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Traffic Concurrency Management Through Delay and Safety Mitigations

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TRAFFIC CONCURRENCY MANAGEMENT THROUGH DELAY AND SAFETY MITIGATIONS

By

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Travelers experience different transportation-related problems on roadways ranging from congestion, delay, and crashes, which are partially due to growing background traffic and traffic generated by new developments. With regards to congestion, metropolitan planning organizations (MPOs) pursue a variety of plans for mitigating congestion. These plans include, amongst other measures, imposing impact fees. The current research evaluates how delay and safety can be incorporated in the mitigation process as special impact fees. This study also evaluates traffic projection methodologies used in traffic impact studies. Traffic volume is a critical factor in determining both current and future desired and undesired highway operations. Highway crashes are also influenced by traffic volume, as a higher frequency of crashes is expected at more congested locations and vice versa. Accurately forecasted traffic data is required for accurate future planning, traffic operations, safety evaluation, and countermeasures. Adhering to the importance of accurate traffic projection, this study introduces a simplistic traffic projection methodology for small-scale projection utilizing three parameters logistic function as a forecasting tool. Three parameters logistic function produced more accurate future traffic prediction compared to other functions. When

validation studies were performed, the coefficient of correlation was found to be above 90 percent in each location. The t-values for the three parameters were highly significant in the projection. The confidence intervals have been calculated at a 95 percent confidence level using the delta method to address the uncertainty and reliability factor in the projection using logistic function.

A delay mitigation fee resulting from increases in travel time is also analyzed in this research. In regular traffic flow, posted speed limit is the base of measuring travel time within the segment of the road. The economic concept of congestion pricing is used to evaluate the impact of this travel time delay per unit trip. If the relationship between the increase in time and trip is known, then the developer can be charged for the costs of time delays for travelers by using that relationship. The congestion pricing approach determines the average and marginal effect of the travel time. With the known values of time, vehicle occupancy, and number of travel days per year, the extra cost per trip caused by additional trips is estimated. This cost becomes part of the mitigation fee that the developer incurs as a result of travel time delays for the travelers due to the development project. Using the Bureau of Public Road (BPR) travel time function and parameters found in 2000 HCM (Highway Capacity Manual), the average and marginal travel times were determined. The value of time was taken as $7.50 per hour after reviewing different publications, which relate it to minimum wage. The vehicle occupancy is assumed as 1.2 persons per vehicle. Other assumptions include 261 working days per year and 4 percent rate of return. The total delay impact fee will depend on the

number of years needed for the development to have effect. Since the developer is charged a road impact fee due to constructions cost for the road improvement, the delay mitigation fee should be credited to the road impact fee to avoid double charging the developer.

As an approach to incorporate safety into mitigation fees, the study developed a crash prediction model in which all factors significantly influencing crash occurrences are considered and modeled. Negative binomial (NB) is selected as the best crash modeling distribution among other generalized linear models. The developed safety component of the mitigation fee equation considers scenarios in which the proposed new development is expected to increase crash frequency. The mitigation fee equation is designed to incorporate some roadway features and traffic characteristics generated by the new development that influence crash occurrence. Crash reduction factors are introduced and incorporated in the safety mitigation fees equation. The difference between crash frequency before and after the development is multiplied by the crash cost then divided by the trips to obtain crash cost per trip. Crash cost is taken as $28,000/crash based on literature review. To avoid double charging the developer, either the road impact fee is applied as a credit to the delay mitigation fee or vice versa. In summary, this study achieved and contributed the following to researchers and practitioners:

- Developed logistic function as a simplified approach for traffic projection
- Developed crash model for crash prediction
- Developed safety mitigation fee equation utilizing the crash modeling
- Developed delay mitigation fee equation using congestion pricing approach

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EXECUTIVE SUMMARY

Chapter 1

Chapter one provides an introduction to the subjects and materials covered in this study. Possible areas where planning and safety can be incorporated together for the purpose of improving capacity and crash reduction, the weaknesses of the current practices related to safety-planning integrations, and common procedures used for traffic impact study are discussed in this chapter. The chapter highlights the core nine-step procedures necessary to follow before making a conclusion on if the new development impacts the existing roadway network or not. These steps include determining study methodology, analyzing existing conditions for the roadways within the impacted network, and gathering and forecasting background traffic. The project trips are calculated through trip generation followed by trip distribution, model split, and trip assignment. With all trips assigned to the appropriate links and intersections, future condition analysis is performed. It is at this step where proposed improvement is laid out. Mitigation is applied if the analysis finds the need for improvement of the transportation facility as a result of the impact from the new development.

Chapter 2

Chapter two is the literature review, discussing what other researchers have done with respect to the objectives of this study. The literature review has shown that different cities and counties use different equations, though the same parameters, to calculate the road impact fee. The findings from the literature highlighted some of the common variables

used in the impact fee calculation equation, mainly trip rates and vehicle miles of travel. The credits in terms of gas tax revenue are deducted from the current road impact fee. The current impact fee equations do not contain any delay or safety mitigation related parameters. With respect to crash occurrences, the literature has shown some roadway and traffic variables generated by new developments to be the initial contributing causes of certain kinds of crashes. These variables are incorporated in the crash prediction model developed in chapter four. The significant variables influencing crash occurrence found in the literature include driveway density, number of trips (traffic), and median type, number of lanes, median width, shoulder width, lane width, directional splits, and vehicle miles of travel. This chapter further discusses how safety and planning can be integrated together. The study has found that safety programs can be included in planning when establishing transportation priority programs. In fact, the findings show inclusion of safety in planning programs will lead to effective performance measures to reduce crashes and improve operations. Through the evaluation and analysis process, planners determine how the system is performing, and what changes in the transportation system will be needed to improve safety. To ensure that safety becomes an integrated part of the plan, incorporating safety into the transportation planning goals and objectives is important. Through safety evaluation, crash patterns, frequency, and high location areas are identified. The statistical analysis is also used to support the observed crash data through descriptive statistics, significance testing, and charts and graphs for presentation. Collision diagrams should be used to trace the crashes with respect to highway locations where they occur. The trends from the collision diagrams will help identify safety

vulnerable points on the highway. Possible crash contributing causes and counter measures should be used as a starting point for crash reduction in the planning process. Both safety and planning engineers should use the results of safety evaluation into their short- and long-range planning.

Chapter 3

Chapter three evaluates the use of different functions for traffic projection at locations where forecasting models and reliable growth rates are not available. Evaluation of different distributions is done as an approach to find the suitable simplified traffic forecasting methodology. Projected traffic determines type and magnitude of improvement to be proposed on the highway; hence, it is an important element in highway design, planning, and safety mitigation. The accurately projected traffic will lead to a better and more appropriately designed transportation facility, while inaccurate projected traffic may lead to an under or over-designed transportation facility. For instance, currently, the Florida Department of Transportation (FDOT) recommends the use of linear regression to project traffic at the locations when a forecasting model is not available. The use of linear regression has some weaknesses including lack of normal traffic growth patterns and inability to limit projection to the desired capacity. Logistic function is found to be more applicable due to its S-shape pattern, which conveys traffic growth pattern, and its ability to limit growth to certain levels, which can be treated as the capacity. The data ranging from 1970 to 2007 at 12 different locations in Palm Beach County, FL, are used to build and validate logistic function. Using optimization tools in

matlab, the program was coded and run, which provided the coefficients of the function variables and corresponding t-values. Since the logistic function developed has three parameters, the t-value for each parameter has been calculated and shows strong significance in the projection. With the fact that the projection process can be affected by reliability and uncertainties due to different factors, the confidence intervals have been calculated utilizing delta method. The existing, fitted, and validated data are compared, and the outputs showed realistic traffic growth with strong correlation coefficients.

Chapter 4

In chapter four, the crash data is modeled using various highway and traffic related variables. Negative binomial (NB) distribution is used to model the crashes because of its capability to model count data and to pass distribution tests for the available crash data. Poisson and zero inflated distributions didn't fit the data after being tested. From the NB model outcome, it was found that ADT, directional split, number of lanes, lane width, median width, shoulder width, and median type indicator are the most significant in crash prediction. The developed crash prediction model is used in developing the safety mitigation fee equation in chapter five.

Chapter 5

Chapter five develops safety and delay mitigation fees in excess of the currently utilized road impact fee. The crash prediction model developed in chapter four and congestion pricing approach are utilized in the safety and delay fee calculations, respectively. The

safety mitigation fee considers crash frequency before and after the development, accident reduction factors, crash costs, and number of project units. The delay mitigation fee considers the travel time delay developed from total and marginal travel time, value of time, vehicle occupancy, and number of travel days in the year. As the approach to countermeasure the impact of increased travel time delay, Stochastic User Equilibrium (SUE) traffic assignment is utilized to assign traffic to the alternate parallel routes. Illustrative numerical examples for road impact fee, safety mitigation fee, and delay mitigation fees are analyzed to elaborate the developed methodologies. Furthermore, this chapter combines safety and delay mitigation fees as a special fee considering both crash and delay reduction.

Chapter 6

Chapter six gives the conclusion and recommendations for future study.

CHAPTER 1

INTRODUCTION

1.1 Overview

There are various steps and analysis used in traffic impact studies. Some of the crucial steps include traffic forecasting and mitigation. Figure 1.1 details the common procedures followed up to the mitigation process. As shown in the figure, traffic projection and what is to be covered in the mitigation are still non conclusive and need more research to be accomplished. The layout of Figure 1.1 forms the skeleton of this study by introducing the simplified approach for traffic projection and special fee in excess to currently practiced road impact fee.

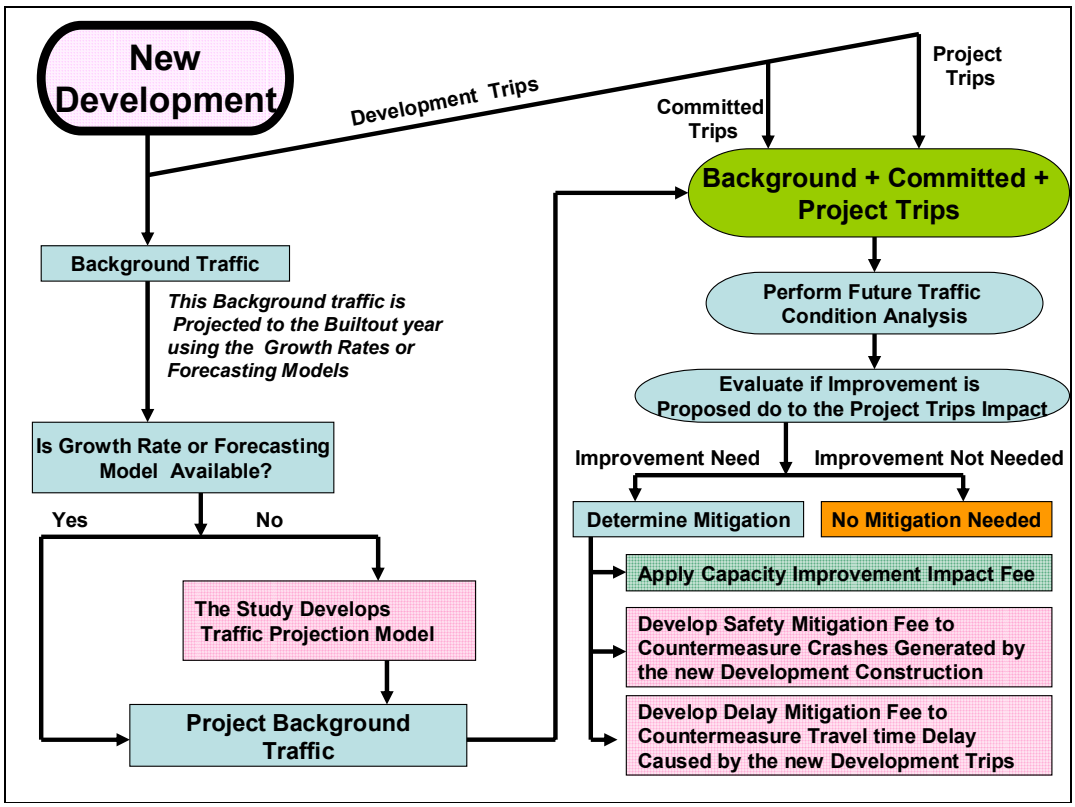


Figure 1.1: Flow chart of the study

1.2 Impact Study Development

Traffic impact study guidelines has been developed at National, State, County and City levels, with each level having some of their own methodologies to conduct the study. While the State can have generalized methodology for traffic impact study, it allows the Counties and Cities to modify the methodologies to match local conditions. For instance, Florida Department of Transportation (FDOT) has created eleven steps for traffic impact study which ranges from the methodology to the permit issuance for any development. These steps are:

- Methodology Development
- Existing Condition Analysis
- Background Traffic
- Trip generation
- Trip Distribution
- Model Split
- Assignment
- Future Condition Analysis
- Mitigation Analysis
- Site Access, Circulation, Parking, Review and Permitting

1.2.1 Methodology Development

The methodology development refers to the initial stage in which all steps and a procedure to be used in traffic impact study is discussed. In this stage formal procedures,

default values, design years, phases and all issues to be used in the study are discussed and agreed. The formal methodology processes are those practiced by the participating agencies agreed in advance prior to performance a site impact analysis.

1.2.2 Existing Conditions Analysis

Existing condition analysis is the step after the methodology has been set up. In this stage, tasks like data collection and existing corridor geometric and traffic are evaluated. Site or corridor location, boundaries and all proposed developments are identified at this stage. All transportation system networks, transit services, pedestrian and bicycle facilities, traffic control data like signals, phase and signing are all identified and documented. The step also gathers social economic and demographic data for potential use in future analysis. In summary, all relevant existing information are collected and gathered at this stage of study. After all existing data has been gathered, existing condition analysis is performed to determine the current capacity in terms of level of service of the facility to be affected by the development. The capacity analysis is performed along each critical intersection and segment of the roadway system identified in the methodology step. The critical intersection and segments are those within specified radius of influence which depends on the size of trips generated by the project. The usual radii of influence are 0.5 miles, 1 mile, 2 miles, 3 miles, 4 miles and 5 miles, depending on the size of the development. The delays, level of service, queue lengths and other operational values for existing condition are extracted from this analysis. The capacity

analysis approach is discussed in methodology stage; usually the latest highway capacity manual (HCM) methodology is used.

1.2.3 Background Traffic

Background traffic are those not generated by the development. They are the traffic counted at the site defined locations then projected to the design or buildout year using provided or developed traffic growth rates. The design year, is the expected buildout of the stated phases of the project. The project trips are added to the background traffic for future analysis. The projection of the traffic can be through manual calculation or through computer based. In case of manual projection, the growth rate has to be developed for projecting the traffic. The future analysis is initially performed using projected background traffic before project trips are added. The results obtained from the analysis with project trips included are compared to the analysis using background traffic only. This gives a way to compare of the effect of the project trips to the transportation facility.

1.2.4 Trip Generation

Trip generation refers to the process of predicting number of vehicle trips to be generated by the proposed development. The most popular method of estimating trips from the development is by using the rates and equations generated by Institute of Transportation Engineer (ITE) through ITE *Trip Generation Handbook*. The equations and rates in this manual are nationwide accepted for trip generations. The first step in trip generation is identification of the type of the proposed land use. Different land uses has different

codes, differentiated with general use of the intensity of the unit. The next step is to identify the size of the proposed development in terms of its units. For instance number of dwelling units for residential, number of rooms for the hotel, square feet for office or retail or industrial, number of beds for hospital, number of holes for golf course etc. These units are inserted in the trip equations or multiplied by the rates to obtain the total trips to be generated by the project. For large size projects, pass-by trips and internal captures are calculated and deducted from the total trips. Pass-by-trips are the intermediate stops to the development due to attraction but their origin and destination are not from the development. Internal captures are the trips which will originate and end within the development.

1.2.5 Trip Distribution

Trip distribution is the step in which the trips generated are distributed to the links impacted by the development. The main determinant of trip distribution is the production and attraction between the development and the surrounding areas. Major links are distributed with trips depending on how they are expected to impact the development. In other words, the link close to the development and which will be used mostly to and from the development site will be having higher percentage of distribution, while those far from the site will have small percentage of distribution. In all cases, the distribution at the node (at the intersection) must balance; meaning the sum of entering percentage must be closely equal to the leaving percentage. Though there are several methodologies used to

calculate the trip distributions like gravity model, ITE Trip Generation Manual has percentage distribution for trips entering and leaving the site.

1.2.6 Model Split

In model split step, different modes of transportation between the development and surrounding areas are estimated. Model split is only estimated for the developments where different modes of transportation are expected to be used. Such kinds of development include development of Regional Impact (DRI), but in small mall developments, model split is not a significant factor.

1.2.7 Trip Assignment

The trips distributed to the major links are further assigned to the destinations at the trip assignment step. Sometimes, trip distribution is done simultaneously with trip assignment. Turning movements at the intersection for the project trips is one example of trip assignment. Trip assignment is an important step in traffic impact study in the sense that, if trips are not accurately assigned then the analysis results will not be accurate and may lead to inaccurate improvement recommendations. Traffic impact reviewers concentrate much to make sure traffic distributions and assignments are correct before approving the recommendation.

