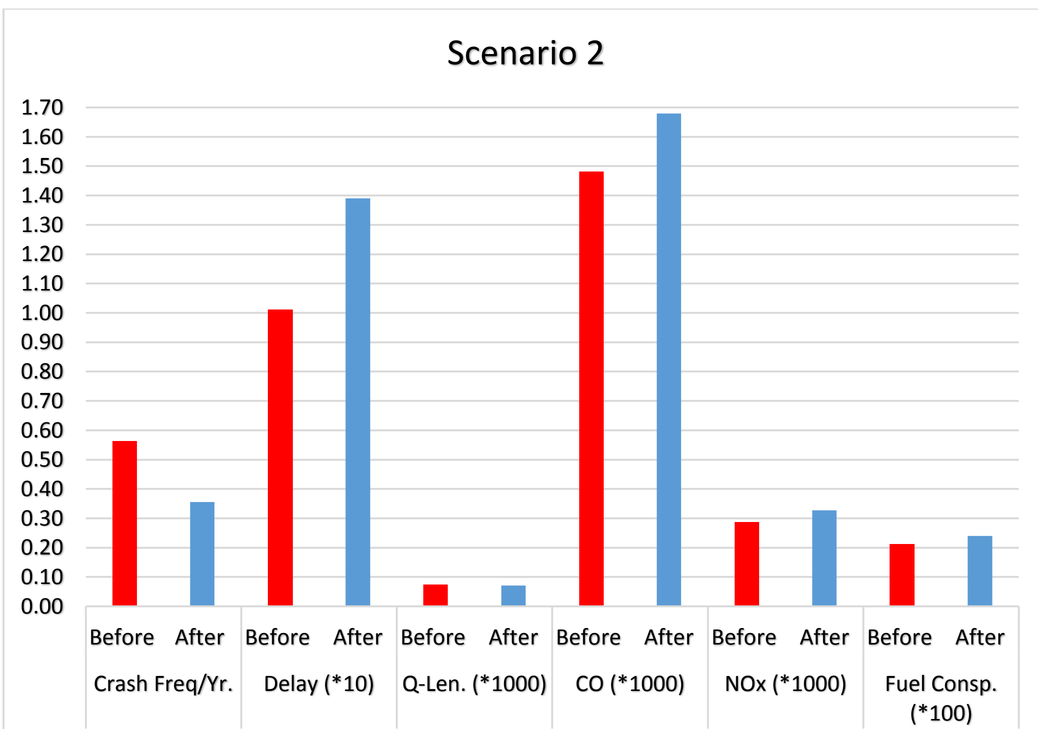


Figure 19: Before and After Differences at E. College Ave & Shortlidge Rd Intersection with 80% AADT

The results of scenario 4, which considers a reduction in AADT by 30 percent, are shown in figure (20). As can be seen, implementing a leading pedestrian interval at the selected intersection, with AADT reduction, will reduce crash frequency by 0.197 crashes per year. Once again the average traffic delay per vehicle will be increased by 4 seconds; however, the maximum queue length will remain the same. Additionally treatment will have little to no effect on the NOx emission and fuel consumption. A slight increase of 192 grams per hour for CO emissions will be seen after implementing the counter measure. It should be noted here that LPI for all scenarios with AADT's ranging between 17,800 and 25,500 vehicles per day, as evident by the first four scenarios, will cause the same average vehicle delay.

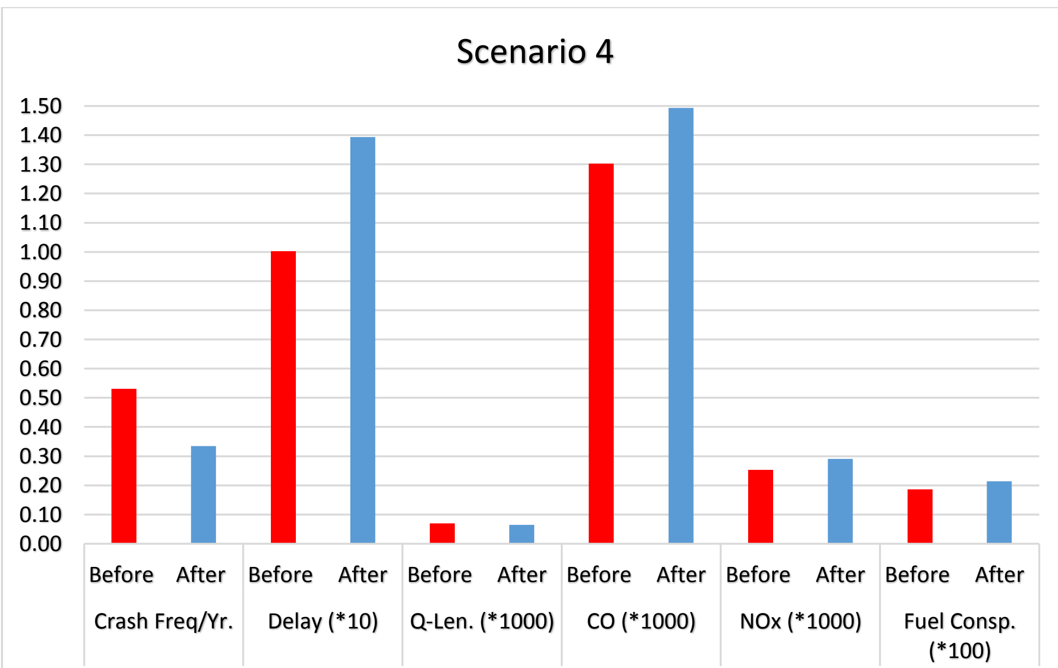


Figure 20: Before and After Differences at E. College Ave & Shortlidge Rd Intersection with 70% AADT

Figure (21), on the other hand, shows the results of the scenario that considered a 10 percent increase in AADT. This uptick in traffic resulted in an after treatment increase of vehicle delay by 9 seconds. Additionally, vehicular emission and fuel consumption both drastically increased after treatment. CO emission increased by 383 gram per hour, NOx emission increased by 74 gram per hour, and fuel consumption increased by 5.5 gallon per hour.

While an increase in delay and emissions was seen, a simultaneous reduction in total crash frequency occurred, resulting in 0.242 fewer crashes per year.

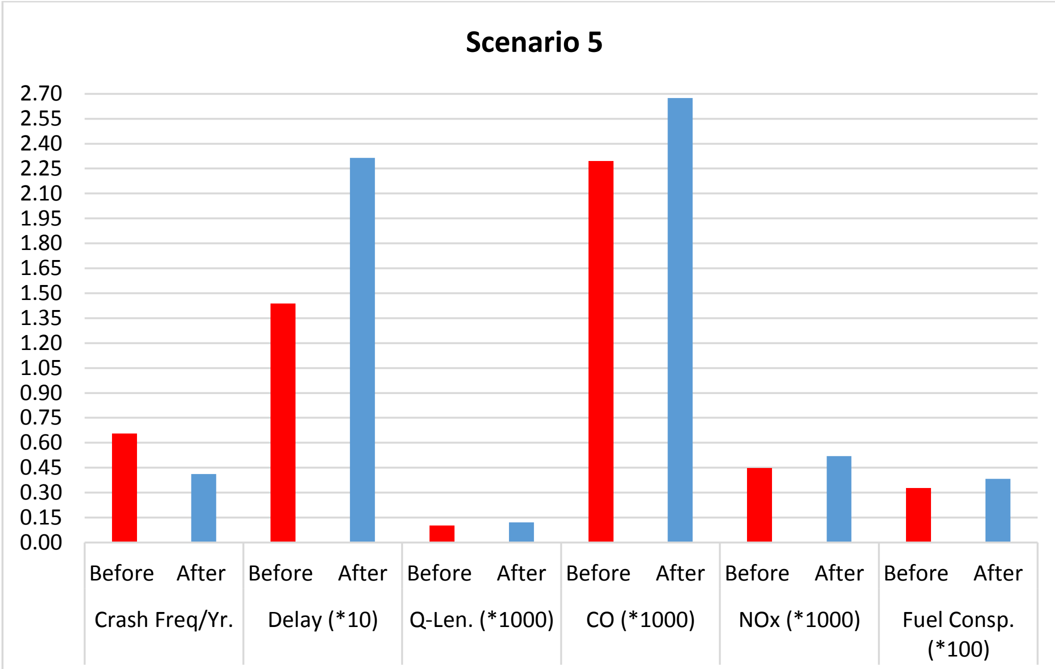


Figure 21: Before and After Differences at E. College Ave & Shortlidge Rd Intersection with 110% AADT

As evident in figures (22) and (23), representing scenarios 6 and 7 were a traffic volume increase of 20 and 30 present occurred, a drastic increase in traffic delay will occur when traffic volume in increase beyond 120% actual AADT. It can be observed that scenario 6 increased after treatment delay by 13 second and scenario 7 by 34 seconds. This means implementing countermeasures at sites with traffic volumes higher than 110% actual AADT can cause high traffic congestion and vehicular emission. Moreover, the comparison between before and after treatment periods showed an increase in CO emission by 694 and 1754 grams per hour, NOx emission increases of 135 and 341 grams per hour, and fuel consumption by 10 and 25.1 gallons per hour for scenarios 6 and 7, respectively. Both scenarios showed an increased crash reduction over the other scenarios of 0.252 and 0.262 crashes per year.