1.2.8 Future Condition Analysis

The trip assignment finalizes the whole process of preparing traffic volumes for future condition analysis. Future analysis determines the impact of the development for the design or buildout year. The significance of the development impact is evaluated in terms of delays, level of service (LOS) and queue lengths. All traffic related factors are considered during future condition analysis. These factors like roadway and human characteristics used as input to LOS analysis are to be according to known methodology like those in highway capacity manual (HCM). After condition analysis, the study identifies deficiencies and proposes improvements needed for the section of the roadway impacted. Addition of number of lanes, lengthening the storage lengths, signal re-timing are some of the common recommendations proposed for improvement. Furthermore, interaction of various elements for proper site access, circulation and parking design on the safety and operations of the adjacent streets and roadway are always part of proposed improvements. These improvement recommendations are only made if the development adversely impacts the roadways. According to Florida Site Impact Handbook (30), developments are considered significantly impacted roadway if:

(i) Level of service on the roadway with the development trips is below the adopted threshold.

(ii) The roadway is currently constrained that cannot expand due to physical policy or other limitations.

(iii) The roadway currently operating below LOS standard and not programmed for improvement within 3-years.

1.2.9 Mitigation

Mitigation step in traffic impact study arise when the roadway is significantly impacted by the development exceeding the threshold level of service. When the analysis indicates the transportation system will operate at a desirable LOS in the development area of influence, no improvements are likely to be required. However, if the development results in undesirable LOS, improvements are evaluated. The site impact analysis determines the deficiency resulted from the traffic added by the proposed development. The examples of mitigation measure may include construction of new road facility, addition of new lanes, improvement of existing transportation management system, improvement of access management of the impacted facilities, site plan or land use changes and so on.

The methodology for determining the developer's fair-share funding of mitigation improvements is identified in the methodology phase of the project. The fair-share is determined in relationship to the number of trips generated by the development and the capacities on an affected roadway segment. The mitigation fee considered is typically negotiated among the applicant and the jurisdiction following the analysis that demonstrates the proposed improvements resulting in an acceptable operating condition along the impacted facility. In some jurisdictions, for smaller developments that are within a concurrency management area, the developer's share of mitigation improvements is an impact fee that is assessed using a predetermined schedule of fees based on the intensity and type of land use. For example, each unit of single-family

detached housing will be associated with a fixed fee. This fee is applied throughout the concurrency management area and reflects the proportional share of improvements required on the area's concurrency management system of roadways.

The concurrency and traffic impact fees can be paid by the developer as a fair share contribution based on the agreed legislation relating to proportionate share mitigation at that particular County. In many cases, the proposed road impact fee methodology is based on a demand and consumption model, which basically charges a new development the cost of replacing the capacity that it consumes on the major roadway system. Currently, in many jurisdictions, the generated trips, the vehicle mile of travel (VMT) expected for the project trips, percentage of non-pass-by trips, cost of construction of one lane per mile and the available credits due to the fuel tax revenue are the major components in calculating the impact fee. That is, for every vehicle-mile of travel (VMT) generated by the development, the road impact fee charges the net cost to construct an additional vehicle-mile of capacity (VMC) subtracting the possible credits mainly due to fuel revenue. While the existing methodologies consider only a few parameters in calculating road impact fee, there are also contradicting assessment of the methodology. According to the memorandum prepared by Center for Urban Transportation Research (CUTR) for FDOT, different issues has been associated with impact fee and proportionate share process in Florida. The issues highlighted in the memorandum included:

- In some areas the traffic impact fee are heavily credited and discounted hence not worthy for the real cost of expected improvement
- In some areas the impact fee is not a mandatory though the concurrency is supposed to be maintained causing shortage of fund for the improvement
- Different procedures and methodologies are used to come out with the impact fee and proportionate share, causing contributions not to designate the intended concurrency
- In some areas, high sized developments are charged higher for per trip cost compared to relatively smaller developments
- The most current developments are charged significantly higher compared to earlier developments because the added trips impact more links
- The developments introduced at locations with existing investments relatively pay less compared to location with few or no existing development
- In some scenarios developers are charged twice for the same development due to double impact

1.3 Traffic Projection

In traffic impact study or any transportation traffic analysis, future projection is an important step. There are different methodologies which are used for traffic projection. The use of forecasting models is the most reliable and widely used methodology for large scale traffic projection. But these models are not always available or their uses can be limited to certain location or particular roadways. In the absence of these models,

different approaches have been proposed for projection. The common methodologies recommended by FDOT are the use of trend or regression analysis. In the regression, the commonly used models include linear, geometric and declining growth. The use of these approaches utilize demographic characteristic, such as population or employment for the study area. Generally a trend analysis is used where sufficient traffic count data are available to establish a trend for each facility segment in the study area or for area wide traffic growth. In Florida for instance, FDOT recommends data for the last five years as a minimum to provide a basis for statistically relevant analysis. Though trend analysis using simple linear regression is recommended but it is highlighted as not appropriate for long-range projections. Based on deficiency in the traffic projection using linear regression approach, this study evaluates different type of functions for traffic projection.

1.4 Crash Prediction Model

Traffic crashes result from the interaction of different parameters including highway geometrics, traffic characteristics and human factors. Geometric variables include number of lanes, lane width, median width, shoulder width, roadway length, and number of intersections, access density and shoulder width while traffic characteristics include AADT and speed. The effect of these parameters can be correlated by predictive models to predict crash rates or frequency at a particular roadway section. To utilize safety and planning in one combined or integrated program, it is important to evaluate how different road and traffic factors significantly influence crash occurrence. The crash prediction model is utilized in this study in which negative binomial model is developed for future

number of crash prediction. The outcome from the prediction model is used in the impact fee equation.

1.5 Travel Time Delay Cost—Use of Congestion Pricing

The concept of marginal travel time can be derived from the economic and finance where marginal cost is termed as the change in total cost that arises when the quantity produced changes by one unit. Knowing demand for travel across a network, one can iteratively solve the set of prices that equate marginal cost and marginal benefits on all links. In practice, demand functions are unknown, but can be by a trial-and-error implementation on a network without knowledge of demand functions but with known link performance functions, observed flows, and observed responses to pricing decisions (60). It can be expressed mathematically as the derivative of the total cost function with respect to the given quantity. With respect to transportation network, the marginal travel time differs from average travel time. This lies on the fact that the cost of producing an additional unit of a trip for instance may increase or decrease due to economies of scale, scope or density in the supply of the transport service. Marginal travel time typically increases with each additional unit of demand, as roads become more congested. Marginal cost with respect to travel time is a concept based on the fact that road users always try to save time indicating that time savings have value. The difference between marginal and actual or normal travel time gives the delay per unit trip made. Congestion pricing methodology is one of the best known approaches for analysing the concept of actual and marginal cost,

hence applied in this study to determine the delay cost caused by the developers on the highway link as a result of additional trips.

1.6 Chapter Summary

There are different procedures for traffic impact study. This chapter has highlighted the core nine step procedures up to mitigation process to be followed before making a conclusion if the new development has impact to the existing roadway network. The steps include study methodology, analyzing existing condition for the roadways within the impacted network and gathering background traffic. The next step is trip generation in which the number of trips expected from the new development is calculated, followed by trip distribution, model split and trip assignment. With all trips assigned to the appropriate links and intersections, future condition analysis is performed. It is at this future condition analysis where improvement is proposed. Mitigation analysis is done by calculating the fee the developer will pay for the proposed improvement. The impact of the new developments with respect to crash occurrence also has been discussed. It has been shown that, with known roadway variables which influence crash occurrence, crashes can be predicted through modeling and with known crash cost, the effect can be valued and included in the impact fee calculation. Delay caused by the new trips on the highway segments also has been discussed in the chapter. By using the congestion pricing methodology, the difference between the regular and marginal travel time can be determined. Knowing the value of time, vehicle occupancy and travel days per year, the delay cost per trip can be determined and included in the impact fee calculation. Finally,

the chapter also introduced simplified approaches for projection approaches for the use in traffic analysis. While the use of growth rates and complex forecasting models are the most common approaches in forecasting traffic, their availability and use are sometimes limited, hence not readily available for use. Furthermore, in some of the current practices, linear regression is used for projection though its linearity growth does not always reflect normal traffic growth.

CHAPTER 2

LITERATURE REVIEW

2.1 Overview

In many local governments, mitigation is part of transportation concurrency growth management intended to ensure that the necessary public facilities and services are available concurrent with the impacts of developments. Transportation concurrency, in other words, is the guideline laid out to ensure roads and intersections operate within desirable level of service for defined road class. To carry out concurrency, the state and local governments define what constitutes an adequate level of service for a particular road. The definitions of level of service are as shown below:

- LOS A……Low traffic density, very low delay, favorable progression
- LOS B……Minimum delay, good progression
- LOS C……Increased delay, fair progression
- LOS D……Long delays, unfavorable progression
- LOS E……High delay, poor progression
- LOS F……Traffic volumes exceed capacity and poor progression

These levels of service can be categorized by delay thresholds, traffic volume thresholds or speed thresholds. For instance, Highway Capacity Manual (HCM) has the following delay thresholds for intersection level of service.

- LOS A……$\leq$ 10.0 Seconds per Vehicle
- LOS B……>10.0 and $\leq$20.0 Seconds per Vehicle

- LOS C……>20.0 and ≤35.0 Seconds per Vehicle
- LOS D……>35.0 and ≤55.0 Seconds per Vehicle
- LOS E……>55.0 and ≤80.0 Seconds per Vehicle
- LOS F……>80.0 Seconds per Vehicle

Using the desirable level of service thresholds, the state and local governments can measure whether the new development along the segment of that particular road could be allowed, and what impact it will have based on the existing demand. Traffic impact study must be conducted to determine if there is a room to accommodate more traffic from the development. If adequate capacity is not available, then several steps are followed, either not to approve the development or allow it to be implemented but under certain conditions. Some of the conditions include allowing the road to operate beyond desired level of service under constrained conditions.

2.2 Concurrency Management

In order to understand the concept of transportation management concurrency and how it is conducted, the following are the literature review on how concurrency management is practiced in some Counties and Cities in Florida. The review considers only determination of level of service standards and methodologies.

In Palm Beach County, the transportation element of the comprehensive plan provides level of service standards for county and local roads excluding State roads. The transportation element requires all intersections to operate within LOS D or better at the peak hours by utilizing Highway Capacity Manual Planning Methodology (Critical

Movement Analysis, CMA) developed in1985. All links are supposed to operate at LOS D or better based on LOS thresholds found in FDOT Level of Service Handbook. The use of current HCM methodology for arterial analysis is also recommended in case FDOT Handbook is not adequate. For roads on the Florida Intrastate Highway System (FIHS), the level of service standard is Level of Service D, in urban areas, and Level of Service B, in rural areas. But the County has found that under certain limited circumstances dealing with transportation facilities, countervailing planning and public policy goals may come into conflict with the requirement that adequate public facilities be available concurrent with the impacts of such development. Under these circumstances, lower level of service standards for specific roadway segments and intersections are allowed. This policy provides for lower transportation facilities level of service standard for certain purposes on roadway segments and intersections. Schools and hospitals fall under this category in which segments and intersections and segments are allowed to operate 30% above the LOS D. Furthermore, developments are allowed in some corridors under constrained condition (CRALLS). But in depth data and analysis must be conducted before the segment declared the CRALLS designation (1).

In Broward County, transportation concurrency is defined under two scenarios. One is Standard Concurrency District and the second is Transit Oriented Concurrency District. The County is divided into 10 locally defined districts in which two of these districts maintain the existing standard roadway concurrency system while the remaining eight districts maintain transit oriented concurrency (2). For both standard and traffic oriented

concurrency, the developers are required to obtain a satisfaction certificate prior to applying for building permit. The level of service for transit oriented concurrency is based on achieving transit headway of 30 minutes or less on 90% of all routes together with establishing at least one community bus center and one additional community bus route. The level of service for standard concurrency districts is LOS D using Urbanized FDOT Table for FDOT most current level of service manual. The exception is for Interstate 75, from ½ mile west of Southwest 184 Avenue to Collier County Line and U.S. 27, from Interstate 75 to Palm Beach County line, which is supposed to operate at LOS B or better (2). The interesting part of Broward County transportation concurrency is the requirement that the proposed development must address the adequacy of the other roadways within the regional network to ensure they will also operate within LOS D, otherwise it must be included in the County Long Range Transportation Plan (3).

Sarasota County has a concurrency management plan in which County and city roads are required to maintain LOS C or better, while urban and suburban roads are required to operate at LOS D or better. State roads in rural areas and interstates are required to maintain LOS B or C. The County uses the most current version of HCM for level of service analysis together with established FDOT Standard level of service tables (4).

Like Palm Beach County, the city of Tallahassee has a concurrency management policy which allows some sections of the highways to operate beyond desired level of service under constrained conditions. In the concurrency manual, some sections of the roadway are allowed to operate at LOS E plus 50% if some conditions are met. For any new

development, concurrency review is conducted first to determine if there is capacity available for the new trips. For the roads with access limitations or major arterials, the capacity must be available before the initial construction of the proposed development. If the impacted facility do not have adequate capacity but improvements are scheduled that will provide necessary capacity to eliminate the existing deficiencies, then conditional approval of the project can be issued. In case there is no way to have extra capacity, the developer is requested to reduce the size of the proposed facility (1).

Miami Dade is one of the Counties with major metropolitan cities in Florida. The County long range plan is to have all roadways in the county to operate at LOS C or better by year 2010. Currently the transportation concurrency have the following policy quoted from the County Comprehensive Development Master Plan (CDMP), Transportation Element (5): The minimum acceptable peak period operating level of service for all State and County roads in Miami-Dade County outside of the Urban Development Boundary (UDB) identified in the Land Use Element shall be LOS D on State minor arterials and LOS C on all other State roads and on all County roads.

2.3 De Minis

There are different impacts levels created by developments, ranging from major impact to de minis. De minis, according to Florida Statute 163.3180(section 6), is defined as impact which or that affects only 1% or below of the maximum generalized level of service volume of the transportation facility impacted by the development. The general approach

for many local governments is to ignore the effect of de minis in the analysis but de minis effect should not the overall facility to operate above 110% of the desired LOS volume. In general, if de minis causes the volume to be 110% or more compared to generalized standard LOS volume, the Statute requires no further developments should be allowed on that corridor. According to the study done by CURT (3), different local governments in Florida define de minis in different ways. For instance, St. Johns County considers the single family detached dwelling unit as de minis, while Sarasota County uses a single family unit and non-residential units of up to 1500 square feet as de minis impacts. The city of Tallahassee (6) has significant thresholds used to determine when mitigation will be required. Project impacts that are less than the applicable significant thresholds are included along with other trips in determining available capacity. If the impact is too minimal then they are ignored.

2.4 Transportation Impact Fee

As defined previously, impact fee is a monetary charge imposed by local government on new development to recoup or offset a proportionate share of public capital facility costs required to accommodate such development with new facilities (8). Impact fees help to hold property taxes down by utilizing the developer to pay for infrastructure improvements. The impact fee is perceived to be an affective growth management tool for the local governments for infrastructure improvement and maintenance. Furthermore impact fee provides an opportunity for planners to negotiate with developers over the

provision of infrastructure. There are certain rules which generally guide the whole process, some of them as listed below:

- For impact fee to be charged there should exist a reasonable connection between the need for additional facilities and the growth resulting from new development.
- There should be a reasonable connection between the expenditure of the fees collected and the benefits received by the development paying the fees.
- The fees charged must not exceed a proportionate-share of the cost incurred or to be incurred in accommodating the development
- Development must benefit from facilities financed by impact fees.
- Fees should be earmarked to finance only facilities that benefit contributing development.
- Fees should be expended within a zone or segment where a development is located.

On the other side, despite their increasing popularity**,** impact fee charges have been associated with some problems including;

- It has been found that the impact fess increase the cost of housing, hence have negative effects upon housing affordability.
- In some areas impact fees have been reported to be calculated unfairly.
- In some cases, impact fees have caused double taxation when new development pays fully for new infrastructure and pays property taxes that may be applied towards new infrastructure.

- Sometimes impact fees shift development to localities without fees; they require a strong market demand for development to be successful.
- Impact fees are often being viewed as "anti-growth" and are therefore sometimes not politically feasible.

While discussing impact fee, it should be noted that there are mainly two ways in which the developer contributes financially to the improvement of the impacted facility; one is through proportion share and secondly is through impact fee payment. The difference between the proportion share and the impact fee is based on the magnitude of the development. While the proportion share is used in major developments like development of regional impacts (DRI), the impact fee is used mainly for small developments which affect small radii. Kristine et al (7) differentiated the impact fee from proportional share, highlighting that, impact fee is based upon a development's impact to a specific facility, and are applied within a designated zone, rather than to an impacted facility. Local governments have their established unit cost for different impact fees. The type of land use for the development is the determinant of the magnitude of the impact fee based on trips rates. The land use with a higher trip generation rate is expected to generate more traffic, hence has higher unit impact fee rate. Some Cities and Counties have developed their own trip rates for individual land uses but most of them still apply the rates listed in ITE Trip Generation Manual 7th Edition.

In the city of Destin, Florida, the impact fee is required to be proportional to the need of the new facilities affected by the new development. The city has a fundamental principle,

that impact fees should not charge new development for a higher level of service than is provided to existing development. While impact fees can be based on a higher level of service than the one existing at the time of the adoption of the fees, two things are required if this is done. First, another source of funding other than impact fees must be identified and committed to fund the capacity deficiency created by the higher level of service. Secondly, the impact fees must generally be reduced to ensure that new development does not pay twice for the same level of service, once through impact fees and again through general taxes that are used to remedy the capacity deficiency for existing development. In order to avoid these complications, the general practice is to base the impact fees on the existing level of service (9). The city of Destin proposed their transportation impact fee using an improved-driven model which divides the cost of growth related improvements for a fixed planning horizon by average daily trips to be generated at the same horizon to determine cost per development unit. The proposed impact fee is calculated under equivalent dwelling unit which is a single family detached dwelling unit. The proposed equation for impact fee calculations by the Destin city is shown below:

Impact Fee = (TRIPRATE*%NEW*LENGTH/SINGLE_FAMLY VMT)*((COST–DEFICIENCY)/NEWEDU))-CREDIT/EDU

Where:

SINGLE-FAMILY VMT = Relative vehicle-miles of travel generated by a single-family detached dwelling unit

TRIPRATE = Average daily trip ends on a weekday (ADT) per unit of development

%NEW = % of ADT that are primary as opposed to passby or diverted-linked trips

LENGTH = Ratio of average trip length for the proposed use to average single family trip length

COST = Total net cost of planned capacity-expanding improvements for roads

DEFICIENCY = The cost of remedying existing deficiencies, if applicable

NEWEDUs = Projected increase in single-family equivalent dwelling units over the planning horizon

CREDIT/EDU = Revenue credit per EDU, if appropriate

City of Destin, FL, Proposed Impact Fee Estimate Model; **Source:** Duncan Associates (Reference # 9)

In another study conducted by Duncan Associates for the town of Farragut, Tennessee (10), total vehicle mile of capacity (VMC) and vehicle mile of travel (VMT) were used as the main determinants of the proposed impact fee. Vehicle miles of capacity refereed to the system wide available capacity within the impacted roadway network. The ratio of the VMC to VMT explain the capacity demand ratio, that is, for every vehicle-mile of travel (VMT) generated by the development, the transportation development fee charges the net cost to construct an additional vehicle-mile of capacity (VMC).

Impact Fee = (TRIPS*%NEW*LENGTH/2)*(COST/VMC)*VMC/VMT–CREDIT/VMT

Where:

TRIPS	*= Trip ends during an average weekday*
% NEW	*= Percent of trips that are primary trips, as opposed to passby or diverted-link trips*
LENGTH	*= Average length of a trip on the major roadway system*
÷ 2	*= Avoids double-counting trips for origin and destination*
COST/VMC	*= Average cost to add a new daily vehicle-mile of capacity*
VMC/VMT	*= System-wide ratio of VMC to VMT on the major roadway system*
CREDIT/VMT	*= Revenue credit per VMT*

Town of Farragut, TN, Proposed Impact Fee Estimate Model; **Source:** Duncan Associates (Reference # 10)

In Lee County, the impact fee equation, VMT determine the expected trip lengths to be generated by the new project trips. Availability of accurate VMT sometimes is not readily available at local level causing some local governments to use national data then apply adjustment factors to reflect local conditions. For instance, the proposed impact fees in Lee County (11) use national travel demand data to calculate VMT then multiply with the adjustment factors to reflect local conditions. The use of adjustment factors correct hidden variables not used in the impact fee equation but which are considered to have effect to the impact fee.

The following are some of the impact fee equations from different Counties in Florida:

Lee County

Impact Fee = (ADT*%NEW*LENGTH*ADJUST/2)*((COST/LANEMILE/AVG LANE CAPACITY–CREDIT)/VMT

Where:

VMT = ADT x % NEW x LENGTH x ADJUST ÷ 2
ADT = Trip ends during average weekday
% NEW = Percent of trips that are primary trips, as opposed to pass-by or diverted-link trips
LENGTH = Average length of a trip on the major roadway system
ADJUST = Adjustment factor to calibrate national travel demand factors to local conditions
COST/LANE-MILE = Average cost to add a new lane to the major roadway system
AVG LANE CAPACITY = Average daily capacity of a lane at desired LOS
CREDIT/VMT = $/GAL ÷ MPG x 365 x NPV
$/GAL = Capacity-expanding funding for roads per gallon of gasoline consumed
MPG = Miles per gallon, average for U.S. motor vehicle fleet
365 = Days per year (used to convert daily VMT to annual VMT)
NPV = Net present value factor (i.e., 12.95 for 20 years at 4.55% discount)

Lee County, FL, Proposed Impact Fee Estimate Model; **Source:** Duncan Associates (Reference # 11)

Alachua County

ATTRIBUTABLE TRAVEL = [(TRIP RATE x TRIP LENGTH)/2] * %NEW TRIPS
NEW LANE MILES = ATTRIBUTABLE TRAVEL / LANE CAPACITY
CONSTRUCTION COST = NEW LANE MILES x CONSTRUCTION COST PER LANE MILE
RIGHT OF WAY COST = NEW LANE MILES x RIGHT OF WAY COST PER LANE MILE
ENGINEERING COST = NEW LANE MILES x ENGINEERING COST PER LANE MILE
TOTAL COST = CONSTRUCTION COST + RIGHT OF WAY COST + ENGINEERING COST
MOTOR FUEL CREDIT = {[(ATTRIBUTABLE TRAVEL * 365) / MPG] * TAX} * PV
NET COST = TOTAL COST - MOTOR FUEL CREDIT
PV = Present Value Factor, Capital Tax Rate = ¢18.5 per Gallon

Alachua County, FL, Impact Fee Estimate Model; **Source:** Alachua Board of County Commissioners

Hillsborough County

{[(# x TGR x TL x (1-%IT)/CL/ 2 x CC x (1-%ILR) minus {[# x TGR x TL x (1-%IT)/2/ 17.16 x$0.089 x 365 x 13.8]} x PC

Description of Elements:

= a. number of dwelling units for residential uses
b. For all land uses, the appropriate measure of size expressed in the Trip Ends Generation Report shall be determined by the County and used in the impact fee formula.
GR = trip generation rate, TL = trip length
%IT = percentage of trip length on the interstate system in Hillsborough County, 22.9%
CL = capacity per lane mile (LOS D = 7,500)
CC = cost to construct one lane mile (% urban + % rural)
%ILR = interstate and local roads (15%) (This term represents the percentage of total travel which is on local roads plus the percentage of interstate travel which represents "thru" trips not attributable to any development in Hillsborough County.)
PC = percentage of impact fee charged (84.3061%)
17.16 = Average number of miles per gallon of fuel consumed per day per vehicle in fleet in Hillsborough County (From the City of Tampa Technical Consideration for a Transportation Impact Fee – February 1987)
$0.089 = paid per gallon of gasoline for which new growth receives credit towards construction of new capacity due to growth.
365 = average number of days in a year
13.8 = the net present value factor at 8% interest over 50 years

Hillsborough County FL, Impact Fee Estimate Model; Source: Consolidated Impact Fee Program

Orange County

NET COST = COST - CREDIT
COST = NETVMT x COST/VMT
CREDIT = VMT x CREDIT/VMT
Where:
VMT = ADT x % NEW x ATL ÷ 2
NETVMT = ADT x % NEW x NETATL ÷ 2
ADT = Trip ends during a weekday
% NEW = Percent of trips that are primary trips, as opposed to pass-by or diverted-link trips
ATL = Average trip length
NETATL = Average trip length on the non-freeway system
÷ 2 = Avoids double-counting trips for origin and destination
COST/VMT = COST/LANE-MILE ÷ CAPACITY
COST/LANE-MILE = Average cost to add a new lane to the major roadway system
CAPACITY = Average daily capacity of a lane at desired LOS
CREDIT/VMT = $/GAL ÷ MPG x 365 x NPV
$/GAL = Capacity-expanding funding for roads per gallon of gasoline consumed
MPG = Miles per gallon, average for U.S. motor vehicle fleet
365 = Days per year (used to convert daily VMT to annual VMT)
NPV = Net present value factor

Orange County FL, Impact Fee Estimate Model; Source: Orange County Road Impact Fee Update

Collier County

Net Impact Fee = Total Impact Cost – Gas Tax Credit
Where:
Total Impact Cost *= ((Trip Rate × Recommended Trip Length × % Non-Passerby) / 2) × (1 - Toll Facility Adj. Factor) × (Cost per Lane Mile / Avg. Capacity Added per Lane Mile)*
Gas Tax Credit *= Present Value (Annual Gas Tax), given 5% interest rate & 25-year facility life*
Annual Gas Tax *= (((Trip Rate × Assessable Trip Length × % Non-Passerby) / 2) × Effective Days per Year × $/Gallon to Capital) / Fuel Efficiency*
Trip Rate = the average daily trip generation rate, in vehicle-trips/day (7.41)
Recommended Trip Length = the actual average trip length for the category, in vehicle-miles (5.88)
Assessable Trip Length = average trip lengths represent travel on the functionally-classified road system, but gas taxes are collected for travel on all roads including local roads; therefore, an adjustment factor of 0.5 miles was added to the recommended trip length of each land use category to account for this (5.88 + 0.50 = 6.38)
% Non-Passerby = adjustment factor to account for trips that are already on the roadway (100%)
The total daily miles of travel generated by a particular category (i.e., rate*length*% non-capture) is divided by two to prevent the double-counting of travel generated among land use codes since every trip has an origin and a destination.
Toll Facility Adjustment Factor = adjustment factor to account for the travel demand occurring on interstate highways and/or toll facilities (12.0%)
Cost per Lane Mile = unit cost to construct one lane mile of roadway, in $/lane-mile ($6,300,248)
Average Capacity Added per Lane Mile = represents the average daily traffic on one travel lane at capacity for one lane mile of roadway, in vehicles/lane-mile/day (10,901)
Present Value = calculation of the present value of a uniform series of cash flows, gas tax payments in this case, given an interest rate, "i," and a number of periods, "n;" for 5% interest and a 25-year facility life, the uniform series present worth factor is 14.0939
Effective Days per Year = 365 days
$/Gallon to Capital = the amount of gas tax revenue per gallon of fuel that is used for capital improvements, in $/gallon ($0.256)
Fuel Efficiency = average fuel efficiency of vehicles, in vehicle-miles/gallon (17.55)

Collier County FL, Impact Fee Estimate Model; Source: Collier County Transportation Impact Fee Study

2.5 Factors Affecting Highway Safety

Many researches in highway safety have focused on different factors which affect roadway safety. The factors are categorized as traffic characteristics, road geometrics, roadway surface condition, weather and human factors. Previous research has shown that geometric design inconsistencies, operations (traffic mix, volume, and speed), environment, and driver behavior are the common causes of accidents. Environmental conditions and driver behavior can seldom be foreseen. They are specific to case, time, and driver; they are also influenced by geometric inconsistencies. Most of the studies have shown the influence of various geometric design variables on the occurrence of accidents and have concluded that not all variables have the same level of influence in all places. This uncertainty in the influence of geometric variables on accidents has prompted researchers to develop mathematical models to better understand the relationship. Mathematical models enable highway agencies to select design standards that are essential to highway safety and to allow comparisons among alternative designs that can optimize the overall safety of the highway system under limited resources and other constraints. These models can also be used to test the sensitivity of accident rates to changes in specific geometric variables. From the relation of factors mentioned above, different researchers have developed the relationship of roadway safety in terms of crash frequency and crash rates, fatality and injury rates and the roadway elements, traffic characteristics, and pavement conditions. Many of these previous studies investigated the relationship of crash rates, or frequency in terms of number of lanes, lane width, presence of median, median width, type of median, shoulder width, AADT, access density, number

of signalized intersections per road segment, speed limit, vertical grade, horizontal curvature, length of roadway segment, weather condition, time of the day and day of the week. The relationship between safety on the highway and factors mentioned above can be the primary focus to be included in transportation planning.

Though there have been a lot of studies related to how different factors affect crash occurrence, but very few studies has generated the common model integrating safety and planning. The effect of roadway geometrics to the crash occurrence has been discussed solely on the safety point of view by many researchers. For instance, the effect of lane width has been discussed under two scenarios, effect of wider and narrow lanes to the crash occurrences. The theory is based on the assumption that the wider lanes have large separation between vehicles moving in adjacent lanes which may provide more room for correction in near-accident circumstances. However some studies have suggested that the narrow lanes make the drivers more attentive on the road, hence can lead to crash avoidance. While these arguments are based on the safety point of view, the planning which forecast the traffic operations, can have different views of the effect of lane width to capacity improvement. By combining the safety and planning considerations on the effect of lane width, the optimized design can be reached which consider both effects.

Number of lanes also has been discussed with respect to crash occurrences. Safety studies do relate presence of higher number of lanes with increase in crash frequency or rates. In their research, Noland and Oh (12) found that increasing the number of lanes was

associated with increased traffic crashes. In another study, Abdel-Aty and Radwan (13) found that more lanes in urban roadway sections are associated with higher crash rates. Garber (14), considered flow per lane and found that there was an increase in the crash rate as the flow per lane increased. These findings contradict the traffic operations which needs more number of lanes as much as possible to increase the highway capacity. The joint study of safety and planning can be appropriate solution on effect of number on safety and operations.

Previous researches also studied effects of the speed to crash frequency. In analyzing crashes in Virginia, Garber and Gadiraju (15) reported that crash rates increased with increasing speed variance on all types of roadways. The crash rates were higher when the mean speed was less than the posted speed. The crash rates decreased to a minimum when the means were approximately equal to the posted speed limit, then continued to increase significantly as the speed increases above the posted speed limit.

The effect of land use and location of the roadway to the highway safety has been considered separately in different studies. Various studies considered suburban, urban or rural areas separately and few of them investigated the three situations in the same crash models. Retting *et al.* (16) studied a simple method for identifying and correcting crash problems on urban arterial streets in Washington DC. They found that urban crashes are often concentrated at specific locations and occur in patterns that can be mitigated through appropriate engineering countermeasures. In another study (17), they considered

safety in rural and small urbanized areas. Comparative risk assessment showed village sites to be less hazardous than residential and shopping sites. Karlaftis and Golias (18) investigated effects of road geometry and traffic volumes on rural roadway accident rates. They developed a methodology which allows for the explicit prediction of accident rates for given highway sections, as soon as a profile of a road is given. Greibe (19) created accident prediction models for urban roads in which he found shopping streets and city center roads having significantly higher accident risk than, for example, residential roads in less densely built-up areas. He concluded, the lower the building density, the lower the accident risk.

2.6 Safety Parameters not included in the Current Impact Fee Equations

From the above review about the current impact fee calculation methodologies, it shows the equation or approach have concentrated on how the new trips will affect the existing capacity but not safety. As discussed in the introduction section, there is a need to incorporate safety in the impact fee equation. Discussed below are some of the variables generated by the new developments and which the literature has shown to be influencing crash occurrence.

2.6.1 Generated Traffic Volumes

The impact of higher volumes to the safety is mainly at congestion point when the number of vehicles exceeds the capacity of that particular road. Qin *et al.* (20) studied effects of higher traffic volumes to crash rates, they found less severe injury but high

frequency of crashes are associated with high traffic volumes. More severe injury but low frequencies of crashes are associated with low traffic volumes locations. In another study, Zeeger *et al*. (21) found that crashes are most likely to occur at high traffic volume roadways since more conflict are created when number of vehicles increase. For low-volume road accidents, the primary causes of crashes were related to geometrics, roadside hazard, terrain, and driveways. Mouskos *et al.* (22) and Hadi *et al.* (23) found that sections with higher AADT levels are associated with higher crash frequencies for all highway types and classes. Garber (14) found an increase in the crash rate as the flow per lane increased. Milton and Mannering (24) found the positive coefficients of AADT in the crash prediction model indicating the increase in number of vehicle at particular section of the road is likely to increase the probability of accident. The positive effect of traffic volumes to crash frequency was also concluded by other researchers like Aruldhas (25), Sawalha (26) and Poch and Mannering (27).

2.6.2 Impacted Access Density

Accessing the proposed development site is an important part of the planning and management of traffic to and from the site. The access points carry all traffic generated by the development, so their locations and spacing determine operations and safety of the corridor. Access management is a comprehensive approach to the control and regulation of all aspects of highway access. The developer is required to examine driveways, median openings, turn-lanes, traffic signals and their relationship to access points to the development. Not well managed access points result into safety concerns. Though well

planned access management reduces conflicts, access points can increase the likelihood of crash. The safety impact due to increased access points is obscured by the traffic volume on intersecting roadways and by vehicle miles of travel. Different studies have related access density with crash frequencies on the highways.