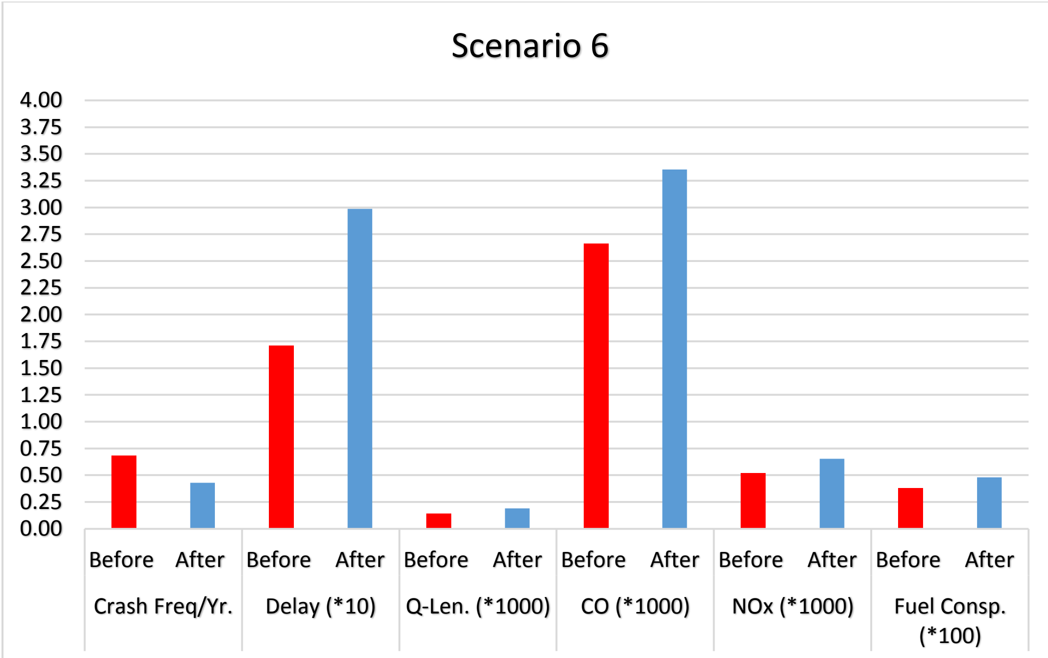


Figure 22: Before and After Differences at E. College Ave & Shortlidge Rd Intersection with 120% AADT

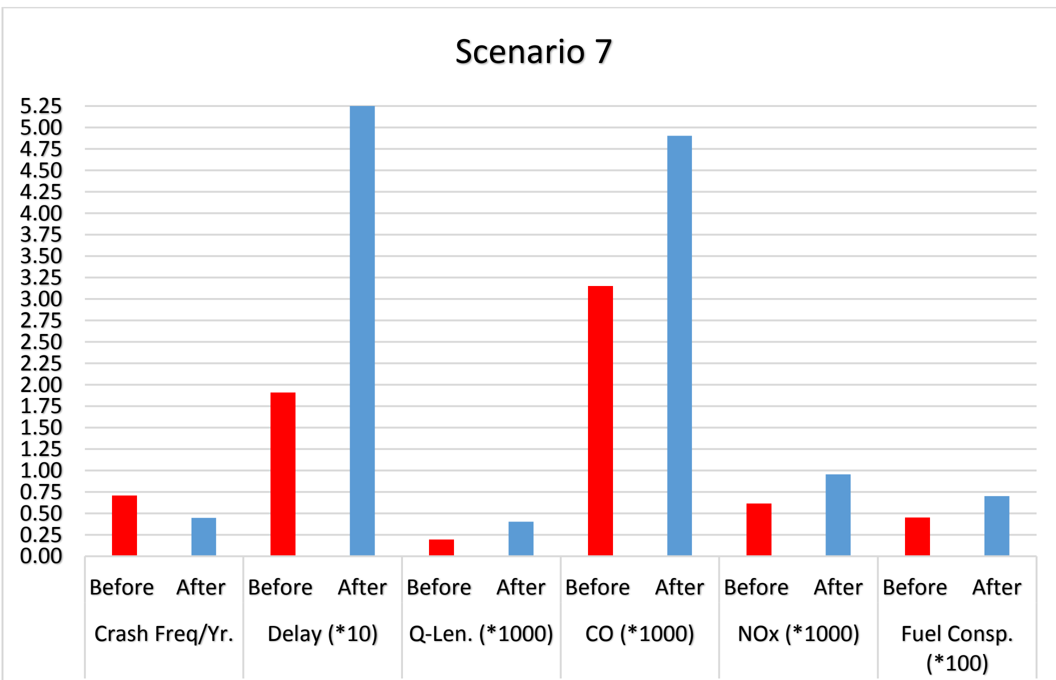


Figure 23: Before and After Differences at E. College Ave & Shortlidge Rd Intersection with 130% AADT

Cost-Benefit Analysis

Understanding the operational costs and safety benefits of a countermeasure is an important aspect to consider before implementing any changes. The intent of the following section is to discuss the economic analysis of adding a 3 second leading pedestrian interval. Chapter 3 of this thesis described the methodology and approaches used to determine crash and operational costs for the cost-benefit calculations. Cost-benefit analysis at varying traffic volumes and number of crashes observed will help States and local agencies determining if implementing the LPI will be economically beneficial or not.

Operation data were analyzed for seven different traffic volume scenarios. Data was collected, considering both before and after implementing the countermeasure, in order to determine the impact of the treatment on operational performance. In terms of safety, The CMF developed for this countermeasure was used to calculate the amount of savings resulting from a reduction in crashes. By applying factors to the base number of crashes, the cost and benefit were observed for varying amounts of crashes, which were crash factors multiplied by the base number of crashes. Operational cost and crash savings due to the treatment for the seven scenarios were combined in order to show the cost and benefit trends after the treatment.

Figures (24) and (25) show the comparison between crash saving, delay cost, and overall operational cost, considering an increase in the number of observed crashes. Leading Pedestrian Interval (LPI) implemented at the intersection of College Ave. & Shortlidge Rd/S Garner St. reduced crash frequency by 0.231 crashes per year at the base condition (actual AADT). This resulted in a saving of $4,630 per year. The total delay cost at the intersection, however, was $89,119 per year and $89,486 per year for operation factors, including delay, CO emission, and NOx emission.

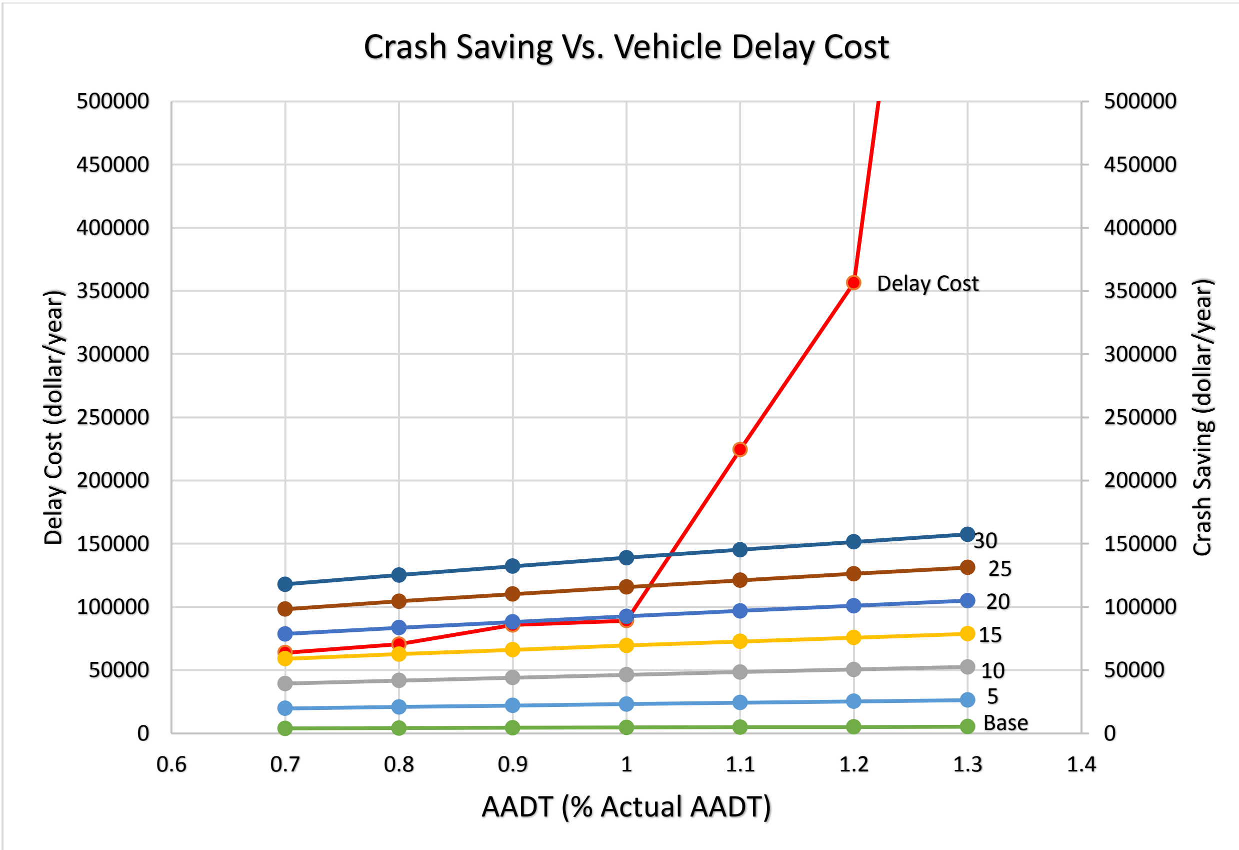


Figure 24: Crash Saving and Delay Cost Comparison for E. College Ave & Shortlidge Rd Intersection

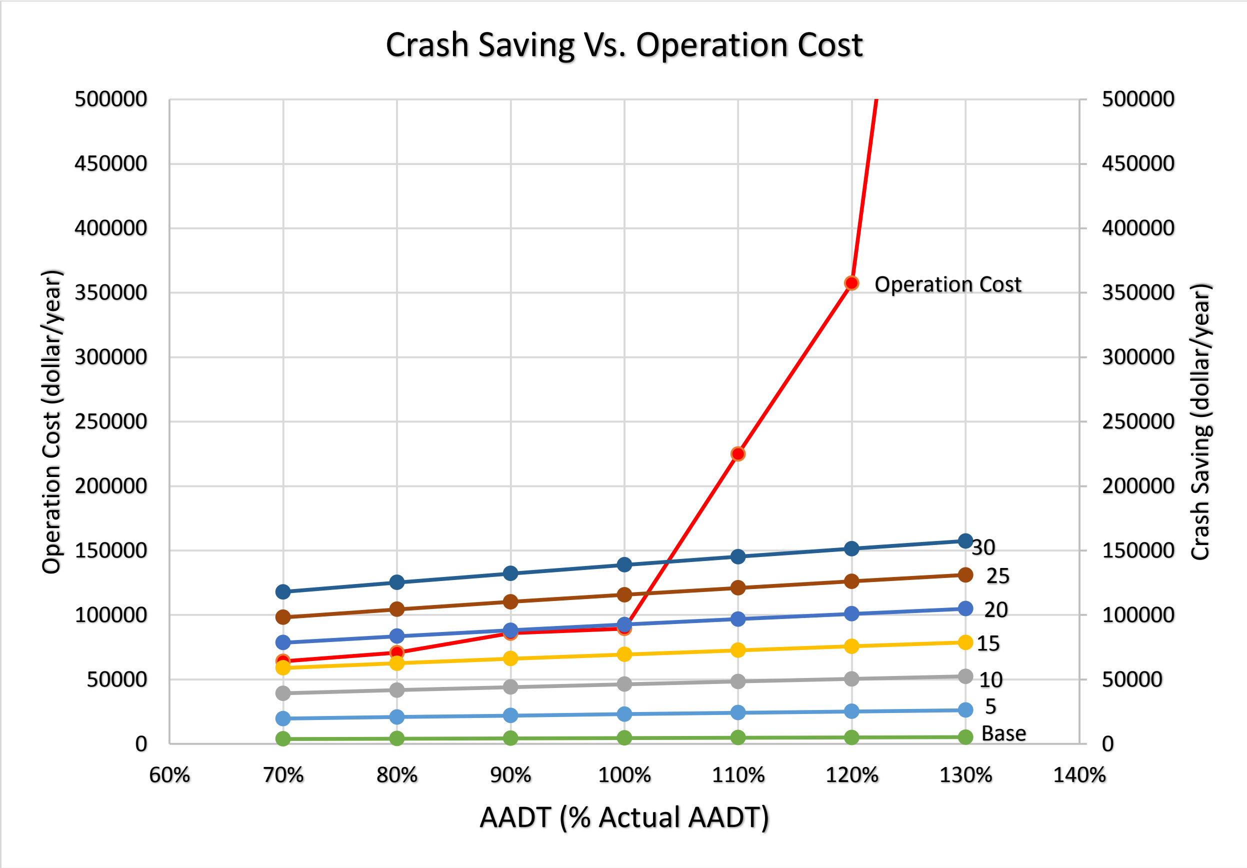


Figure 25: Crash Saving and Operation Cost Comparison for E. College Ave & Shortlidge Rd Intersection

The treatment, therefore, resulted in a cost of approximately $ 84,856 per year at the selected intersection within average crash observation (Base) due to the treatment. Additionally, the other scenarios considered showed the same cost-benefit trend. A decrease in AADT by 30, 20, or 10 percent could lead to a significant benefit if the treated intersection observed more than 10 crashes per year. An increase in AADT by 30 percent led to a significant increase in cost of $1,017,380 per year. The highest benefit ($49,406 per year) can be obtained when the treatment is implemented at the actual AADT with number of observed crashes 18.77 crash per year. In summary, the results show that the cost of implementing a leading pedestrian interval is higher than the benefit at the base condition. Table (19) summarizes cost-benefit analysis for all scenarios at different crash observations.