The study done in New Jersey (22) on the impact of access driveways to crash rates found approximately 30% of crashes at the study segment were caused by presence of access points in which 25% where vehicles entering/exiting through access points have impact to/from the mainline. Karlaftis *et al* (18) found access control were one of the most important factors in crash occurrence. Mouskos *et al*. (22) found access density and intersection spacing having positive coefficients in crash model. Gluck *et al.* (28) found increasing access points by 50% per mile would increase crash probability 30%. Papayannoulis *et al.* (29) found a road with 60 access points per mile would have triple the accident rate compared to 10 access points per mile. From literature, it is obvious that any development resulting in creating new access points (driveway), also creates certain kind of safety problems. The impact of these access points to the safety will depend on the existing access density. The new access created at segment already saturated will have more safety impact compared to one created at less density area. The developers being the source of these new access points, then impact fee should have parameter which takes into account safety component of the development in consideration to access density.

2.7 Integrating Safety and Planning

Highway safety has been the first priority for many transportation agencies. In Florida, for example, from 1997-2001 there were an annual average of 246,440 reported crashes with about 236,055 injuries and 2926 fatalities (31). The crash occurrences are the result of combination of different factors ranging from human, roadway, environmental and traffic characteristics. Human causes refer to those crashes which results as errors made by the drivers or pedestrians. The human factor is documented to be the more than 60% of all crashes occurring on the highway. Roadway causes are the crashes which are due to design deficiencies. These are not many and are easy to trace and fix. Environmental crashes are those related to weather conditions, pavement conditions which occur occasionally, like heavy rainfall or natural disasters like hurricanes. Traffic characteristics, though not solely the cause of the crash, sometimes accelerate the cause. Congestion is one of the situations where traffic can be termed as the contributing cause of the crash. These causes of crashes can be evaluated and included in safety and planning programming. The safety evaluation is the procedure of identifying locations having significant crash trends then using the trend to identify possible causes.

Identification of crash trends is very essential in developing and implementing comprehensive safety countermeasures using engineering, enforcement and education. The evaluation always includes the review of the entire roadway network, or portion of network, and to identify sites with potential for safety improvement. The segments within the networks in which evaluation reviews are always targeted include:

- Segments of intersections with high accident frequencies
- Locations with high proportion of specific accident type,
- Locations with sudden increase in mean accident frequency
- Locations with steady increase in mean accident frequency
- The programmed corridors (including those affected by new developments)
- Segments ranked with potential for safety improvement

Before one can identify the types of strategies or investments that can improve safety, the safety challenge must first be understood. This means not only understanding the "big picture" from the perspective of numbers and incidence of road-related fatalities and major injuries, but also becoming knowledgeable about some of the leading contributing factors. The best examples of safety conscious planning began with a comprehensive collection and analysis of data, which often includes conducting research on what factors are most important with respect to fatalities or personal injuries (32).

2.7.1 Safety Evaluation Procedures

The process of safety evaluation starts by crash analysis. The crash used in safety evaluation always cover several years of historic trend, usually 5-years though the range will depend on evaluation objectives. The crash data are obtained from the databases or from the crash forms maintained by the Counties or States. From the crash data, the clues are documented e.g. the frequent contributing causes, age of the drivers and weather conditions. While documenting the clues, the evaluator can ask questions, why did the driver decided to do and that which ended to the crash occurrence. In case of the presence

of the curve, the review has to evaluate if the excessive speed was the cause of the crash and the frequency of excessive crashes. The evaluator also needs to document any crash related to pass-by for two-lane highways, and crashes related to failing to yield right of way.

The statistical analysis is very important part of safety evaluation. Descriptive statistics about the mean, median, significance of certain kind of crash will lead to logical conclusion about the crash pattern. The frequency of crash types like rear end, sideswipe, angle, left turn and the like can be sketched into histograms or any other kind of graphs for comparison. Apart of statistical analysis of the crashes, collision diagrams are used to visualize the crashes at the locations where they occurred on the highway. The safety reviews use the collision diagrams to trace the trend of crashes and relate them with certain roadway features. From the statistical analysis and collision diagrams, the safety evaluator can relate the occurrence of certain type of crashes with certain kind of road alignment, geometry, land use, neighborhood or environmental condition. Also certain kind of signing and markings can be associated with certain kinds of crashes from the collision diagrams. The found suspicious clues like roadway features, certain kinds of designs, traffic characteristics, land use and environmental conditions associated with certain types of crashes are entered into safety programs.

Transportation planners need to know in advance areas of concerns in order to be included in the planning process. Safety evaluations highlight areas of concern with

respect to highway safety, hence these evaluations can be incorporated into short or long term transportation plans. Considering the new developments with known type of land use, the prior safety evaluation will have type of crashes associated with such kind of developments whereby countermeasures can be taken during improvement or operation stages. For example, if prior safety evaluations found more driving under influence crashes associated with neighborhoods with many night clubs, any new night club proposed must be forecasted to generate these kinds of crashes and prior prevention be taken in the planning stage.

Although the public demands a safe transportation system, safety historically has not been an explicit part of transportation planning. A clear need has developed for safety to be considered as part of the planning process instead of as a reactionary consideration as it has been (33). The question which arises is how safety can be incorporated in transportation planning. Another question is what comes first between safety program and transportation planning. Kelvin (34) gave opinion of how to integrate safety and planning initiatives. He highlighted suggested a programming safety improvements to address roadway "hotspots" or collision-prone locations, reflecting road safety considerations as a key decision-making parameter in evaluating projects and programming expenditures and establishing inherently safe transportation networks as one some of the ways to connect safety and planning together.

Some of the safety hotspots which can be the key elements in planning stage include historical crash history and pattern. There are regular or common crash patterns related to certain kinds of community, land use, and design parameters which can be foreseen in planning stage. Safety engineers use crash summary and pattern to identify possible cause of crash at particular road location. The first step is to summarize the crashes by severity level, in this stage, the severity of crashes are tallied and descriptive statistics documented. Higher frequency of fatalities and severe injury will give clue of something not well either on the geometry of the road or pavement. The next stage is to classify the crashes by type, in this case, the crashes are tallied by collision type e.g. rear end, head on, sideswipe, angle, left turn, right turn, collision with pedestrians, collision with roadside objects an so on. These collision types help to identify the weakest spot if it is driver related, geometric, signal pavement, lighting or weather. The third step is to draw or sketch the crashes on the collision diagram. The collision diagram will show the crashes by location on the road, this assist in identifying the dangerous locations. Furthermore, significance level analysis can perform to determine if the spot is a high crash location or not.

The result from crash analysis will help transportation planning by knowing roadway features or environmental conditions are associated occurrence with certain type of crashes. For instance, if the presence of many access points are associated with many angle and left turn crashes, the planning must make sure the number of access points are

limited as much as possible. If congested condition is associated with more number of rear end crashes, then the planning must ensure the congestion is controlled.

To improve highway safety, four E's are is usually addressed, which are engineering, education, emergency, and enforcement. Engineering is the area where both safety and planning connect together to develop physical improvements to the transportation system. Since physical improvements to the transportation system are a shared responsibility of engineering and planning staff, the planner's role will be to inform the transportation infrastructure improvement process with safety principles and data and facilitate development of engineering safety strategies within the overall process.

2.7.2 The use of Geographic Information System

Geographic Information Systems (GIS) can be utilized by both safety engineers and plannersto share planning and safety information. Safety engineers and planners can have the ability to analyze crash data and use GIS to map high-crash areas, and define safety problems. While the distribution of crash data can be presented specially using GIS, planning programs can also be presented in the same way. Using GIS layer system and attributes, safety and planning programs can be integrated. Safety and planning engineers understanding of crash and planning data and performance measures is a key for developing comprehensive approaches to safety. Planners are accustomed to managing diverse groups to help them understand an issue and develop solutions.

2.7.3 The role of legislation

The jurisdiction's legislation on highway safety and planning is another important factor in integrating the two. Well improvement transportation safety must involve legislative governing transportation policy. The legislation always depends on the data provided by transportation engineers to lay out general policy. Legislation also control crash related behaviors like engagement in dangerous behaviors like driving under influence and other by-laws like the use of seat belt and cell phone talking while driving.

2.7.4 The role of planners

Table 2.1 summarizes some of the regular crash types, their possible causes and possible counter measurer. All of the listed countermeasures can be achieved by incorporating the transportation planners. For example, some of the possible causes of rear end crashes at the signalized intersections include inadequate signal timing, poor visibility, large volume of pedestrian crossing and slippery surfaces. Using safety evaluation procedures and past crash history at signalized intersections, the countermeasure can be improved through planning. In any short and long range transportation planning, the planners need to foresee the need of each intersection and install or improve warning devices, signals, adjust signal intervals and speed limits. Apart from intersection crashes, in general transportation planners are trained to analyze operations at the corridor level. Many aspects of corridor management provide opportunities for safety improvements. The planners can improve safety by including good pedestrian and bicycle facilities in their programs not only helps to reduce congestion, but can reduce the number of vehicle trips

and lower roadway exposure. Corridor intersection treatments such as signal optimization can significantly improve travel times and reduce levels of frustration and aggressive driving. Access management policies can have a significant impact on both the capacity and safety of roadways. Individual intersection improvements can make turning movements safer for both drivers and pedestrians. Transportation planners can work with safety engineers to identify operations and infrastructure problems and help program improvements. Planners also can work with enforcement on corridor-based efforts at enforcing traffic laws, reducing impaired driving, analyzing speeds, and increasing safety belt use.

Transportation planners need to be familiar with the crash system in their States or Counties. As pointed out earlier, crash data will help planners in identifying high-crash corridors and intersections, determining the types of crashes, identifying contributing factors and determining key human factors or behaviors that are associated with number and severity of crashes. Once the planners understanding the major transportation safety issues, countermeasures can be developed, starting with the areas with the highest number of and most severe crashes.

2.7.5 Other safety-planning integration approaches

According to National Cooperative Highway Research Program (NCHRP, 32), incorporating safety into transportation-planning often means integrating safety into all

aspects of an agency's operations. The report underlines the following requirements as the way to integrate safety and planning together:

- Planning: Collecting and maintaining a record of crash, traffic and highway data; analyzing available data to identify hazardous highway locations; conducting engineering study of those locations; prioritizing implementation;

  conducting benefit-cost analysis; and paying special attention to railway/highway grade crossings.

- Implementation: Scheduling and implementing safety improvement projects and allocating funds according to the priorities developed in the planning phase.
- Evaluation: Evaluating the effects of transportation improvements on safety including the cost of the safety benefits derived from the improvements, the crash experience before and after implementation, and a comparison of the pre- and post-project crash numbers, rates, and severity.

The effective integration of safety considerations into transportation-planning requires the collaborative interaction of numerous groups (32). In most cases, who is involved will depend on what issue is being addressed. For example, a bicycle safety program focused on child safety might involve enforcement agencies, governor highway safety representatives, local public works agencies, school administrators, parent organizations, churches, local store owners and business associations, emergency response providers, and civic associations. It is therefore difficult to identify in a generic sense who should be involved in safety conscious planning. The key, however, is collaboration, and the key to successful collaboration is identifying for each participant what benefit each receives

through participation. The metropolitan planning organization (MPO) should be responsible for developing a regional transportation plan and a transportation improvement program. The MPO engages in planning studies, program development, and policy formulation leading to improved transportation system performance. The MPO also collects data for operational performance of the transportation system. They are also most often the developers and users of regional models that are used to analyze transportation system performance. For both activities, that is, data collection and analysis, the MPO will have an important role in efforts to consider safety more comprehensively in the transportation-planning process.

2.7.6 Assessing Safety-Planning Integration

The above sections discussed different approaches as the approach to integrate safety and planning together. In summarizing the above discussion, NCHRP (32) has highlighted questions and checklists to be reviewed before and after the program intended to integrate safety and planning. These checklists include the following questions:

- In any planning vision, safety program must be included
- In at least one planning goal, at least two goals related to safety must be included
- Safety-related performance measures must be part of the set programs
- Safety-related data must be used in problem identification and solutions seeking
- Evaluation criteria used for assessing the relative merits of different strategies and projects must include safety-related issues

- The products of the planning process must include at least some actions that focus on transportation safety
- To the extent that a prioritization scheme is used to develop a program of action for an agency, safety must be one of the priority factors
- All of the key safety stakeholders must be involved in the planning process

2.8 Chapter Summary

Literature review has shown cities and counties use different equations though same with almost same parameters to calculate road impact fee. The most common variable used in these equations is the trip rate for the type of the land use to be constructed. In relation to the trip rate is the vehicle mile of travel which multiply the number of trips generated with the expected length in miles to be used by these vehicles. The credits in terms of gas tax are deducted from the impact fee, since they are taken as benefit to be resulting from the development. Literature has shown current impact fee equations contain any safety related variables. Some roadway variable related to new developments have been found in the literature to be the source of certain kind of crashes, which can be incorporated in planning or in impact fee calculation. The variables like driveway density, number of trips (traffic) and vehicle mix have positive impact to crash occurrence, any development impacting these parameters increase probability of the crashes, hence need to be considered.

The chapter also discussed how safety and planning can be integrated together. The study has found that safety issues can be included as a factor in planning potential future projects and in establishing transportation priority programs. Inclusion of the safety in planning programs will lead to effective performance measures to reduce crashes and improve operations. Safety must be evaluated and analyzed in which the findings incorporated in the short or long term transportation plans. Through the evaluation and analysis process, planners determine how the system is performing and what changes in the transportation system will be needed to improve safety. In order to ensure that safety becomes an integrated part of the plan, incorporating safety into the transportation planning goals and objectives is important. Integration of these two important community issues should start by safety evaluation. Through safety evaluation, crash patterns, frequency and high location areas are identified. The statistic analysis is also used to support the observed crash data through descriptive statistics, significance testing and charts and graphs for presentation. Collision diagrams should be used to trace the crashes with respect to highway locations where there occurs. The trend from the collision diagram will lead to identification of safety vulnerable points on the highway. Possible crash contributing causes and counter measures should be used as a starting point for crash reduction in the planning process. Both safety and planning engineers should use the result of safety evaluation into their short and long range planning.

Table 2.1: Common crashes at the intersections, possible causes and measures

Crash Type	Possible Causes	Possible Countermeasure
Right-angle collisions at unsignalized intersections	Restricted sight distance	Removing sight obstructions, restricting parking near corners, installing stop signs and warning signs, improving street lighting, reducing speed limit on approaches
	Large total intersection volume	Installing signals
Right-angle collisions at signalized intersections	Poor visibility of signals	Installation of advanced warning devices
	Inadequate signal timing	Adjusting change interval, providing all-red clearance interval, installing signal actuation, retiming the signals and providing progression
Rear-end collisions at unsignalized intersections	Pedestrian crossing	Improving signing or marking of pedestrian crosswalks or relocation of the crosswalks
	Not aware of intersection	Improving warning signs
	Slippery surface	Overlaying the pavement, providing adequate drainage, grooving the pavement and reducing speed
	Large turning vehicles	Creat exclusive turn lanes, prohibit turns and increase curb radii
Rear-end collisions at signalized intersections	Poor visibility of signals	Installing advance warning devices, relocating signals and removing obstacles
	Inadequate signal timing	Adjusting change interval and providing progression through a set of signalized intersections
	Pedestrian crossings	Improving signing or marking of pedestrian crosswalks
	Slippery surface	Overlaying the pavement, adequate drainage, grooving the pavement and reducing speed limits
Pedestrian crashes at intersections	Restricted sight distance	Removing sight obstructions, installing pedestrian crossings, improving pedestrian crossing signs and rerouting pedestrian
	Inadequate protection for pedestrians	Adding pedestrian refuge islands
	Inadequate signals	Installing pedestrian signals
Fixed-object collisions	Objects near traveled way	Removing obstacles near roadway, installing barrier curbing, installing breakaway feature to light poles, signposts and protecting objects with guardrail
Sideswipe collisions	Roadway design inadequate for traffic conditions	Improving pavement markings, channelizing intersections, creating one-way streets, median divider and widening lanes
Night crashes	Poor visibility	Improving street lighting, improving delineation markings and warning signs
Collisions at driveways	Left-turning vehicles	Installing median divider and two-way left-turn lanes
	Improperly located driveway	Regulating spacing of driveways and minimum corner clearance and consolidating adjacent driveways

CHAPTER 3

TRAFFIC PROJECTION: SIMPLIFIED APPROACHES

3.1 Overview

The magnitude of mitigation which the developer is supposed to pay depends on the proposed improvement which also depends on the amount of trips generated from this new development. The project trips are added to the projected existing traffic (background) to the buildout year. Failure to project accurate traffic will lead to erroneous recommended proposed improvement. Not only for new developments, but also and future traffic condition study will need accurate and reliable projection. This makes traffic projection one of the crucial stages in any traffic related study. Accurate methodology should be used to project traffic by considering the scale of the study, data availability and geographical location, among other factors. Apart from leading to make accurate proposed improvements, future highway safety plans will rely mostly on the projected traffic. Accurately projected traffic provides the basis to optimize the functional design of transportation facility.

Different methodologies are currently used for traffic projection depending on the geographical location, size of the study, analysis tool and many other factors. Where built models are not available, the growth rates if available are always used to project traffic to the design year. Growth rates are used as compounded growth e.g. $(1+r)^n$ where 'r' is a growth rate and 'n' the number of years. Under this approach, the compounded or linear

growth rates are calculated from the trend of traffic from different years. The difficulty of this approach arises if no sufficient historical traffic data is available, the likely case for many single intersection and corridors and mainly for the new developments. While not all corridors or intersections will have enough historical traffic data, but also some of them may result with inconsistent traffic trend, meaning no defined trend to enable accurate growth rate calculation. Inconsistency (rise and fall) or inadequacy of historical traffic data can lead to over or under estimation of growth rates. Some of highway's economic analysis models assume traffic growth-rate pattern, which is based on one or more projected traffic volumes *(49)*.

Apart from the use of growth rates, various modeling tools and software have been created to assist in traffic projection. These models utilize mainly socio economic factors to forecast traffic to the desired benchmark year. Though they are accurate and highly technically rated, some of these models are sophisticated, and involve massive arithmetic in some stages and sometimes not easily interpretable. States, Counties and Cities have developed large scale models at defined geographical location for future traffic forecast. For instance, south Florida has different urban area transportation study (UATS) models like TCRPM, SERPM4, Palm Beach County UATS Model, Broward County UATS Model and others. Though these UATS models always yield approximate projection, they are data intensive and calibration costly. Furthermore, UATS models need highly technical personnel to run and interpret the outputs. In many cases, the projections from UATS models are used to interpret local conditions. No study has been done to determine

the effect of using the projection from these large scale models to local conditions like single isolated intersection or along short corridor. According to the study done by South Dakota Department of Transportation Office of Research *(50)*, in most cases, the primary traffic forecasting developments are taking place at states with larger and denser populations where sophisticated automated processes, more abundant data, and traffic modeling software are being employed. Less populated rural states are having apparent difficulties implementing advanced traffic forecasting procedures and traffic models that are normally structured with an emphasis on urban perspectives. Conditional differences at rural states and less available resources at rural states were suggested as the primary factors contributing to this traffic forecasting developmental lag. While traffic forecasting models are well utilized in different locations, it becomes difficult to apply them when projecting traffic for the new developments. Based on these facts, application of these models in isolated locations can lead to inaccurate traffic projection. For these models to be applicable in use for small scale projection, then a combination of various factors is should be involved to transfer large scale to small scale use. For instance, social economic variables related to the radius of the study should be reviewed and updated if necessary to reflect local condition. Furthermore, the use of large scale models requires high capital investment and initialization, maintenance, and calibration.

Florida Department of Transportation (FDOT) has highlighted the procedures for traffic forecasting at locations without developed traffic model (59). The department allows utilization of gasoline consumption, population data, vehicle registration, census and

comprehensive plans to develop growth rates at the locations where traffic forecasting models are not available. The department recommends the use of linear regression is as one of the immediate approach for projecting traffic using any of the historical trend data related to traffic growth listed above. Furthermore, the department allows at least recent ten years of data if available is recommended for developing linear regression (sample size). The projection through linear regression is then required to be checked for reasonableness and consistency. The weakness of the use of linear regression is inability to reflect real world traffic growth. Linear regression assumes continuous traffic growth at constant rate and even beyond the limit capacity depending on the number of lanes. In normal traffic growth, when the road is opened for traffic operations, there is slight increase in traffic, and then the growth will rise at high rate, but flatten to horizontal (gradual growth) at the end when the capacity or equilibrium is reached. In this case, the use of linear seems not to be the most accurate compared to available methodologies which can project traffic in absence of forecast models.

Based on the importance of accurate traffic projection and considering the problems facing small scale projection for new developments and areas without enough historical or modeling data, this study introduces a simple approach for small scale traffic projections beyond linear regression. Small scale is referred as projection involving small corridor or at isolated intersection. This simple approach utilizes the use of five different functions as a simplified approach to traffic forecasting in which the best one is selected for traffic projections. These five simplified functions include logistic function, power

function, log or transformed power function, combined power-exponential function, and log or transformed power-exponential function. These functions are in the following forms;

$$\text{Power Function}: V = AX^b \tag{3.1}$$

$$\text{Log Power}: V = \ln(AX^b) \tag{3.2}$$

$$\text{Power - Exponetial}: V = AX^b * \exp(CX) \tag{3.3}$$

$$\text{Log Power - Expo}: V = \ln(AX^b * \exp(CX)) \tag{3.4}$$

$$\text{Logistic}: V = \frac{C}{(1 + A * \exp(-bX))} \tag{3.5}$$

The above functions were selected for traffic forecasting based on their distribution nature which convey or resemble traffic growth trends. The use and handling of these functions is also simple, which can be utilized and interpreted easily. The functions need only one accurate and reliable source of historical data like traffic counts, employment data, economic data, population data, number of registered vehicles or any kind of historical traffic related data which have direct relation and influence to traffic growth. The historical data is used to build the function which is then validated using available traffic count.

Apart from being simple and easy to follow, the nature of these functions resemble regular traffic growths, where, traffic have higher growth during the initial years after opening of the transportation facility but decrease gradually to nearly saturation point towards the design year. The functions can be fitted using any of the variables which

have effect to traffic growth like employment, population, number of registered vehicles, gas receipts and others *(51)*. The growth trend of the mentioned factors at particular location will have direct reflection of traffic growth trend. For instance, employment trends traditionally influence traffic generation and can be used as a substitute wherever counts are not available. When the economy is robust and jobs are plentiful, there is an increase in traffic. Conversely, when employment trends are downward, traffic volumes generally decline. On the other end, the number of motor vehicle registrations can be the good indicator of traffic stability or growth hence used for projection *(52)*.

This study therefore evaluates these five functions as the immediate potential projection approach at locations where forecasting model is not available. In developing these model functions, historical traffic data ranging from 1970's to 2007 from twelve locations in Palm Beach County are used for fitting the distributions. The locations include three along Indiantown Road in Jupiter, three along Okeechobee Road in West Palm Beach, four along US-441, one along SR-800, and one along SR-710.

3.2 Theory of Small Scale Traffic Projection

The idea of developing methodologies for small area traffic projection has been foreseen by different researchers at different study levels and approaches. Lee *et al (51)* proposed an alternative methodology to model and forecast network traffic for planning applications in small and medium-sized communities where resources hinder the development and applications of 4-step models (trip generation, distribution, model split

and traffic assignment). The methodology known as Path Flow Estimator used land uses, traffic counts and origin-destination tables as constrains. The resulting trip table from the methodology reflected the trip-making propensity of the land use configuration in the study area, thus making evaluation of different land use strategies possible. The study recommended future research to enhance the proposed approach such that the impacts of long-range, area-wide growth can be modeled within the same framework. Perone *et al (53)* used cumulative analysis for generating traffic by expected development on a parcel-specific basis for small city in western Oregon, in which a travel demand model was not available. In this methodology, a GIS-based buildable lands inventory based on tax assessor records was updated and queried to identify vacant and underdeveloped parcels inside the city limits. GIS tools were used to allocate growth based on the location and attractiveness of available land, its access to urban services, and plan designations. Though the methodology was simple but the use of GIS can restrict its application to locations where GIS maps are not available. Anderson *et al (54)* developed a direct demand forecasting model using multiple linear regressions in a small urban community to predict future traffic volumes to support transportation planning. The model was intended as an alternative to produce future traffic volume compared traditional travel demand models with less reliance on computer applications of the four traffic demand modeling process. Some of the variables used in the regression included highway functional class, number of lanes, population, employment and mobility information. The developed regression was capable of explaining 82% of the variability in the traffic counts.

The use of small scale traffic projection can also be useful in trip generation analysis like in traffic impact studies. The construction of small stores, retail shops, small residential houses always impact small radius which eventually involve analysis of isolated intersections or short corridor. Kikuchi *et al (55)* examined the trip generation characteristics of neighborhood- and community-level shopping centers. The study examined three approaches to the estimation of trip generation rate; based on the fixed features of the shopping center, based on the use of internal capture rate and based on the sum of weighted attraction of individual stores. They found that, the model based on fixed features was the most reasonable model for practical application.

3.3 Technicality of the Selected Projection Functions

The selected modeling distributions can be categorized into three main groups, logistic, exponential and power distributions. These distributions have some common general characteristics but vary in shape and patterns. They also differ in the limits of lower and upper values of their distributions. It is from these characteristics (shape, limits and relevancy) which led to be tested in traffic projection.

3.3.1 Logistic Function

As will be discussed in next sections, logistic function became the best for traffic projection due to its s-shaped distribution, resembling regular traffic growth. Logistic distribution can be termed as the combination of the open and bounded exponential functions as elaborated in the sketch in Figure 3.1;

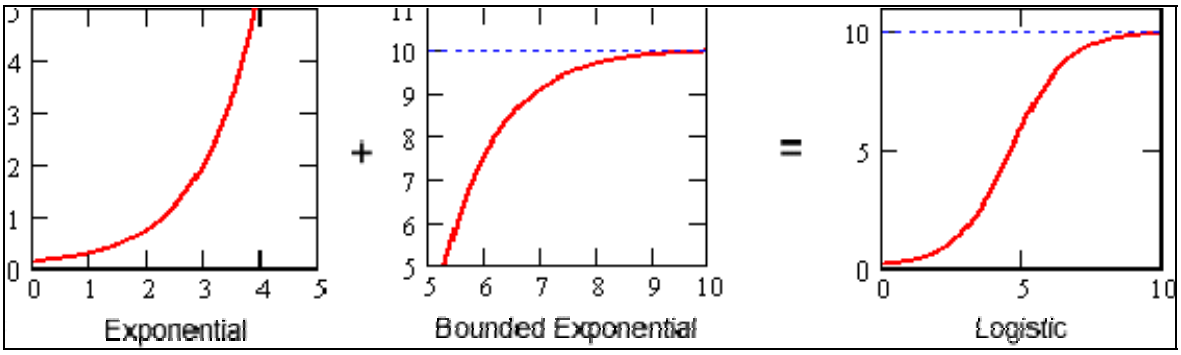


Figure 3.1: Exponential and Logistic Functions General Form

Logistic distribution graph follows an S-shaped trend called sigmoid. The S-shape may either rise from the x-axis to the limiting value, or drop from the limiting value to the x-axis. The limiting value may be raised and lowered, and the rate at which the curve travels between the two horizontal asymptotes may vary, but this basic sigmoidal shape is found in all logistic graphs. Considering sketches in Figure 3.1, the first open exponential growth is at an increasing rate. Since the growth is exponential, the growth rate is actually proportional to the size of the function's value. The second exponential growth is usually called bounded exponential growth. It takes a decaying exponential and subtracts it from a fixed bound. As the decaying exponential dies out, the difference rises up to the bound. This kind of function models growth that is limited by some fixed capacity. The logistic functions therefore combine the first kind of exponential growth, when the outputs are small, with the second kind of exponential growth, when the outputs near capacity. This then ends up with a “Logistic functions” which basically can be termed as a capacity limited exponential growth.

The logistic function used in this study is a three parameter function of equation (3.5) as shown below.

$$V = \frac{C}{(1 + A * \exp(-BX))} \tag{3.6}$$

Where V = the projected traffic volume

The three parameters of the logistic produce its characteristic behavior. The parameters A and $\exp(-BX)$ are simply the y-intercept and the base of the component exponential function $A * \exp(-BX)$. As in other exponential functions, the base $\exp(-BX)$ is restricted to positive values. The significance of the parameter C depends on the behavior of this exponential function. In the short term, when x is near 0 and $\exp(-BX)$ is near 1, the value of the function is approximately $\frac{C}{(1+A)}$, regardless of the exponential's larger behavior. If $\exp(-BX)$ decays ($\exp(B) > 1$), the denominator approaches 1, and the function as a whole grows to the value of the numerator C. It is this latter behavior, in which the function rises up to and eventually levels off at a constant horizontal asymptote that is seen as "Capacity-limited" growth. Indeed, the function never exceeds the value C. Thus, the parameter C is often called the capacity limiting value or, in the description of traffic projection, the carrying maximum capacity.

Linking the logistic function characteristics with traffic projection, the small initial growth rates which then accelerate up to a point of inflection, after which the growth slows down and eventually approaches a limiting value, is a typical resemblance for

normal traffic growth. If the traffic data can display the entire progress of a logistic function's S-shaped growth, then we can make the choice of a logistic model a typical one for traffic projection under normal traffic growth. In the early stages of logistic growth, the outputs are rather small. This is because the exponential in the denominator is rather large. In particular, it is much larger than 1, and adding the 1 + ... in the denominator has little affect on the output. Likewise, the latter part of logistic growth can be difficult to distinguish from bounded exponential growth. Near its limiting value, logistic growth behaves approximately like the function $\frac{C}{(1+A*\exp(-BX))}$. If we take for instance, C=54600 (maximum LOS E daily capacity for 6-lane roadway) as our limiting value and look at the latter stages of the data's growth, we once can see an approximation of exponential growth. The parameter 'C' is just the data's limiting value. Unless the data displays the latter stages of logistic growth, and the limiting value is quite obvious, it takes some trial and error to find a value for this parameter that will fit the data.

Concluding about the use of logistic function in future traffic projection, it is better to analyze the growth stages with respect to traffic patters. Apart from applying the whole function all together, the shape of the logistic function allows splitting the growth rates into phases. The idea of calculating growth rates into stages (phases) is an essential element in transportation planning and management. The construction of any highway facility involves use of funds in which well planned implementation phases will reduce

the cost. Phasing of growth projections takes into account the elasticity of travel demand, initial price of travel (user costs, including operating, and travel time and safety costs); the state's traffic growth projection; and price elasticity to project future traffic volume in each funding period *(57)*. The phasing traffic projection extracted from this function can also be useful for Intelligent Transportation System where technology changes would likely be implemented before the next phase of construction *(58)*. With the splitting of the forecasting term, the uncertainty of hidden traffic flow can be traced and solved at intermediate stages.

3.3.2 Power Function

Power function has the general formula $y=Ax^B$, where y is the dependent variable, x the independent variable, "A" and "B" are constants. In the case of traffic volumes, y is the traffic volume; x is the number years from the base year. The constant "b" is the scaling exponent while "a" is initial defining value, usually the value of traffic volume at the base year. The scaling factor "b" is the most important output from the power function, since it is the determinant of size of traffic to be projected from the base year. It is generated automatically when power function is fitted as shown below;

$$y = Ax^B$$
$$\ln(y) = \ln(A) + B * \ln(X) \qquad (3.7)$$

If the graph is plotted of 'ln(Y)' vs. 'ln(X)' then the gradient of the graph will be the value of scaling factor 'B'.

The lower the scaling factor, the smaller the difference between the base year traffic and the projected design year traffic and vice versa. In order to match the available traffic count, the scaling factor can be fine-tuned for the model to match the targeted volume in terms of validation. Validation of the function can only be done on the scaling factor if prediction is higher or lower than the expected traffic after considering other conditions. After fixing the scaling factor "b", the constant term or initial value "a" is picked from the data available for any year adopted as the base. The general form of the power function is shown in Figure 3.2, reflecting regular trend expected for traffic growth on a corridor or at particular location as the land use is developed and community growth matures.

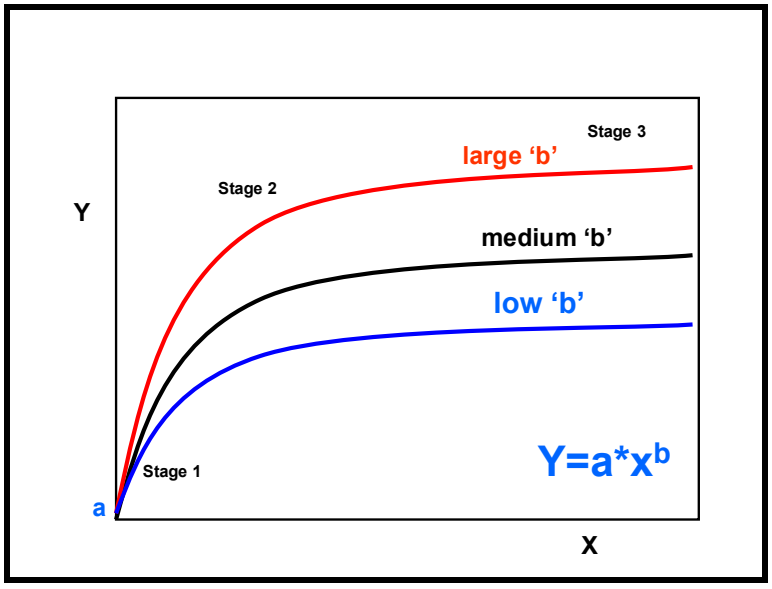


Figure 3.2: Power Function General Form

As for logistic function, the growths from power function can be divided into three major stages, stage 1 where the traffic grow at higher rate after opening of new highway, the at stage 2 the growth gradually start to decrease and at stage 3 when the highway is

expected to have less growth. The first stage has higher rate growth because drivers are attracted to the new or improved highway. This attraction could be based on factors like increased capacity (less congested route), better driving surface (new pavement), shorter travel time (less delay), safer road and other potential attractions. Many of these drivers will divert to the improved facility from other congested regional roads. Similarly, some drivers who formerly traveled in off-peak hours on the facility to avoid severe congestion will shift back to peak hours, adding to peak hour volumes when congestion is most noticeable to commuters *(56)*. In general, the use of power function allows gradual growth at later stages making it preferable over the later *(52)*.