Table 20: Safety Benefit (saving) and Operational Cost of all Scenarios for E. College Ave & Shortlidge Rd Intersection

Scenarios		Crash Factor						
		Base (1)	5	10	15	20	25	30
70% * AADT	No. of Crashes	0.53	2.66	5.31	7.97	10.62	13.28	15.93
	Benefit/Cost	-60140	-44419	-24768	-5118	14533	34184	53835
80% * AADT	Crashes	0.56	2.82	5.64	8.46	11.29	14.11	16.93
	Benefit/Cost	-66602	-49899	-29021	-8142	12737	33615	54494
90% * AADT	No. of Crashes	0.60	2.98	5.96	8.94	11.91	14.89	17.87
	Benefit/Cost	-81455	-63822	-41782	-19741	2299	24340	46381
100% * AADT	No. of Crashes	0.63	3.13	6.26	9.39	12.51	15.64	18.77
	Benefit/Cost	-84856	-66337	-43189	-20040	3108	26257	49406
110% * AADT	No. of Crashes	0.65	3.27	6.54	9.82	13.09	16.36	19.63
	Benefit/Cost	-220070	-200701	-176489	-152277	-128066	-103854	-79642
120% * AADT	No. of Crashes	0.68	3.41	6.82	10.23	13.64	17.05	20.46
	Benefit/Cost	-352319	-332129	-306893	-281656	-256419	-231182	-205945
130% * AADT	No. of Crashes	0.71	3.54	7.09	10.63	14.18	17.72	21.27
	Benefit/Cost	-1017380	-996396	-970167	-943938	-917709	-891479	-865250

Countermeasure 3: Exclusive Pedestrian Phase (i.e. Barnes Dance)

This section describes the approach used to complete and evaluate, including safety evaluation and operational evaluation, the effectiveness of adding an exclusive pedestrian phase to the regular two-phase permissive signal timing, which allows pedestrians to cross in any direction, including diagonally (Chen et al., 2012).

Study Site

A set of treatments have been done in NYC in order to reduce pedestrian-vehicle crash frequency at intersections. One of those treatments involved the Barnes Dance. Chan et al. (2012) mentioned in their study that Barnes Dance was mostly implemented in areas where pedestrian volumes are high. W 96th St & West End Ave intersection is the targeted intersection for evaluation in this study. W 96th St & West End Ave intersection is a four leg signalized intersection. West End Ave represents the minor approaches of the intersection with an average daily traffic volume of 12464 veh/day. West End Ave has one lane for through movement with two exclusive lanes for turning movements. W 96th St represents the major approaches of the intersection with an average daily traffic volume of 23000 veh/day. Each approach has two lanes for the through movement with left-turn sharing movement and one exclusive lane for right-turn movement (see figure 26). All treated intersections have experienced high level of pedestrian activity. Highway Safety Manual (HSM) provides a table of daily pedestrian volume depending on the level of activity (see table 21). Median-high activity level (1500 ped/day) was assumed as a level of activity in the targeted intersection (W 96th St & West End Ave). Pedestrian-movement signals and cross walks are presented at the intersection. Signal phasing and cycle length of the intersection are obtained using Synchro software, with consideration being given to before and after treatment phasing.

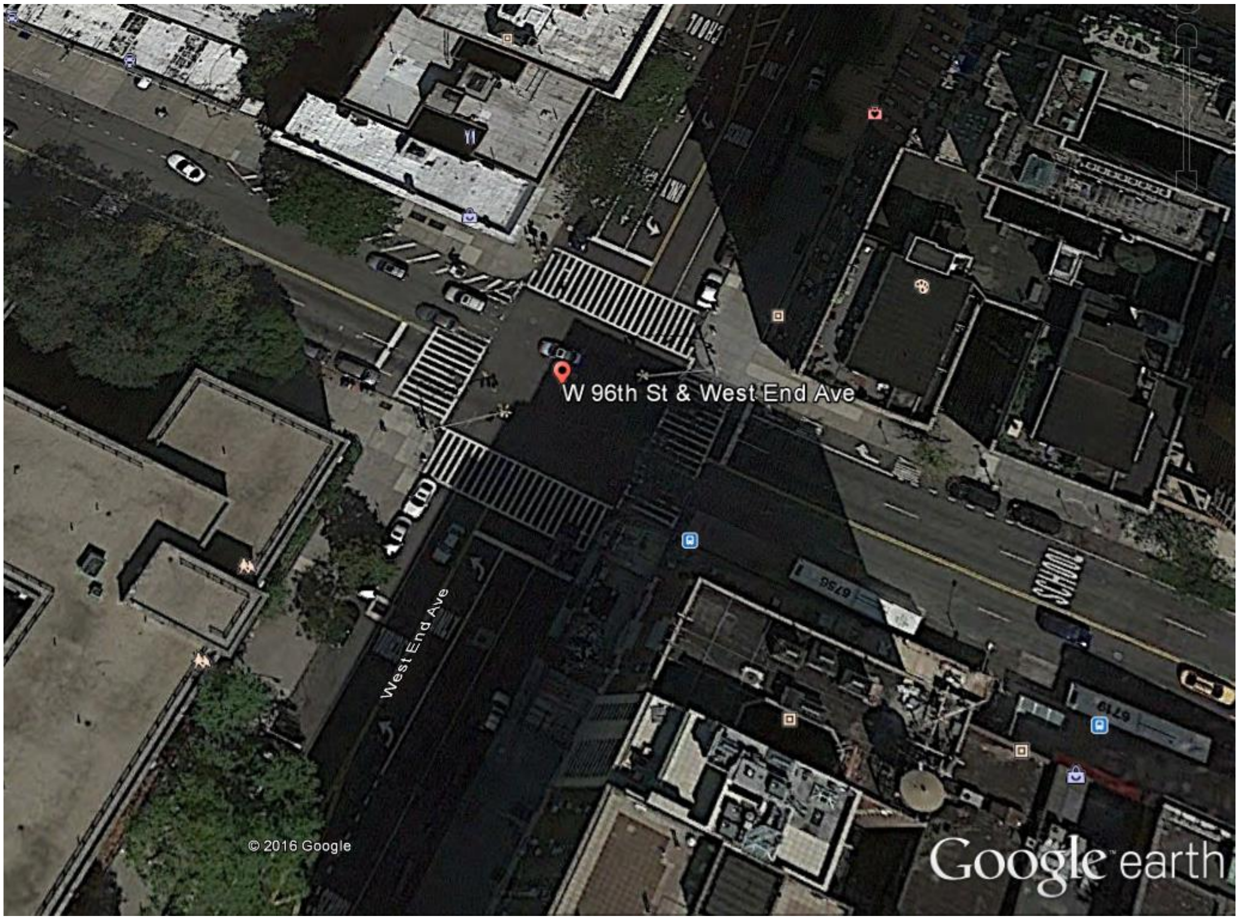


Figure 26: W 96th St & West End Ave intersection As Shown in Google Earth

Table 21: Pedestrian Crossing Volumes Based on Pedestrian Activity (HSM, 2009)

Level of Pedestrian Activity	Pedestrian Volume (ped/day)
High	3200
Medium-high	1500
Medium	700
Medium-low	240
Low	50

Safety Analysis

This section demonstrates the selection of the Safety Performance Functions (SPFs) for the base condition of the treated intersection as conducted in the Highway Safety Manual. Note that the base condition is represented by the selected location, as described in the previous section, with the recent Annual Average Daily Traffic (AADT).

Safety Performance Function (SPF) Selection and Calibration

W 96th St & West End Ave intersection is a four-leg signalized intersection. The intersection has experienced a number of pedestrian-vehicle crash type; the treatment has been implemented due to potential conflicts between pedestrians and motorists. The SPF value will be calculated for pedestrian-vehicle collisions only due to the occurrence of that type of crashes. As long as all crash severities of the targeted case study intersection have been recorded, all severity levels will be considered.

Accident Modification Factors (AMF)

In order to estimate the average crash frequency at W 96th St & West End Ave intersection, the selected SPFs for pedestrian-vehicle collisions should be adjusted for individual geometry design and traffic control features. AMF_1 through AMF_3 are applied for the total vehicle-pedestrian collision only. The higher AMF gives a higher crash frequency, and vice versa. The accident modification factors for the targeted intersection are calculated as follow.

Bus Stop AMF (AMF_1)

The absence of bus stops near the intersection (no bus stop within 1000 ft. of the center of the intersection) is the base condition (HSM, 2009). Three bus stops were presented within 1000 ft. of the treated intersection. Therefore, the value of AMF_1 is equal to 4.15 (see table 22).

Table 22: Accident Modification Factor for Number of Bus Stops (HSM, 2009)

Number of bus stops within 1,000 ft. of the intersection	**AMF_7**
1 or 2	2.78
3	4.15

School CMF (AMF_2)

The absence of schools near the intersection (no school within 1000 ft. of the center of the intersection) is the base condition (HSM, 2009). Due to the present of school within the intersection zone, the value of AMF_8 is equal to 1.35 (see table 23).

Table 23: Accident Modification Factor for Number of Schools (HSM, 2009)

Presence of schools within 1,000 ft. of the intersection	**AMF_8**
No School present	1
School present	1.35

Alcohol Sales Establishments CMF (AMF_3)

The absence of alcohol sales establishments near the intersection (no alcohol sales establishments within 1000 ft. of the center of the intersection) is the base condition (HSM, 2009). No alcohol sales establishments are presented within 1000 ft. of the treated intersection; therefore, the value of AMF_9 in this case is 1.

Vehicle-Pedestrian Collisions

The general SPF for pedestrian-vehicle collisions at the W 96th St & West End Ave intersection can be calculated as follow:

$$N_{ped} = exp\ (a + b * ln\ (AADT_{tot}) + c * ln\ (\frac{AADT_{min}}{AADT_{maj}}) + d * ln\ (pedvol) + e * N_{lanesx}$$

Where:

AADT on major approaches (W 96th St) = 23,000 veh/day

AADT on minor approaches (West End Ave) = 12,464 veh/day

Total intersection AADT ($AADT_{tot}$) = 35,464 veh/day

Daily pedestrian volume (pedvol) = 1,500 ped/day

Maximum number of traffic lanes crossed by a pedestrian (N_{lanesx}) = 5 lanes

a, b, c, d, and e are the regression coefficients that can be found in the following table:

Table 24: SPFs for Vehicle-Pedestrian Collisions at Signalized Intersections (HSM, 2009)

Intersection Type	a	b	c	d	e
3SG	–6.60	0.05	0.24	0.41	0.09
4SG	–9.53	0.40	0.26	0.45	0.04

Because the targeted intersection is a 4-leg signalized intersection, then the value of a, b, c, d, e can be determined using table (24) as –9.53, 0.40, 0.26, 0.45, 0.04, respectively. The predicted crash frequency (N_{ped}) is:

$$N_{ped} = exp\ (-9.53 + 0.40 * ln\ (35464) + 0.26 * ln\ (\frac{12464}{23{,}000}) + 0.45 * ln\ (1500) + 0.04 * 5)$$

$$N_{ped} = 0.13\ crash/year$$

Predicted Average Crash Frequency before Implementing the Countermeasure

The predicted crash frequency at W 96th St & West End Ave intersection ($N_{predicted\ int.}$) is calculated for vehicle-pedestrian collisions type only. Vehicle-pedestrian collisions should be calibrated for individual geometry design and traffic control features using the Accident Modification Factors (CMF_1, CMF_2, and CMF_3).

$$N_{predicted\ int.} = N_{ped.} * CMF_1 * CMF_2 * CMF_3$$

$$N_{predicted\ int.} = 0.13 * 4.15 * 1.35 * 1 = 0.75\ crash/year$$

Predicted Average Crash Frequency after Implementing the Countermeasure

The countermeasure installed was comprised of adding exclusive pedestrian phase to the regular cycle time (i.e. Barnes Dance). The Crash Modification Factor Clearinghouse provides a CMF of 0.49 for Barnes Dance treatment. This means that the crash frequency after implementing the countermeasure is expected to be 0.369 crashes per year; or in other words a reduction of 0.384 crashes per year can be seen.

Operational Performance Analysis

This section applies the methodology used for operational performance analysis of adding exclusive pedestrian phase. Field data from Chen et al. study and Google earth is used for the analyses of signal timing. Data is collected from VISSIM simulation for the two before and after treatment periods for operational measures, including approach delay, queue length, vehicular emission, and pedestrian delay within that area. The computations of these measures are discussed next.

Synchro Simulation and Results

Optimized signal timing for the W 96th St & West End Ave intersection was obtained using Synchro. The input data required for this included intersection geometry and traffic volume for each approach. Intersection-movements phasing are presented by regular two-phase permissive signal timing. Prior to the treatment, pedestrians (Walk Time and Flash Don't Walk) are served by minimum green time of the corresponding vehicle movement. After treatment, pedestrians are allowed to cross in any fashion, including diagonally, and they are served by an exclusive phase (phase 105) for 33 seconds (see figure 27).

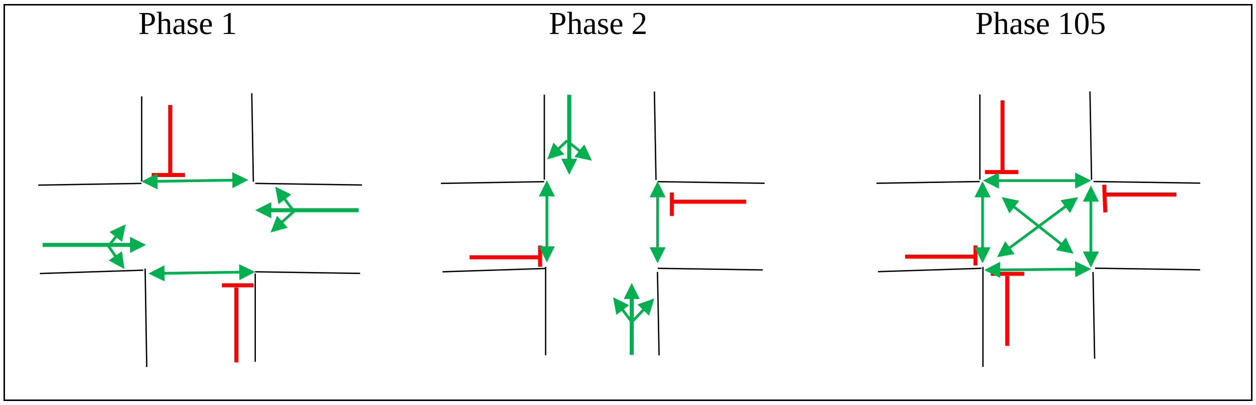


Figure 27: Three Phases of Signal Timing Showing the Barnes Dance

Synchro results show that the signal has a 60 second cycle length and two phases for the before period. West End Ave movements (including turning movements) are represented by phase 1 signal group number 1 and 2, while W 96th St movements, including turning movements, are represented by phase 3 signal group 3 and 4. Pedestrian-movements are represented by signal group number 101, 102, 103, and 104. Each movement is served by minimum green time of the corresponding vehicle movement (see figure 29a).

After treatment (stops vehicle traffic in all directions and allows pedestrians to cross in any direction) exclusive pedestrian phase will be added to the regular two-phase permissive signal timing. The signal would have three phases with a 93 second cycle length. In this case Phase 3 will allow pedestrians to cross in any fashion. Pedestrians will be served for 33 second during the exclusive phase. Pedestrians will also be served by the minimum green time of the corresponding vehicle movement (see figure 29b).

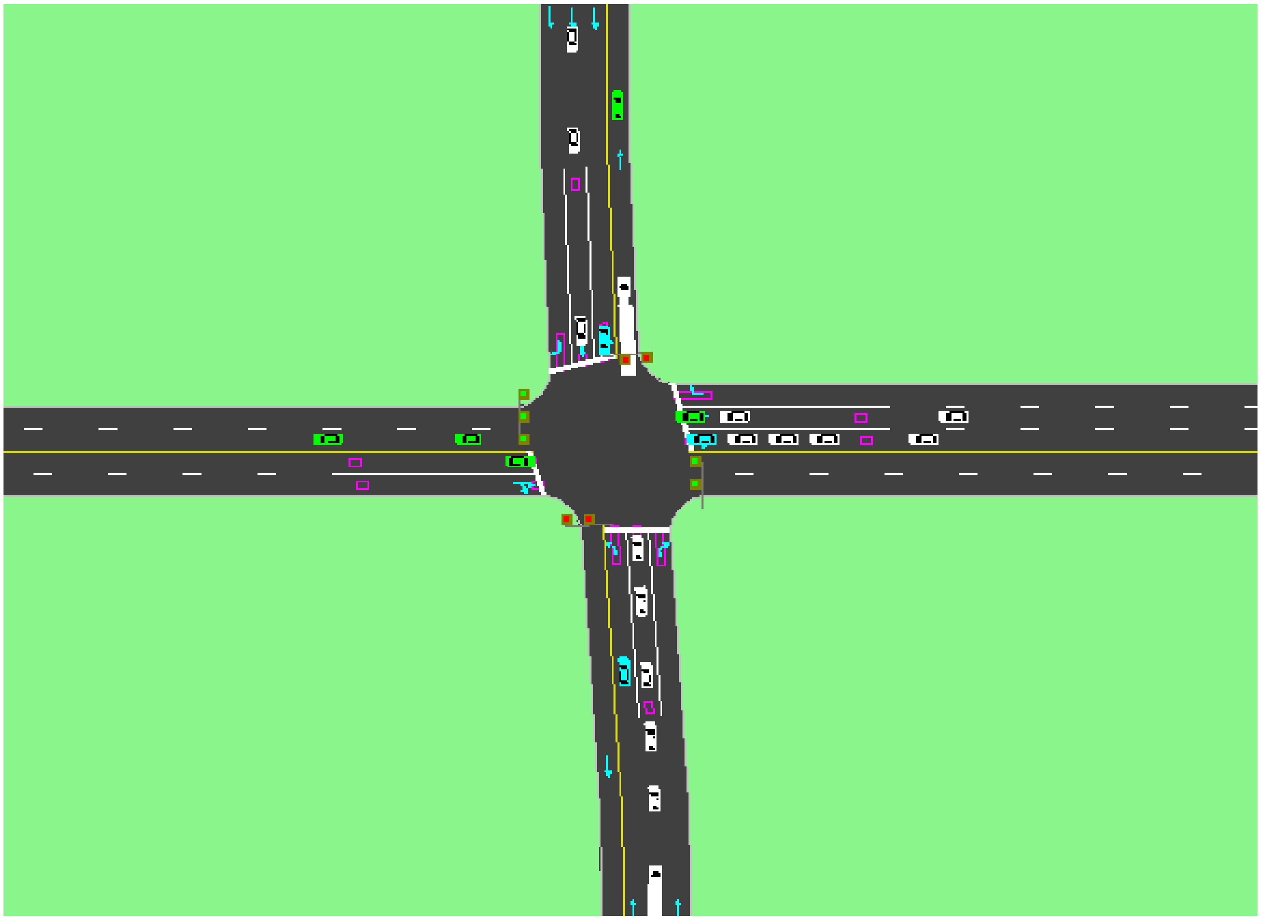


Figure 28: Synchro Simulation of W 96th St & West End Ave Intersection

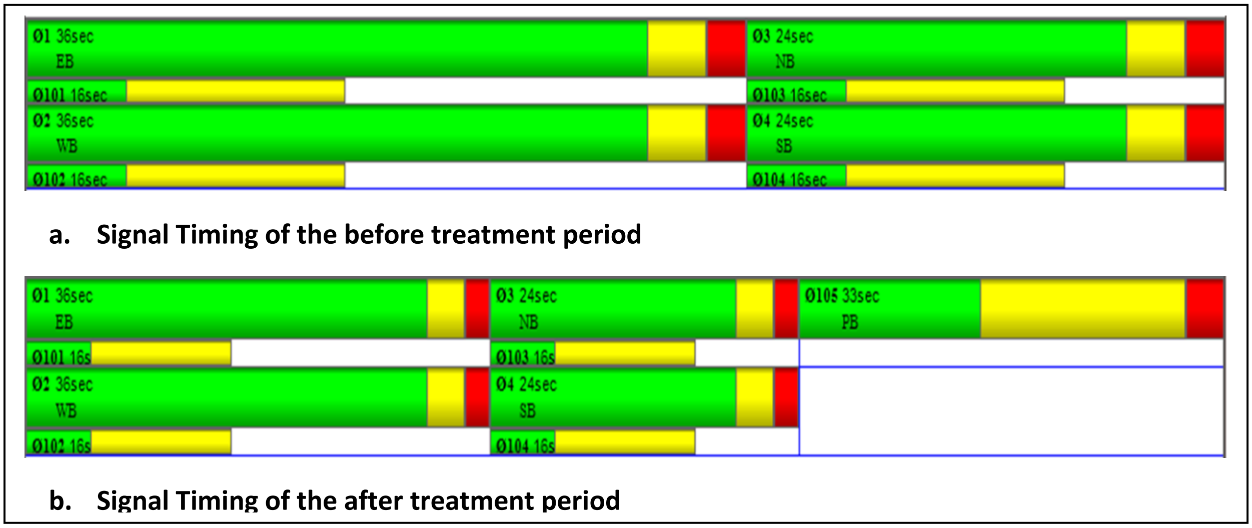


Figure 29: Signal Timing and Cycle Length of W 96th St & West End Ave Intersection