3.3.3 Transformed (log) Power-Exponential Function

The log transformed power-exponential function was also tested for potential traffic projection. This function basically takes the product of power and exponential and then transforms it. The combination of the power and exponential make the function to be a three parameter instead of two. The distribution is presented as $V=\ln(AX^B * \exp(CX))$ which can be expanded as $\ln(A)+B\ln(X)+CX$.

3.3.4 Other functions

The other two functions tested in this study for possible use in traffic projected are either derived from power function, exponential function or are the log transformed power and exponential functions. These other functions tested include log-power which is logarithmic transformation of the power function, e.g. $\ln(AX^B)$. The other function is

combined power function and exponential (product of power and exponential) in the form of $AX^B * \exp(CX)$.

3.4 Projection Modeling Data

The corridor traffic data used in generating the traffic projection functions were obtained from Florida Department of Transportation (FDOT) District 4, Traffic Information CD. Only locations with continuous traffic data of 29 years or above were downloaded. This means only location with historical data from 1978 or before was selected. The selected roadway locations are currently either, 4-lanes, 6-lanes or 8-lanes sections. The details for each roadway locations are as follows;

Indiantown Road (SR-706):

- East of Center St (data from 1976-2007, LOS E capacity = 4920 vph)
- East of Turnpike Entrance (data from 1970-2007, LOS E capacity = 3270 vph)
- West of Dixie Highway (data from 1970-2007, LOS E capacity = 3270 vph)

Okeechobee Road:

- East of Florida Turnpike (data from 1976-2007, LOS E capacity = 6360 vph)
- West of Florida Turnpike (data from 1978-2007, LOS E capacity = 4920 vph)
- East US-441(data from 1970-2007, LOS E capacity = 4920 vph)

US-441 (SR 7):

- South of Forest Hill Blvd (data from 1970-2007, LOS E capacity = 4920 vph)
- N. of Boynton Beach Blvd (data from 1970-2005, LOS E capacity = 4920 vph)
- North of Atlantic Blvd (data from 1974-2005, LOS E capacity = 4920 vph)

- North of Clintmoore Road (data from 1970-2005, LOS E capacity = 4920 vph)

Beeline Highway: South East of SR 706 (from 1978-2005, LOS E capacity = 4920 vph)

SR 800: East of US-1 (data from 1972-2004, LOS E capacity = 3270 vph)

3.5 Fitting projection models

Using the data described above, the optimization program was coded in matlab (see Appendix) utilizing FMINCON which finds a constrained minimum of a function of several variables starting at an initial estimate. One part of the data was used for training and the other part for validation. The given parameters for each model were existing traffic volume and number of year for each data point (X). The output coefficients where were ln(A), B, C and A. depending on respective model. These coefficients where related to each model as follows;

$$\text{Power Function}: V = AX^B = \exp(\ln A) * X$$
$$\text{Log Power}: V = \ln(AX^B) = \ln(A) + B\ln(X)$$
$$\text{Power - Exponetial}: V = AX^B * \exp(CX) = \exp(\ln A) * X^B * \exp(CX)$$
$$\text{Log Power - Expo}: V = \ln(AX^B * \exp(CX)) = \ln(A) + B\ln(X) + CX$$
$$\text{Logistic}: V = \frac{C}{(1 + A * \exp(-BX))}$$

The combined plots showing existing data and the fitted models are shown in Figure 3.3(a) to Figure 3.13(a). Deep analysis from these fitted plots showed logistic function and log transformed power-exponential function giving good prediction compared to others. Figure 3.3(b) to Figure 3.13(b) shows the plots for existing, logistic and log power-exponential function. Among the functions, logistic function showed more

reasonable prediction compared to log power-exponential function. The logistic function was therefore selected as the most appropriate distribution for projection among the tested functions.

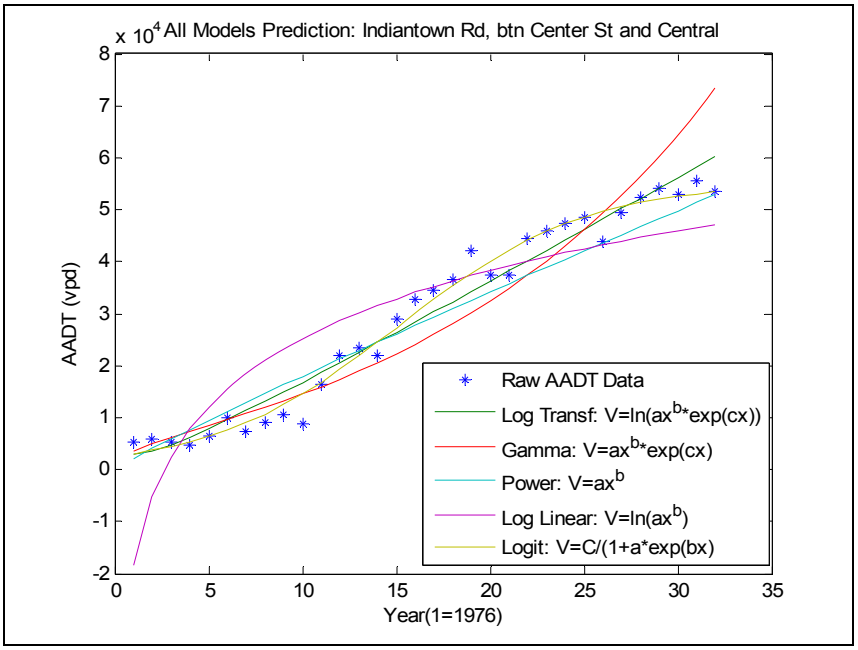


Figure 3.3(a): All Models Prediction at Indiantown Rd, W of Center St

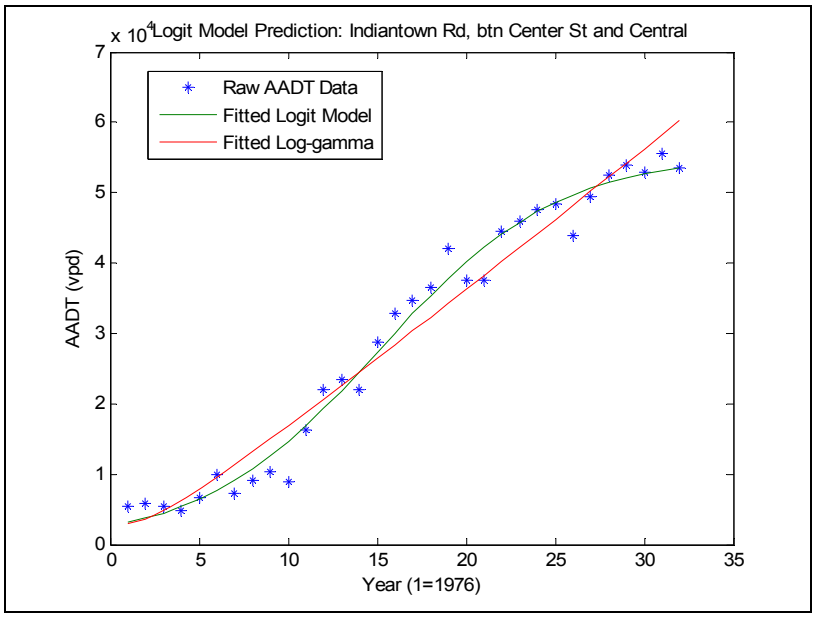


Figure 3.3(b): Logit and Log Power-Expo Prediction at Indiantown Rd, W of Center St

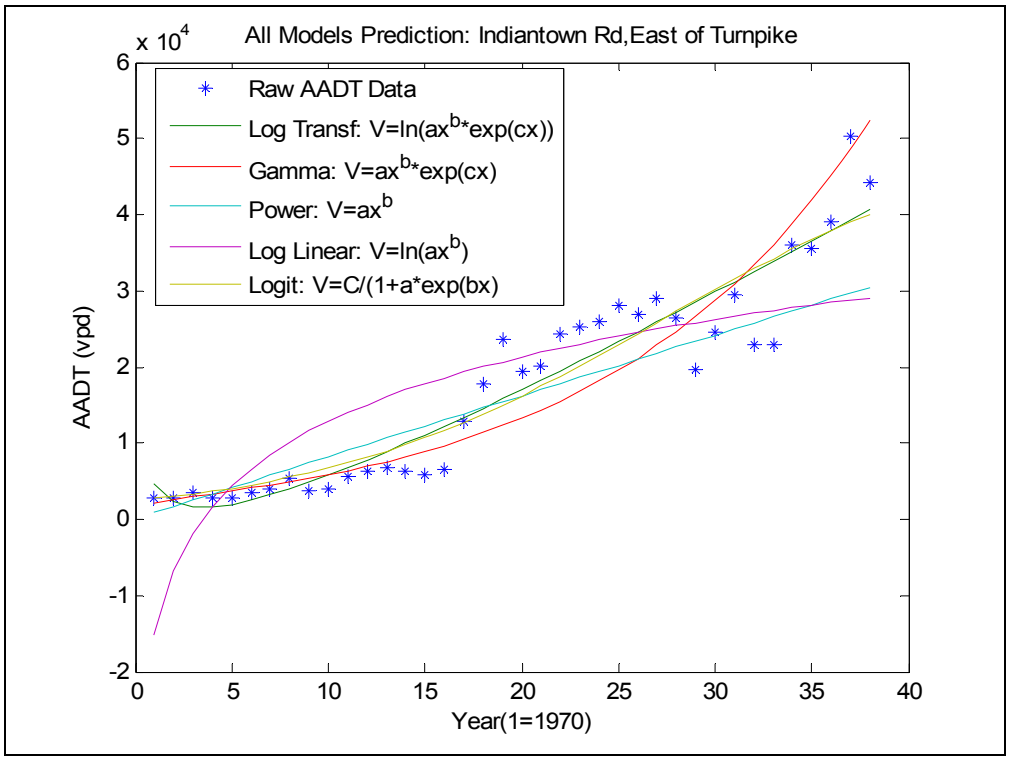


Figure 3.4(a): All Models Prediction at Indiantown Rd, E of Turnpike

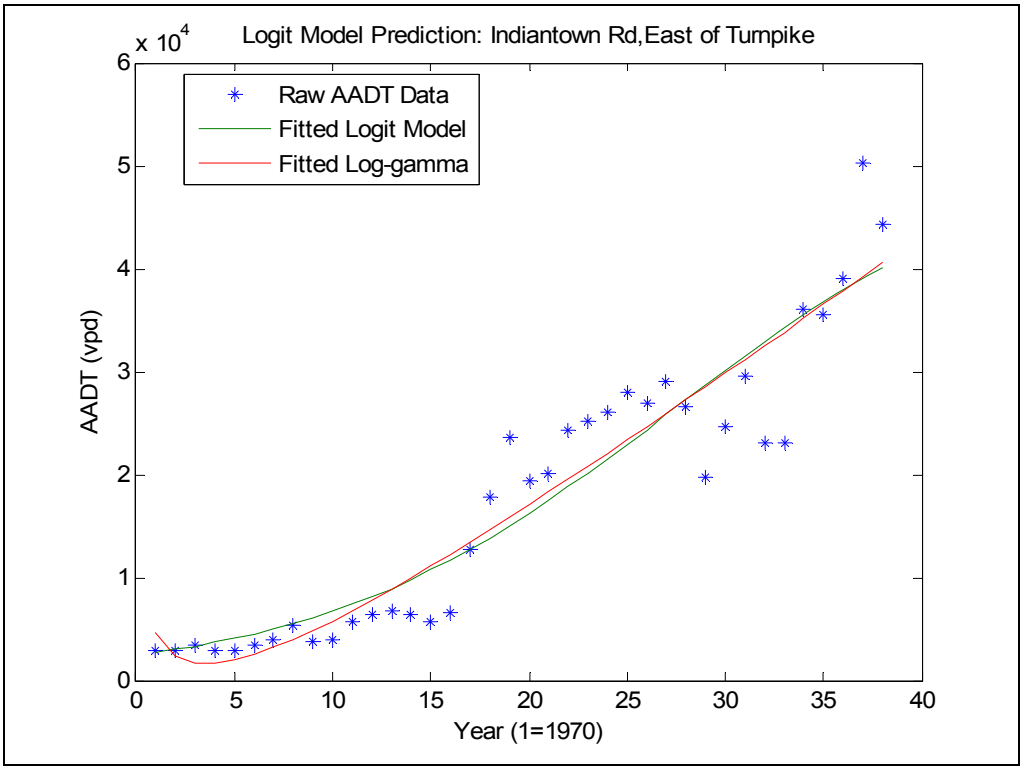


Figure 3.4(b): Logit and Log Power-Expo Prediction at Indiantown Rd, E of Turnpike

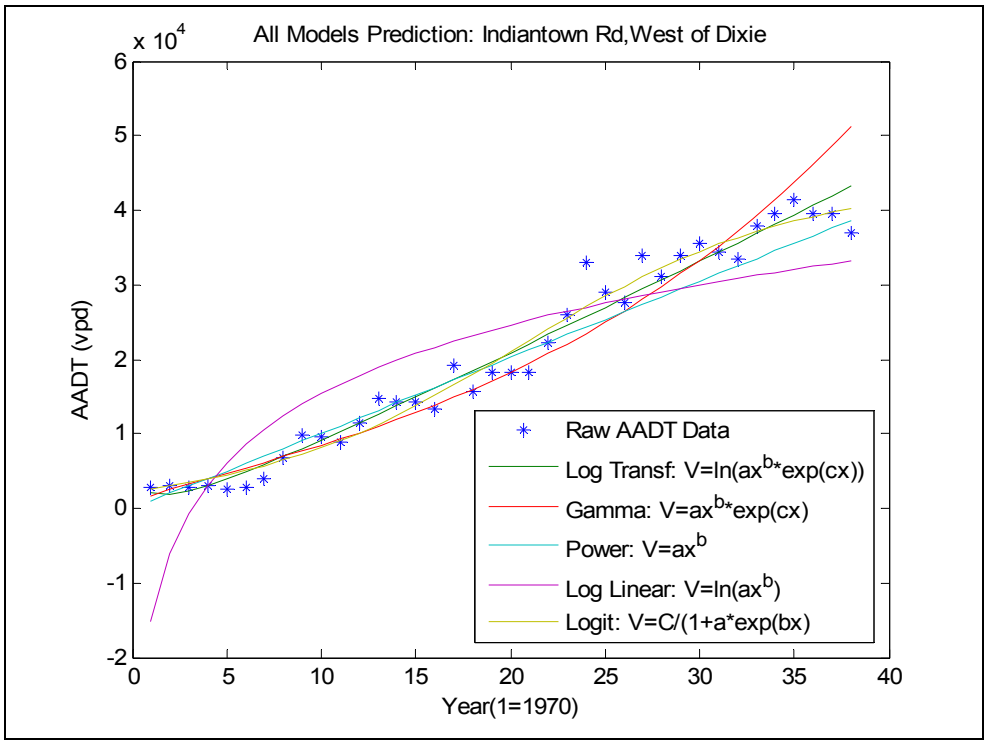


Figure 3.5(a): Models Prediction at Indiantown Rd, W of Dixie Hwy

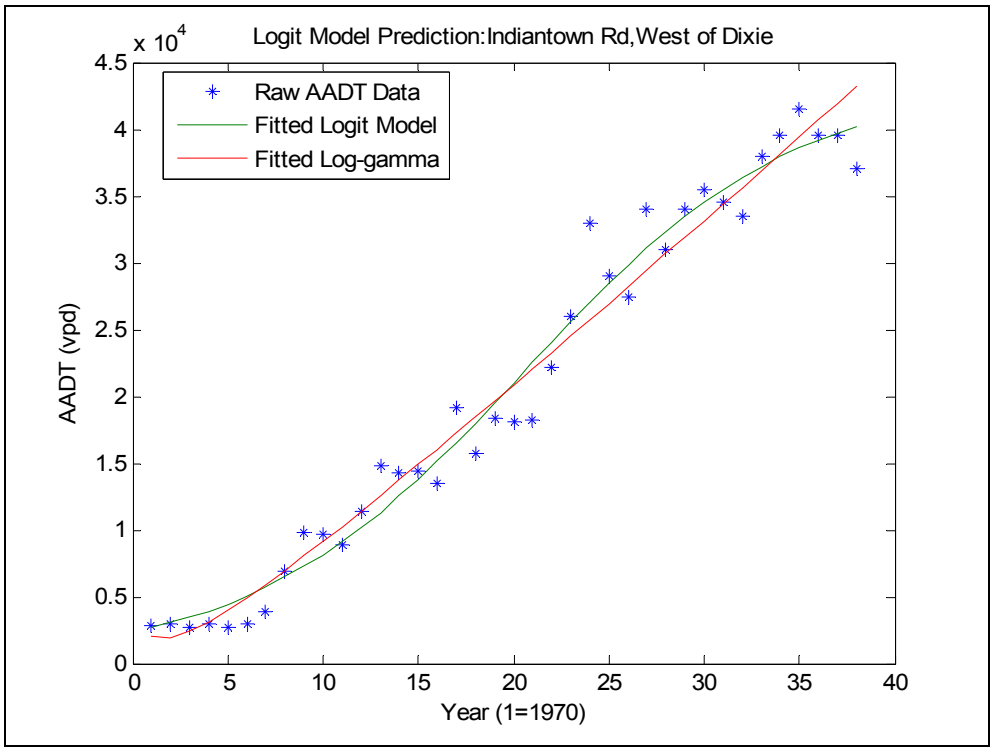


Figure 3.5(b): Logit and Log Power-Expo Prediction at Indiantown Rd, W of Dixie Hwy