Results and Discussion

The intent of this section is to cover the details of the findings for the third countermeasure. This thesis considers the impacts of the Barnes Dance on safety and operation and provides in which condition this countermeasure is cost-effective. Different locations were simulated by varying the AADT, base values and geometries were taken from case studies developed by the Crash Modification Factor Clearinghouse. Table (25 and 26) below show the results of both safety and operational analysis, including pedestrian delay, per intersection for the before and after treatment periods, respectively. The safety measures consisted of the number of pedestrian-vehicle crashes per year before and after treatment. The operational measures consisted of change in vehicle delay, queue length, vehicular emission, fuel consumption, and pedestrian delay for the periods before and after treatment. The results were collected for only six scenarios in this countermeasure. Scenarios with more than 110% of the AADT led to non-reliable results as the intersection became very jammed and out of reach (for certain vehicles) for the VISSIM evaluation area.

Table 25: Results of Before Treatment Period for W 96th St & West End Ave Intersection

Before Treatment								
Scenarios	**Predicted Crashes (Crash/year)**	**Total Number of Vehicle**	**Delay (sec/veh)**	**Q-Length (ft.)**	**CO Emission (g)**	**NOx Emission (g)**	**Fuel Consumption (gal)**	**Ped. Delay (sec)**
70%*AADT	0.65	2479	11	38	2632	512	37.66	21
80%*AADT	0.69	2827	12	64	3105	604	44.42	24
90%*AADT	0.72	3123	17	390	3790	737	54.22	28
100%*AADT	0.75	3475	18	236	4241	825	60.67	29
110%*AADT	0.78	3824	25	505	5432	1057	77.71	29
120%*AADT	0.81	4009	51	510	8036	1563	114.96	44

Table 26: Results of After Treatment Period for W 96th St & West End Ave Intersection

Before Treatment							
Scenarios	**Predicted Crashes (Crash/year)**	**Delay (sec/ veh)**	**Q-Length (ft.)**	**CO Emission (g)**	**NOx Emission (g)**	**Fuel Consumption (gal)**	**Ped. Delay (sec)**
70%*AADT	0.320	33	197	6599	1284	94.40	27
80%*AADT	0.337	30	118	7031	1368	100.58	34
90%*AADT	0.353	56	505	9815	1910	140.42	35
100%*AADT	0.369	64	510	11144	2168	159.43	36
110%*AADT	0.383	80	510	13178	2564	188.52	36
120%*AADT	0.397	86	510	14367	2795	205.53	51

A Crash Modification Factor (CMF), for an exclusive pedestrian phase, was developed to determine the reduction in vehicle-pedestrian. A CMF value of 0.49 was determined, indicating that after implementing the countermeasure a 51 percent reduction in crashes of all severity would be seen. While this decrease in crashes occurs, an operation analysis reviled that a negative influence on traffic delay, CO and NOx emissions, as well as fuel consumption would occur. A total of seven scenarios were considered with varying Annual Average Daily Traffic (AADT) in order to demonstrate the impact of implementing the countermeasure on safety and operation. Figures (30) through (35) show the before and after treatment crash and operation results.

The results of the first scenario, which consider the actual AADT (case study), are shown in figure (30). As can be seen, the implementation of the exclusive pedestrian phase countermeasure reduces the crash frequency by 0.384 crashes per year. However, a 46 second increase in the average traffic delay per vehicle occurs as a result of the treatment and the intersection would experience a significant increase in queue length. Additionally, an increase in CO and NOx emission and fuel consumption can be seen. The after treatment increases in CO and NOx emission and fuel consumption are 6903 grams per hour, 1343 grams per hour, and

98.75 gallons per hour, respectively. Additionally, higher pedestrian delay was observed at the exclusive pedestrian phase (After treatment).

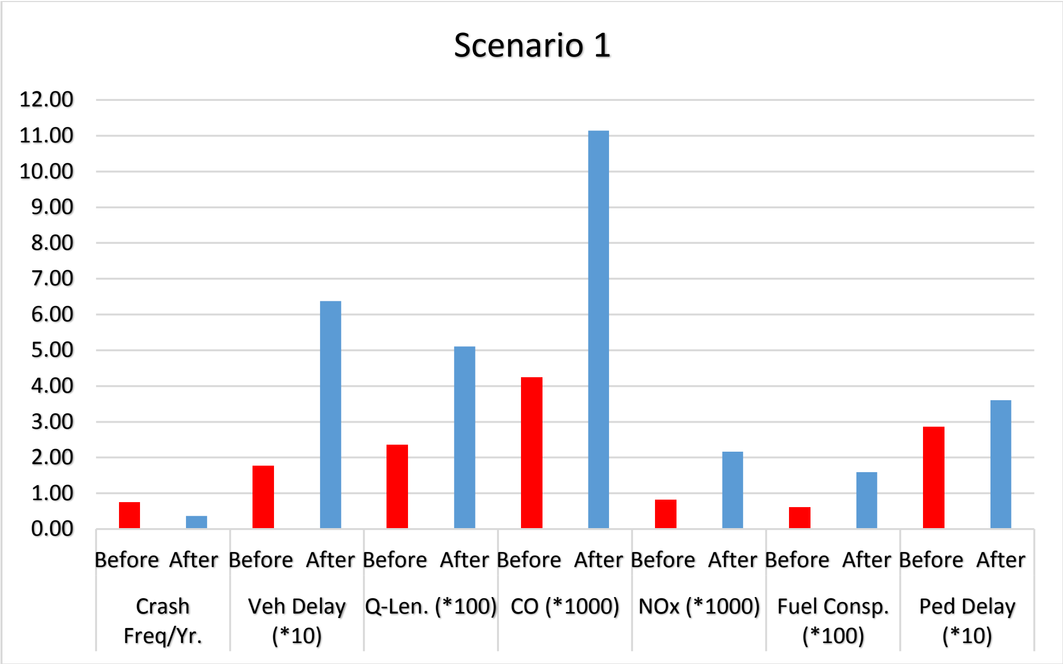


Figure 30: Before and After Differences at W 96th St & West End Ave Intersection with Actual AADT

The results of the second and third scenarios, were a reduction of 10 and 20 percent of actual AADT was considered, is shown in figures (31) and (32). As was expected, a decrease in traffic volume resulted in a lower crash frequency reduction. A reduction of 0.368 and 0.351 crashes per year was seen as a result of treating the intersection, considering the second and third scenarios, respectively. Simultaneously, 39, 18 second increase in average vehicle delay affected both the second and third scenarios, respectively. Also, an increase in emissions and fuel consumption were seen. The increases in emissions were 6025 and 3926 grams per hour of CO and 1172 and 764 grams per hour of NO_x, for the second and third scenarios respectively.

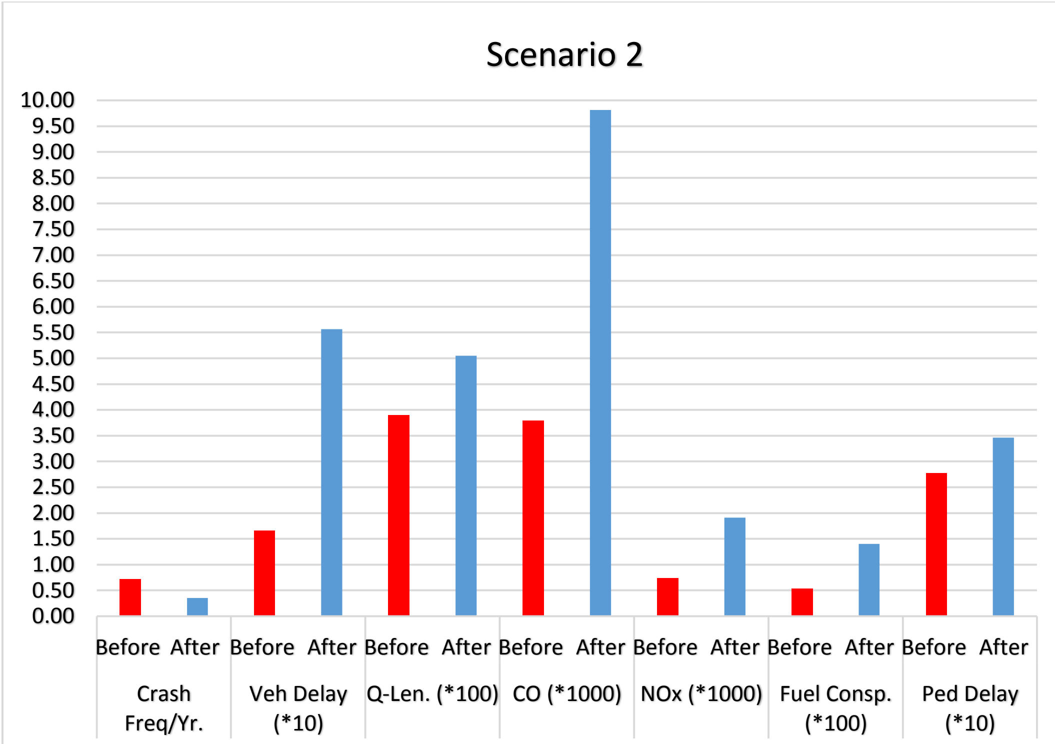


Figure 31: Before and After Differences at W 96th St & West End Ave Intersection with 90% AADT

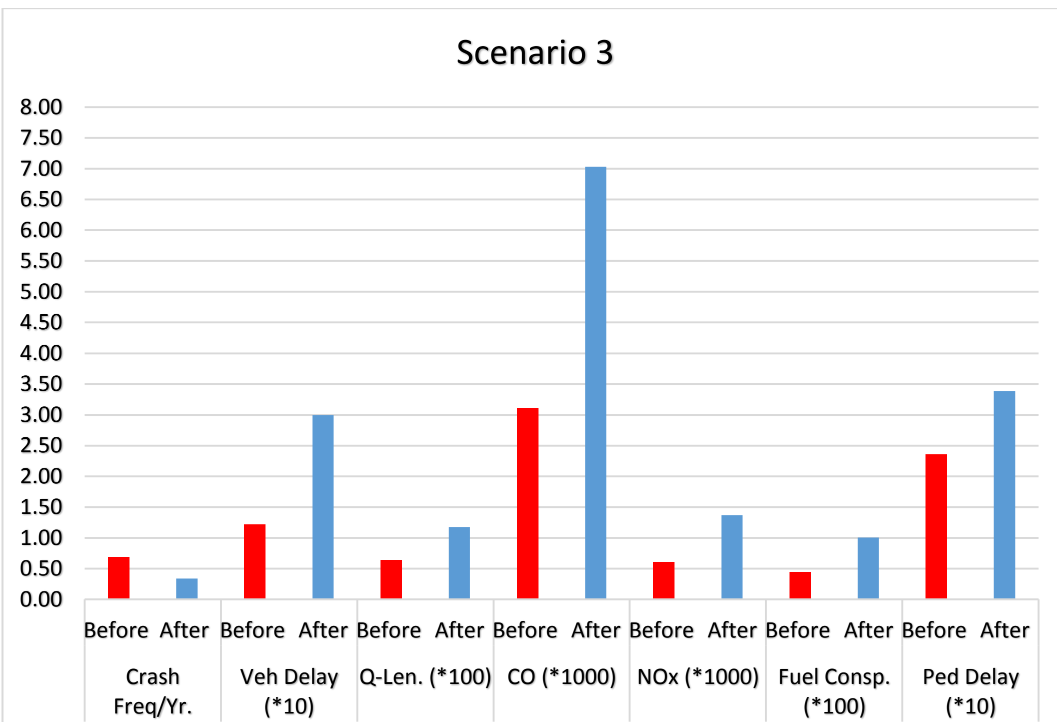


Figure 32: Before and After Differences at W 96th St & West End Ave Intersection with 80% AADT

The results of scenario 4, which considers a reduction in AADT by 30 percent, are shown in figure (33). As can be seen, implementing an exclusive pedestrian interval at the selected intersection, with AADT reduction, will reduce crash frequency by 0.333 crashes per year. Once again the average traffic delay per vehicle will be increased by 22 second; also, the maximum queue length will significantly increase. An increase of 3966 grams per hour for CO emissions will be seen after implementing the counter measure. The average pedestrian delay at the treated intersection will increase by 0.6 second after treatment.

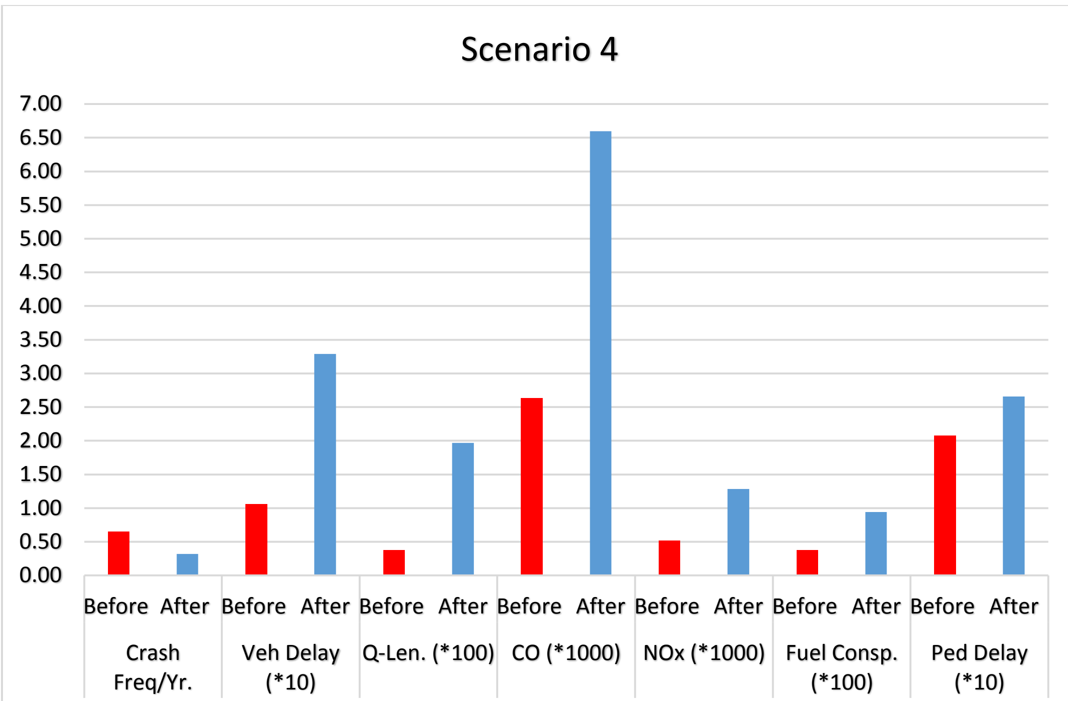


Figure 33: Before and After Differences at W 96th St & West End Ave Intersection with 70% AADT

Figure (34), on the other hand, shows the results of the scenario that considered a 10 percent increase in AADT. This uptick in traffic resulted in an after treatment increase of vehicle delay by 54 second. Additionally, vehicular emission and fuel consumption both drastically increased after treatment. CO emission increased by 7746 grams per hour, NOx emissions increased by 1507 grams per hour, and fuel consumption increased by 111 gallons per hour. While an increase in delay and emissions was seen, a simultaneous reduction in total crash frequency occurred, resulting in 0.4 crashes per year.

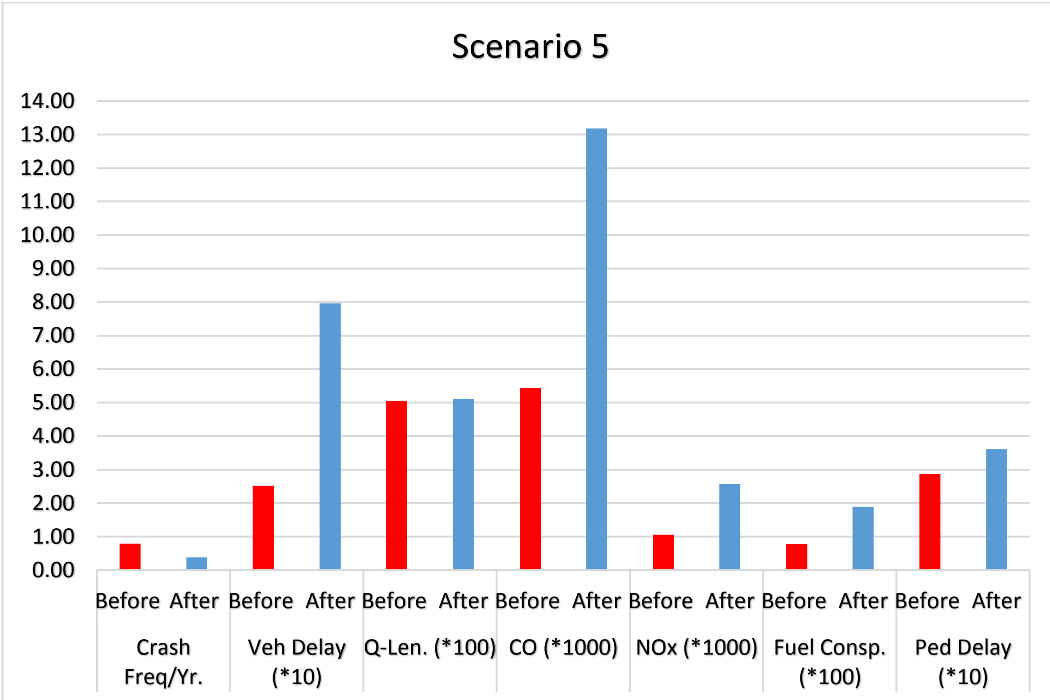


Figure 34: Before and After Differences at W 96th St & West End Ave Intersection with 110% AADT

As evident in figure (35), representing scenarios 6 were a traffic volume increase of 20 present occurred, a lower increase in traffic delay will occur when traffic volume in increase beyond 110% AADT. It can be observed that scenario 6 increased after treatment delay by 35 second. This means, Scenarios with more than 110% of the AADT led to non-reliable results as the intersection became very jammed and out of reach (for certain vehicles) for the VISSIM evaluation area.

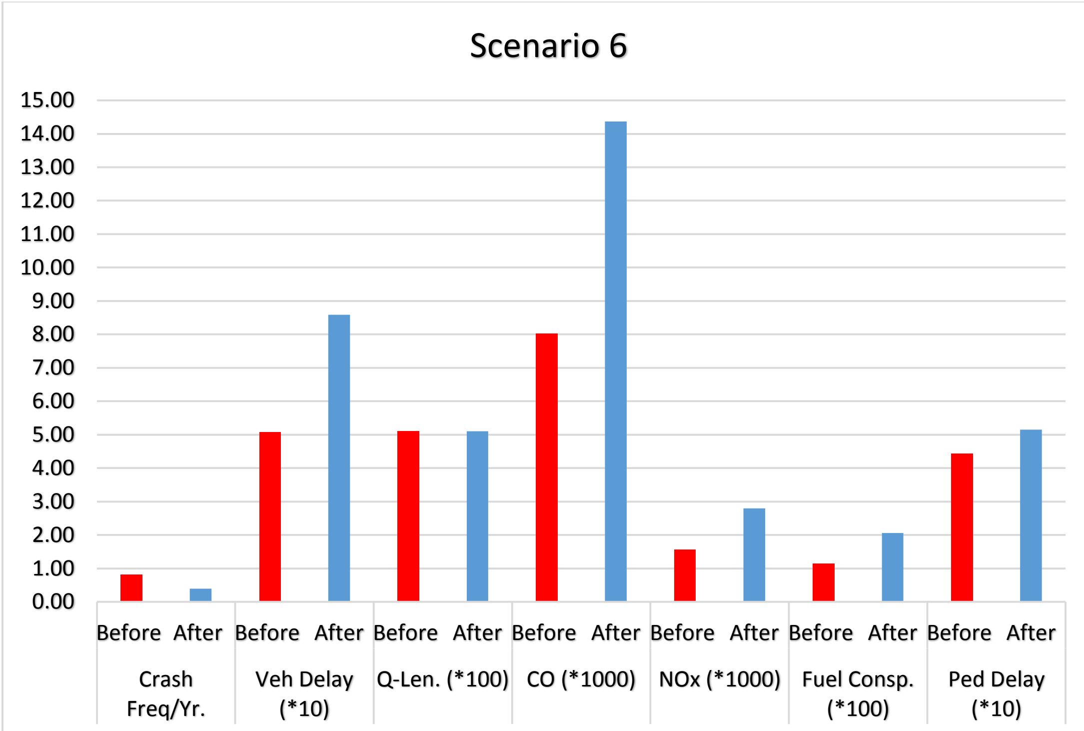


Figure 35: Before and After Differences at W 96th St & West End Ave Intersection with 120% AADT

Cost-Benefit Analysis

Understanding the operational costs and safety benefits of a countermeasure is an important aspect to consider before implementing any changes. The intent of the following section is to discuss the economic analysis of adding an exclusive pedestrian phase (33 second) to the regular signal timing. Chapter 3 of this thesis described the methodology and approaches used to determine crash and operational costs for the cost-benefit calculations. Cost-benefit analysis at varying traffic volumes and number of crashes observed will help States and local agencies determining if implementing the Barnes Dance will be economically beneficial or not.