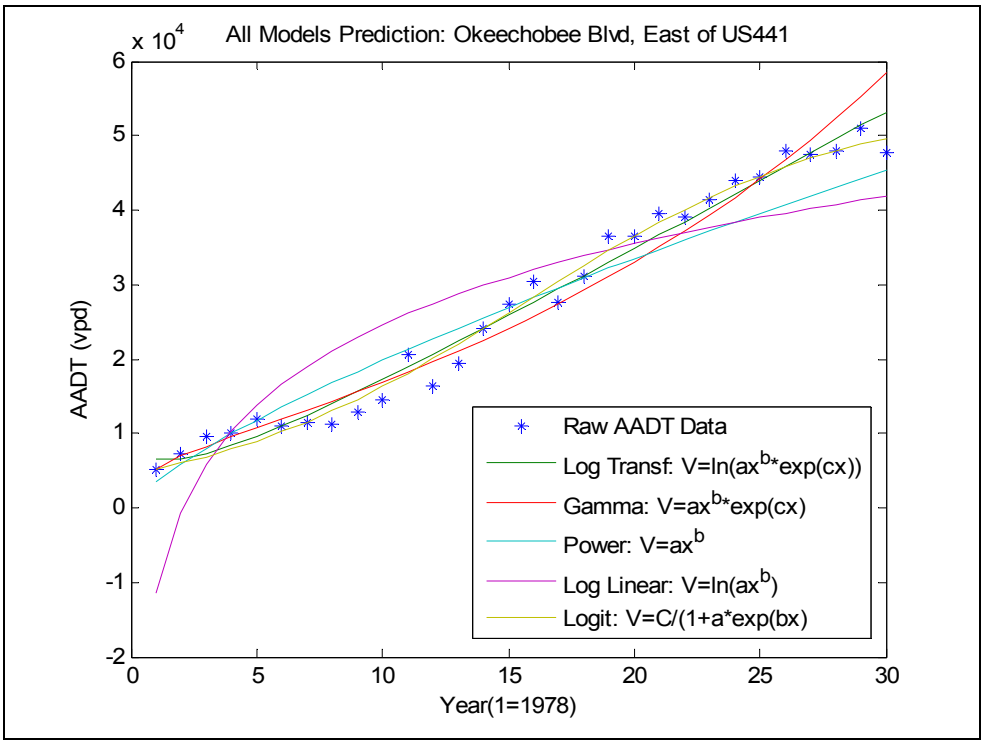


Figure 3.6(a): All Models Prediction at Okeechobee Blvd, E of US-441

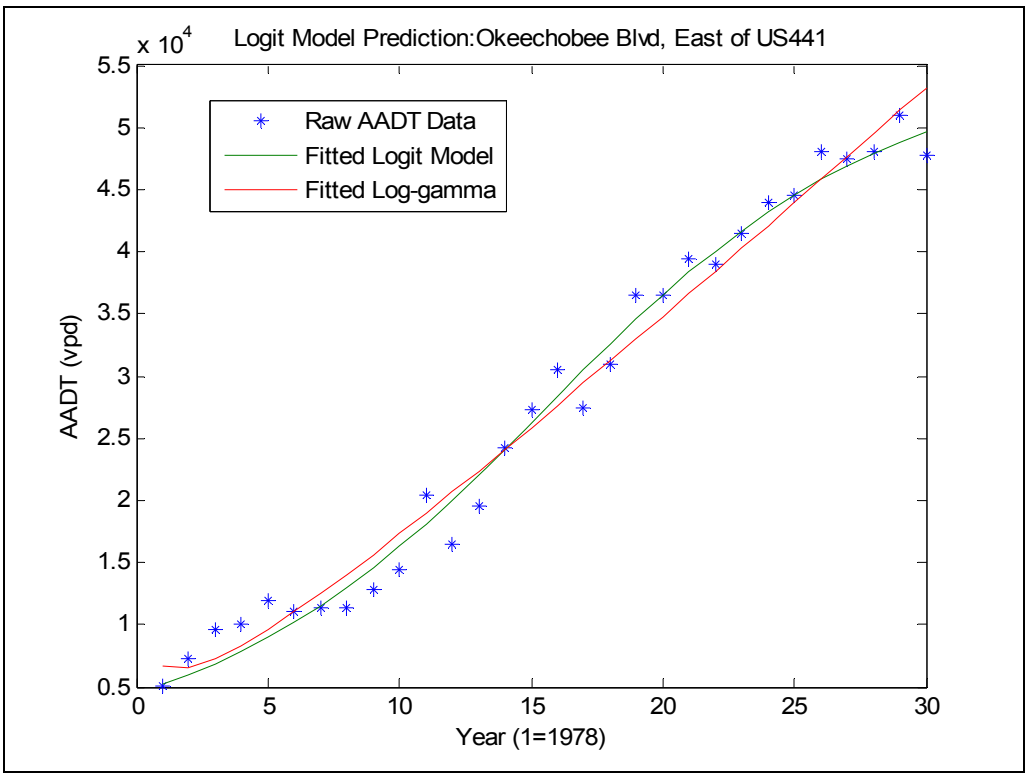


Figure 3.6(b): Logit and Log Power-Expo Prediction at Okeechobee Blvd, E of US-441