Operation data were analyzed for six different traffic volume scenarios. Data was collected, considering both before and after implementing the countermeasure, in order to determine the impact of the treatment on operational performance. In terms of safety, The CMF developed for this countermeasure was used to calculate the amount of savings resulting from a reduction in crashes. By applying factors to the base number of crashes, the cost and benefit were observed for varying amounts of crashes, which were crash factors multiplied by the base number of crashes. Operation cost and crash savings due to the implementation of exclusive pedestrian phase for the six scenarios were combined in order to show the cost and benefit trends after the treatment.

Figure (36) shows the comparison between crash saving and overall operational cost, considering an increase in the number of observed crashes. An exclusive pedestrian phase implemented at the intersection of W 96th St & West End Ave. reduced crash frequency by 0.384 crashes per year at the base condition (actual AADT). This resulted in a savings of $7,673 per year. The total delay cost at the intersection, however, was $894,990 per year and $ 896,970 per year for operation cost, including delay, and CO & NOx emission costs.

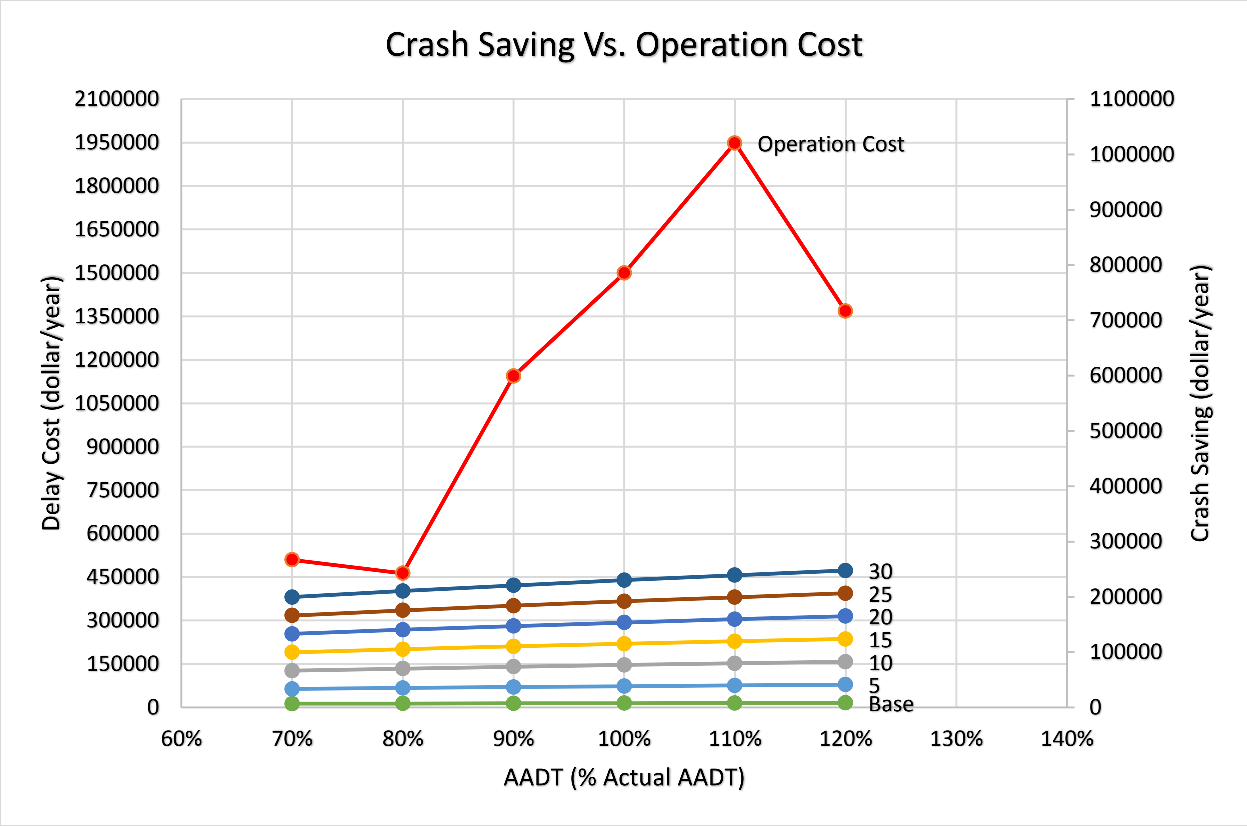


Figure 36: Crash Saving and Operational Cost Comparison for W 96th St & West End Ave Intersection

The treatment, therefore, resulted in a loss of approximately $889,296 per year at the selected intersection only due to the treatment. Additionally, the other scenarios considered showed the same cost-benefit trend. An increase in AADT by 10 percent led to a significant increase in loss of $1,157,986 per year. Other higher observed crashes scenarios showed higher crash saving due to the treatment but still lower than operation cost (no benefit). Therefore implementing an exclusive pedestrian phase led to higher cost than benefit at all scenarios. Table (27) summarizes cost-benefit analysis for all scenarios at different crash observations.

Table 27: Safety Benefit (Saving) and Operational Cost of all Scenarios for W 96th St & West End Ave Intersection

Scenarios		**Crash Factor**						
		Base (1)	**5**	**10**	**15**	**20**	**25**	**30**
70% * AADT	**No. of Crashes**	0.65	3.26	6.52	9.78	13.05	16.31	19.57
	Benefit/Cost	-297417	-270805	-237541	-204276	-171012	-137747	-104483
80% * AADT	**Crashes**	0.69	3.44	6.88	10.32	13.76	17.20	20.64
	Benefit/Cost	-269354	-241283	-206193	-171104	-136014	-100925	-65836
90% * AADT	**No. of Crashes**	0.72	3.61	7.21	10.82	14.43	18.03	21.64
	Benefit/Cost	-676372	-646947	-610165	-573382	-536600	-499818	-463036
100% * AADT	**No. of Crashes**	0.75	3.76	7.52	11.28	15.05	18.81	22.57
	Benefit/Cost	-889296	-858604	-820239	-781873	-743508	-705142	-666777
110% * AADT	**No. of Crashes**	0.78	3.91	7.82	11.72	15.63	19.54	23.45
	Benefit/Cost	-1157986	-1126101	-1086245	-1046388	-1006532	-966675	-926819
120% * AADT	**No. of Crashes**	0.81	4.05	8.09	12.14	16.18	20.23	24.28
	Benefit/Cost	-809960	-776945	-735677	-694409	-653141	-611873	-570605

Changing Pedestrian Volumes (Cost-Benefit Analysis)

The intent of the following section is to discuss the economic analysis and results of implementing the exclusive pedestrian phase (33 second) at intersections with different daily pedestrian volumes. Note that vehicle traffic volume (AADT) is assumed to be constant (actual AADT) for all scenarios. Cost-benefit analysis at varying pedestrian volumes and number of crashes observed will help States and local agencies determining if implementing the Barnes Dance will be economically beneficial or not.

Operation data were analyzed for seven different pedestrian volume scenarios; 700, 1500, 3200, 6500, 10000, 20000, and 30000 pedestrian per day. Data was collected considering both before and after implementing the countermeasure. Once again, by applying factors to the base number of crashes, the cost and benefit were observed for varying amounts of crashes, which were 3, 6, 9, 12, and 15 times the base number of crashes.

Figure (37) compares the operational cost and crash savings as an increase in the number of observed crashes is considered. An exclusive pedestrian phase implemented at the intersection of W 96th St & West End Ave. with pedestrian volume of 30000 ped/day reduced crash frequency by 1.477 crashes per year. This resulted in a saving of $29,542 per year. The total delay cost at the intersection, however, was $912,577 per year and $914,681 per year for operation cost, including delay, and CO & NOx emission costs.

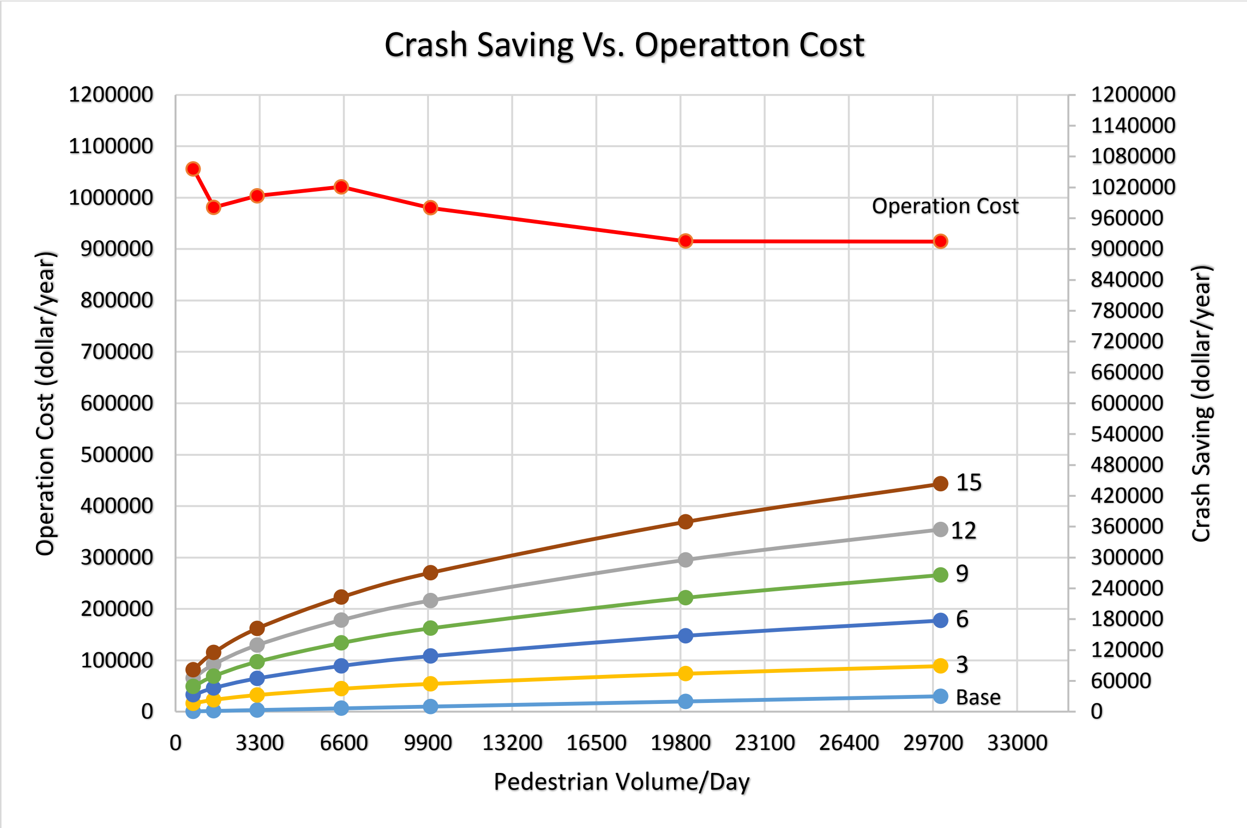


Figure 37: Crash Saving and Operational Cost Comparison for W 96th St & West End Ave Intersection

The treatment, therefore, resulted in a loss of approximately $885,139 per year at the selected intersection only due to the treatment. Additionally, the other scenarios considered showed the same cost-benefit trend. Other higher observed crashes scenarios showed higher crash saving due to the treatment but still lower than operation cost (no benefit). Therefore, implementing an exclusive pedestrian phase led to higher cost than benefit at all scenarios.

CHAPTER 5

CONCLUSION, RECOMMENDATION, AND LIMITATION

This study aimed to address the safety and operational effectiveness of three countermeasures: minor approaches left-turn changing phase from permitted-to- protected, leading pedestrian interval (LPI), and exclusive pedestrian phase (Barnes Dance). General Safety Performance Function (SPFs), acquired from the Highway Safety Manual (HSM), were used to calculate the average number of crashes for all crash types; these values were set as the base case. Crash Modification Factors (CMFs) available in Crash Modification Factor Clearinghouse for these countermeasures were used to determine the savings resulting from the reduction in crashes due to traffic crash treatment.

Seven scenarios with varying traffic volume were evaluated for both safety and operational performance. The base scenario used the actual Annual Average Traffic Volume (AADT); other scenarios are considered to be using varying volumes calculated by multiplying the actual AADT by 0.7, 0.8, 0.9, 1.1, 1.2, and 1.3, respectively, for each of the scenarios.

This thesis analyzed impacts of the three crash countermeasures and provides in which condition these countermeasures are cost-effective by computing both saving from crash reduction and additional costs due to changes in intersection operations. Again, the SPFs were used to calculate the average number of crashes. Therefore, by applying factors to the base number of crashes, the cost and benefit were observed for varying number of crashes, which were crash factor multiplied by the base number of crashes. The study used a range of average daily traffic values in order to provide a general guideline to help decision makers when determining cost-effective countermeasures.

The three countermeasures were all different from each other. However, the results always showed higher operational costs than crash savings at the base condition. Changing the left-turn phase on minor approaches from permitted-to-protected showed high reduction in crashes; however, higher traffic delay and emission can be seen after the treatment. A decrease in AADT led to less delay cost, but was offset by lower crash savings. Simultaneously, an increase in AADT led to a significant increase in operation cost per year. On the other hand, the cost-benefit analysis of the treated intersection showed that crash saving overtakes operational cost if the countermeasure is implemented at intersections with crash frequency higher than 6 crashes per year. And the highest benefit ($121,390 per year) can be obtained when the treatment is implemented at intersections having 110% of actual AADT with number of observed crashes 18.93 crash per year.

Leading Pedestrian Interval (LPI) was another countermeasure evaluated. The results also showed an increase in traffic delay and emission, especially if the treatment implemented at intersections with higher AADT. The cost-benefit analysis after the treatment showed that LPI treatment is economically beneficial only at intersections with crash frequency higher than 9 crashes per year. And the highest benefit ($49,406 per year) can be obtained when the treatment is implemented at the actual AADT with number of observed crashes 18.77 crash per year.

The last evaluated countermeasure was the Barnes Dance (i.e. exclusive pedestrian interval). Two main variables were considered in this countermeasure at the treated intersections: AADT and pedestrian volume. The results showed that there is a significant impact on the capacity of the road after implementing the Barnes Dance. Much higher vehicle delay and queue length are experienced at intersections with exclusive pedestrian phase. Also, the results show that the extra phase would increase pedestrian delay. The cost-benefit analysis after the treatment showed that the cost of implementing Barnes Dance is much higher than the benefit.

Therefore, it is recommended that agencies consider operational cost at the treated intersections. Traffic delay and emission should be considered simultaneously when the engineering countermeasures are implemented. This thesis also recommends implementing the countermeasures at intersections with high crash frequency. Implementing the countermeasures at intersections with high crash frequency can save money over the long run. Future researches can focus on the evaluation of operational cost and effectiveness of other engineering countermeasures along with providing locations of implementation.

This study has some limitations, leaving them topics for future research. First, the SPFs, acquired from the Highway Safety Manual, should be calibrated for the local condition for more accurate crash predictions. However, because no crash data was available for the treated intersections, calibration was only considered for the design features and signal characteristics. Moreover, Crash Modification Factors (CMFs), acquired from the Crash Modification Factor Clearinghouse, were estimated for all crash severities rather than defining the actual severity at the treated intersections. Therefore, because the severity of the crash could not be determined, an underestimate for the crash cost may exist. Another limitation is the overestimation of the operational analysis. For example, after implementing the countermeasure, such as the Barnes Dance, drivers may change their route to reach the destination, due to driver preference. Therefore, a change in driver behavior would be seen and would have an effect on the analysis and result. Operational evaluation has been done for isolated intersections rather than the whole network, in effect simulating only one route from the origin to the destination.

BIBLIOGRAPHY

Abou-Senna, H., Radwan, E., Westerlund, K., & Cooper, C. D. (2013). Using a traffic simulation model (VISSIM) with an emissions model (MOVES) to predict emissions from vehicles on a limited-access highway. *Journal of the Air & Waste Management Association*, *63*(7), 819-831.

Akgungor, A. P., & Bullen, A. G. R. (1999, February). Analytical delay models for signalized intersections. In 69th ITE Annual Meeting, Nevada, USA.

Ban, X., Herring, R., Hao, P., & Bayen, A. (2009). Delay pattern estimation for signalized intersections using sampled travel times. Transportation Research Record: Journal of the Transportation Research Board, (2130), 109-119.

Campbell, D. T., & Stanley, J. C. (1963). Experimental and quasi-experimental designs for research on teaching. American Educational Research Association.

Chen, L., Chen, C., & Ewing, R. (2012). The relative effectiveness of pedestrian safety countermeasures at urban intersections—Lessons from a New York City experience. In Transportation Research Board (TRB) 91st Annual Meeting, Washington, DC.

Davis, G., & Aul, N. (2007). Safety Effects of Left-Turn Phasing Schemes at High-Speed.

De Vlieger, I. (1997). On board emission and fuel consumption measurement campaign on petrol-driven passenger cars. *Atmospheric Environment,31*(22), 3753-3761.

Dion, F., Rakha, H., & Kang, Y. S. (2004). Comparison of delay estimates at under-saturated and over-saturated pre-timed signalized intersections. Transportation Research Part B: Methodological, 38(2), 99-122.

Donham, K. J., Leistikow, B., Merchant, J., & Leonard, S. (1990). Assessment of US poultry worker respiratory risks. *American journal of industrial medicine*, *17*(1), 73-74.

Edara, P., Sun, C., & Breslow, S. (2014). Evaluation of J-turn Intersection Design Performance in Missouri. Missouri Department of Transportation.Chu, H. C., & Meyer, M. D. (2009). Methodology for assessing emission reduction of truck-only toll lanes. Energy Policy, 37(8), 3287-3294.

EPA, Fact Sheet Proposed Revisions to the National Ambient Air Quality standards For Nitrogen Dioxide.

Fayish, A., & Gross, F. (2010). Safety effectiveness of leading pedestrian intervals evaluated by a before-after study with comparison groups. Transportation Research Record: Journal of the Transportation Research Board, (2198), 15-22.

Ghasemlou, K., Aydın, M. M., & Yıldırım, M. S. (2015). Comparison of delay time models for over-saturated traffic flow conditions at signalized intersections. International journal of advanced science and technology, 84, 9-18.

Hallmark, S. L., Guensler, R., & Fomunung, I. (2002). Characterizing on-road variables that affect passenger vehicle modal operation. *Transportation Research Part D: Transport and Environment*, *7*(2), 81-98.

Highway Safety Manual (HSM), (2009). AASHTO. *Washington, DC*, *529*.

Hummer, J. E., Haley, R. L., Ott, S. E., Foyle, R. S., & Cunningham, C. M. (2010). *Superstreet Benefits and Capacities* (No. FHWA/NC/2009-06).

Inman, V. W., & Haas, R. P. (2012). *Field evaluation of a restricted crossing U-turn intersection* (No. FHWA-HRT-11-067).

Knoop, V. L. (2009). Road Incidents and Network Dynamics: Effects on driving behaviour and traffic congestion. TU Delft, Delft University of Technology.

Kostyniuk, L. P., Molnar, L. J., St Louis, R. M., Zanier, N., & Eby, D. W. (2011). Societal costs of traffic crashes and crime in Michigan: 2011 update.

Mousa, R. M. (2002). Analysis and modeling of measured delays at isolated signalized intersections. Journal of transportation engineering, 128(4), 347-354.

Nesamani, K. S., Chu, L., McNally, M. G., & Jayakrishnan, R. (2007). Estimation of vehicular emissions by capturing traffic variations.*Atmospheric Environment*, *41*(14), 2996-3008.

Olszewski, P. (1993). Overall delay, stopped delay, and stops at signalized intersections. Journal of Transportation Engineering, 119(6), 835-852.

Plan, A. S. H. S. (1998). American Association of State Highway and Transportation Officials. *Washington, DC*.

Ries, F.J., Hystad, P. & Gouge, B. (2010). Minimizing children's nonresidential exposure to traffic-related pollution." National Collaborating Centre for Environmental Health.

Tian, Z., Urbanik, T., Engelbrecht, R., & Balke, K. (2002). Variations in capacity and delay estimates from microscopic traffic simulation models. Transportation Research Record: Journal of the Transportation Research Board, (1802), 23-31.

U.S Environmental Protection Agency. National Ambient Air Quality Standards (NAAQS). Available at: http://www.epa.gov/air/criteria.html (Accessed July 27, 2015).

Xi, J., Li, W., Wang, S., & Wang, C. (2015). An Approach to an Intersection Traffic Delay Study Based on Shift-Share Analysis. Information, 6(2), 246-257.

Xu, X. H., Ge, Z. Y., & Gao, C. (2013, December). Analysis Research of Delay of Signalized Intersection Based on VISSIM Simulation Software. In Applied Mechanics and Materials (Vol. 409, pp. 1081-1084).