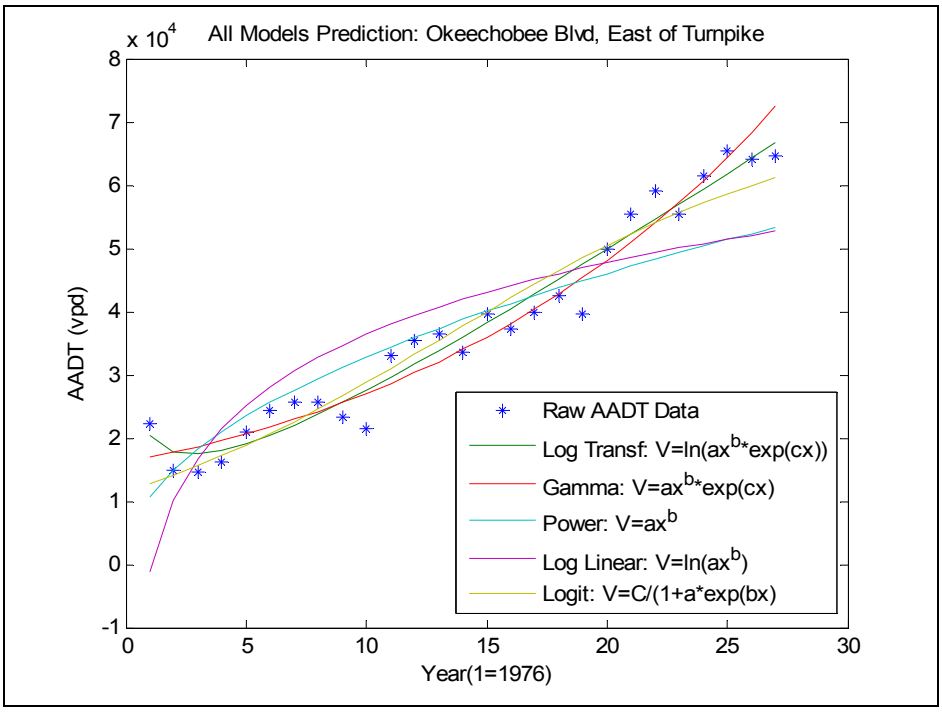


Figure 3.7(a): All Prediction at Okeechobee Blvd, E of Turnpike

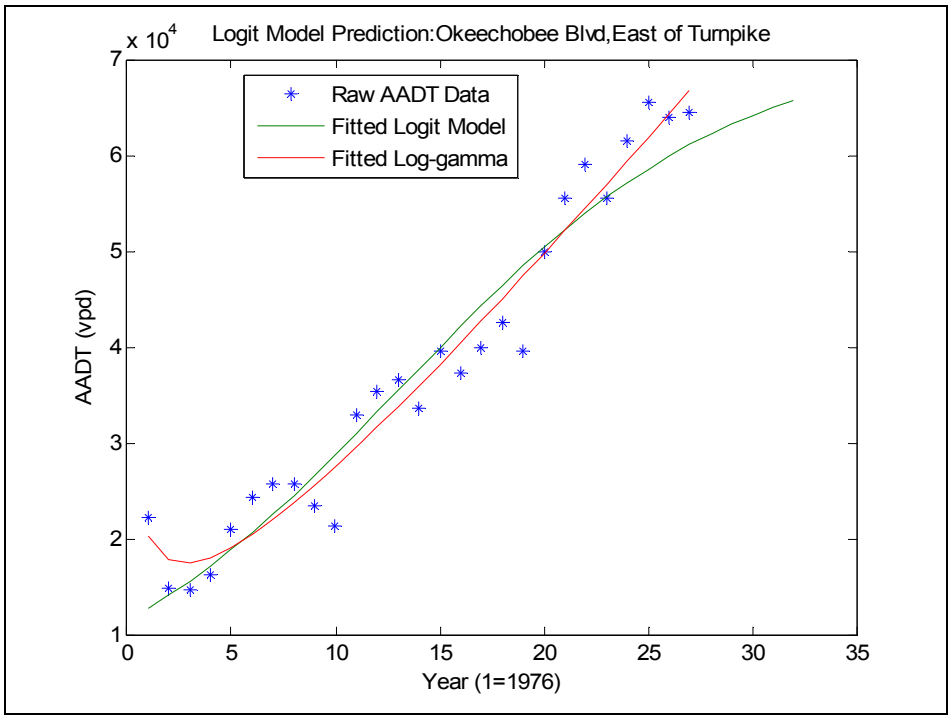


Figure 3.7(b): Logit and Log Power-Expo at Okeechobee Blvd, E of Turnpike

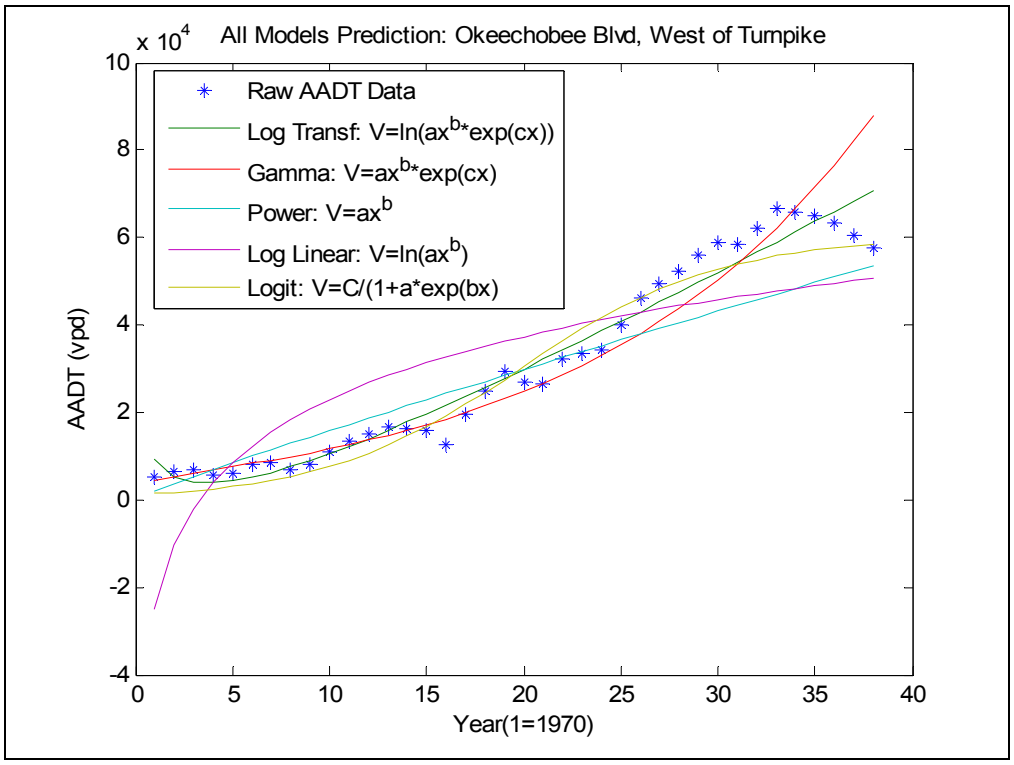


Figure 3.8(a): All Prediction at Okeechobee Blvd, W of Turnpike

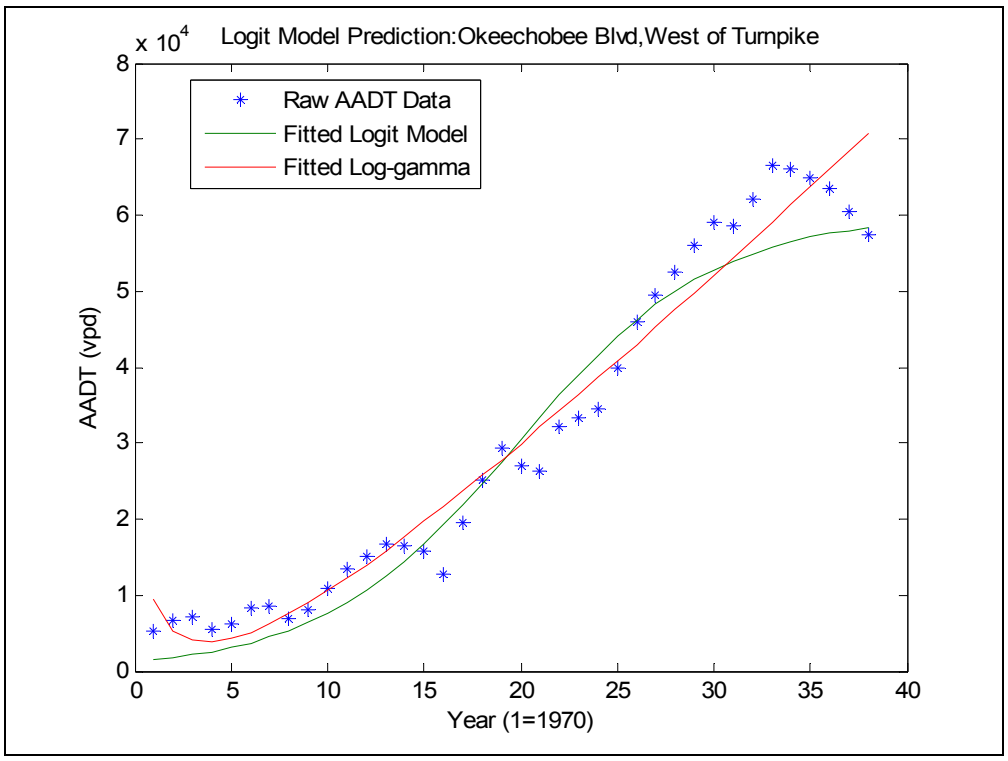


Figure 3.8(b): Logit and Log Power-Expo at Okeechobee Blvd, W of Turnpike

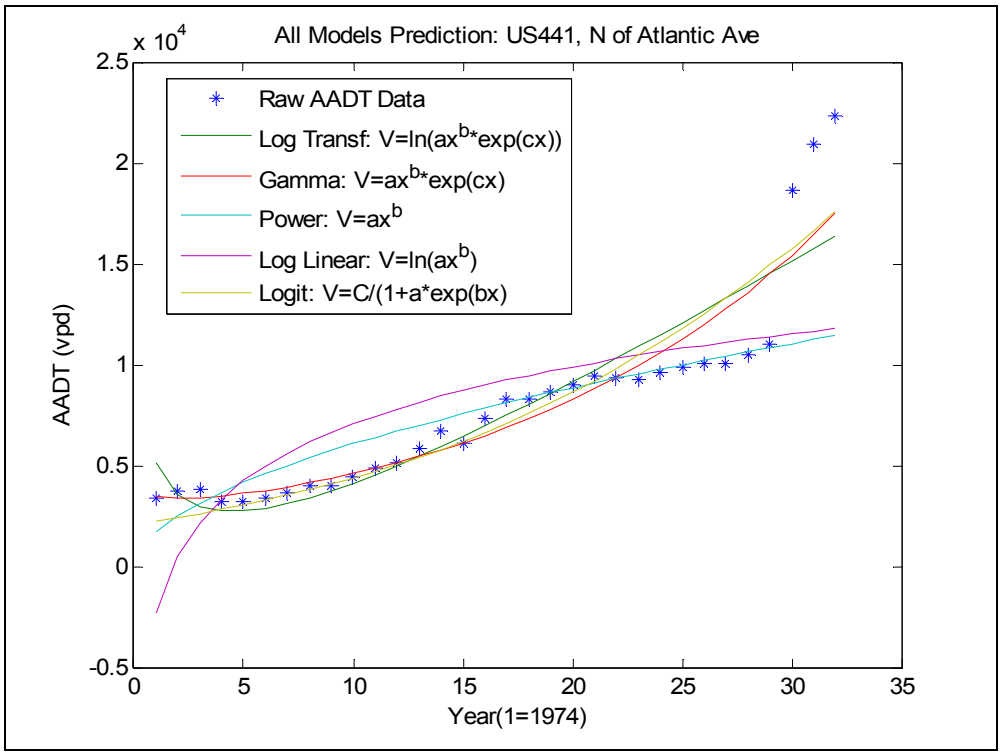


Figure 3.9(a): All Prediction at US-441, N of Atlantic Ave

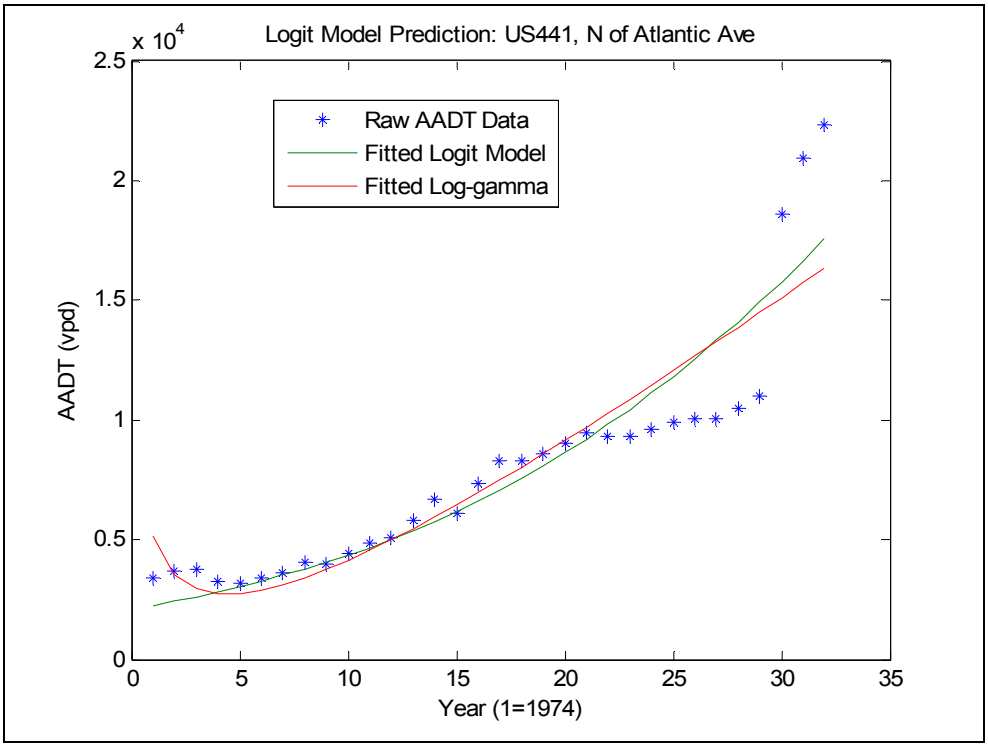


Figure 3.9(b): Logit and Log Power-Expo at US-441, N of Atlantic Ave

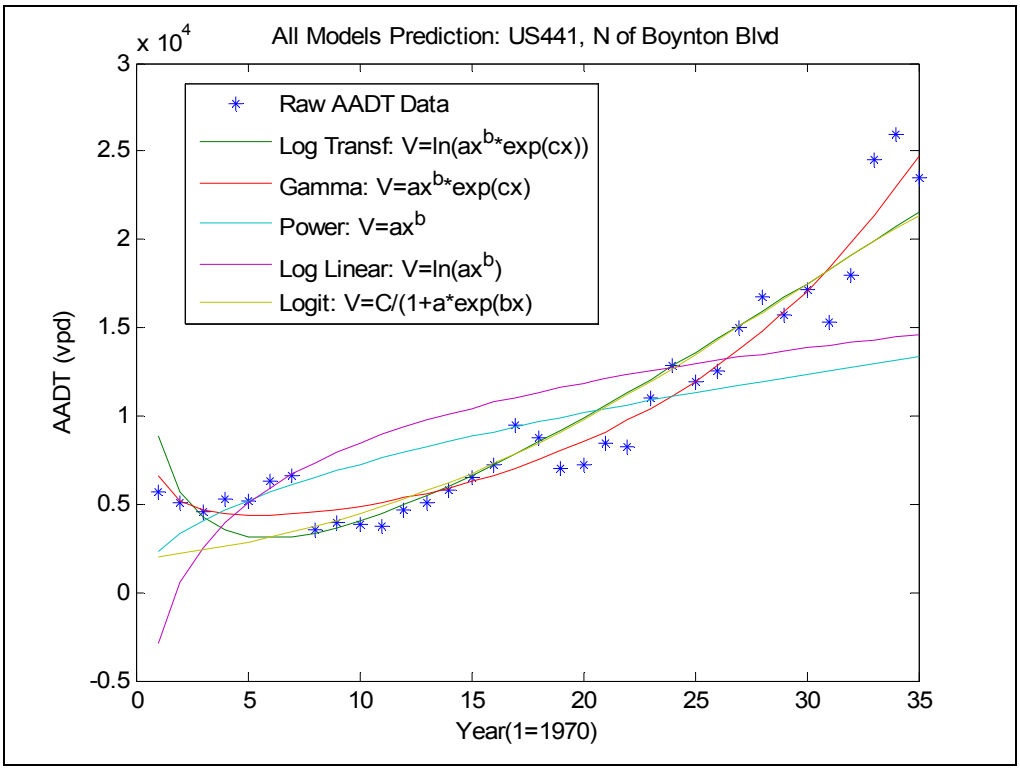


Figure 3.10(a): All Prediction at US-441, N of Boynton Beach Blvd

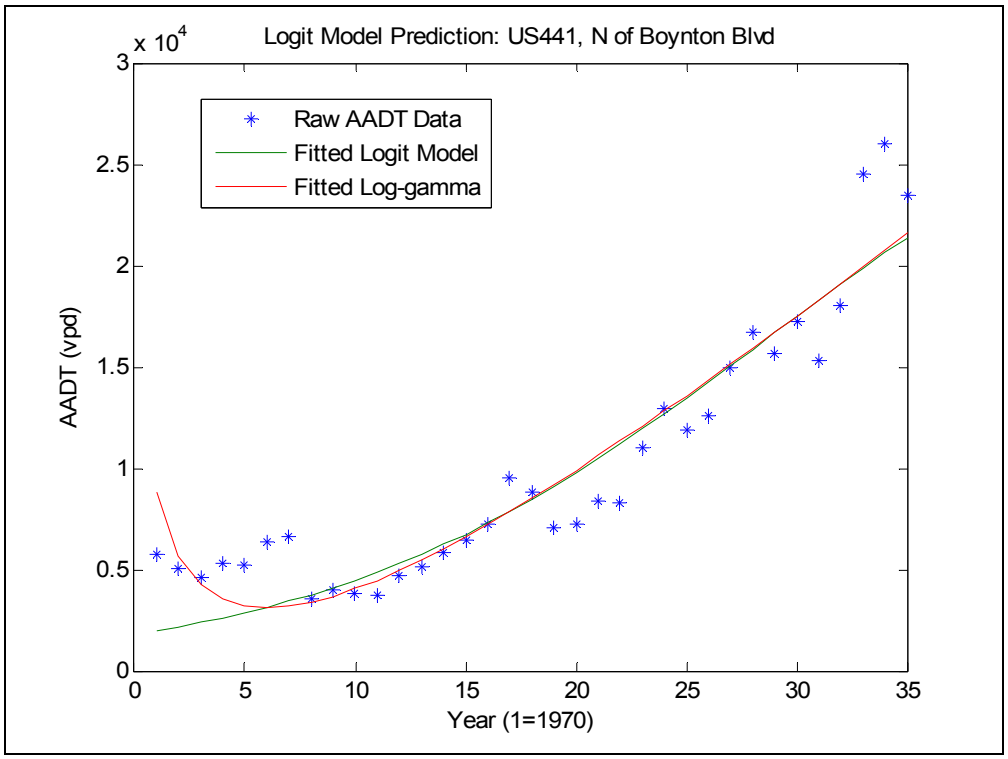


Figure 3.10(b): Logit and Log Power-Expo at US-441, N of Boynton Beach Blvd

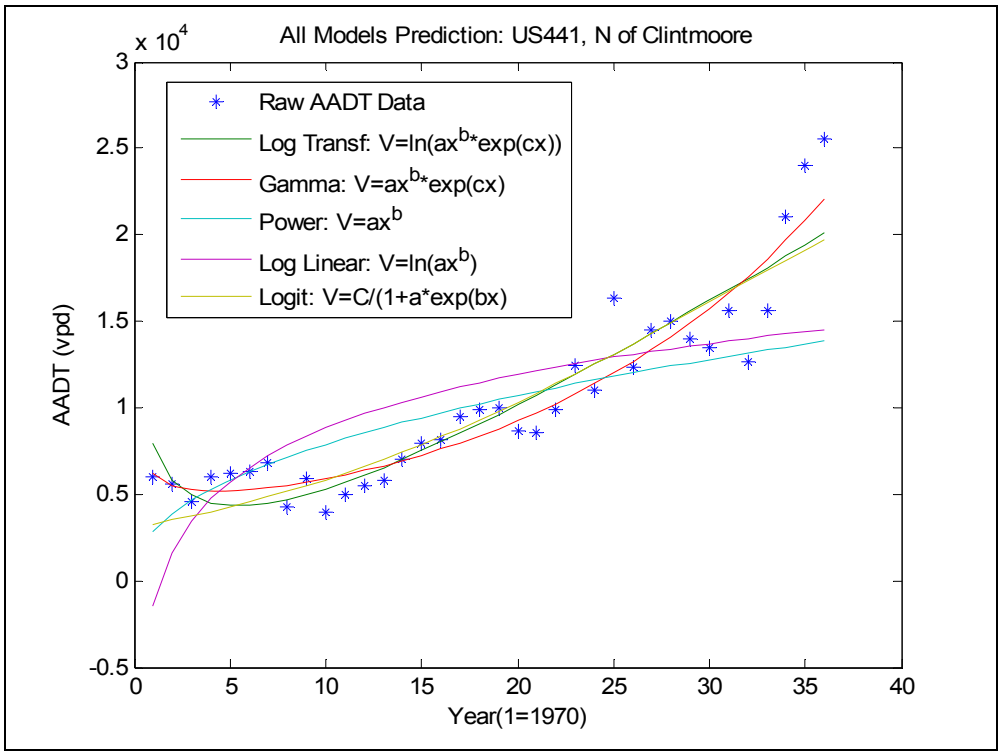


Figure 3.11(a): All Prediction at US-441, N of Clintmoore Rd

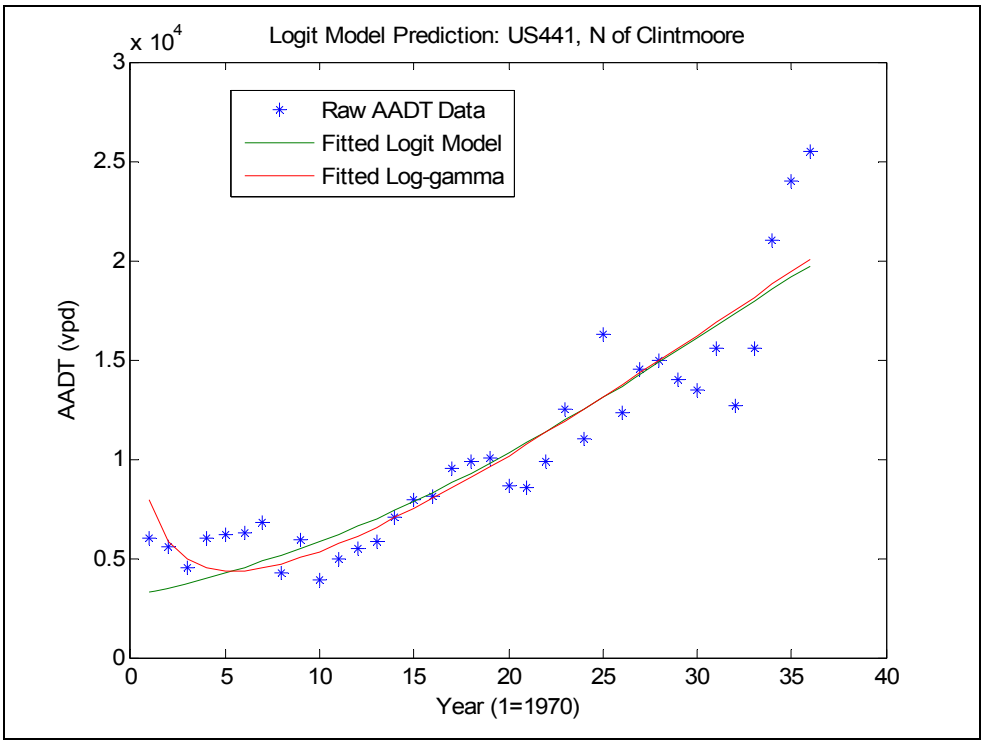


Figure 3.11(b): Logit and Log Power-Expo at US-441, N of Clintmoore Rd

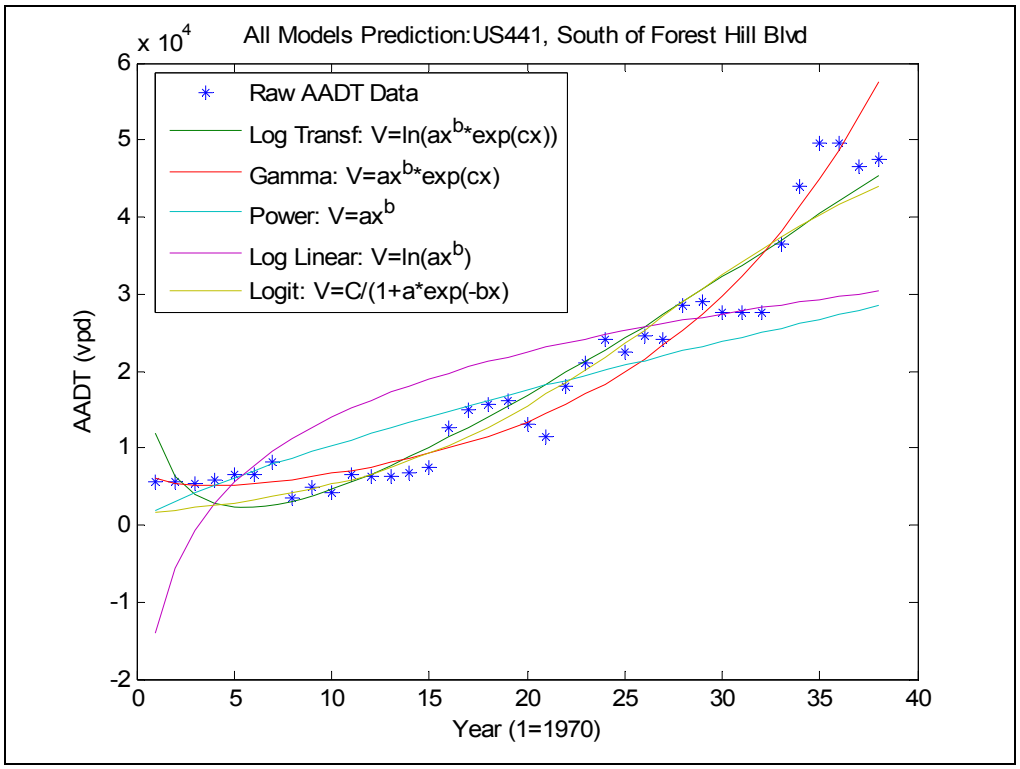


Figure 3.12(a): All Prediction at US-441, S of Forest Hill Blvd

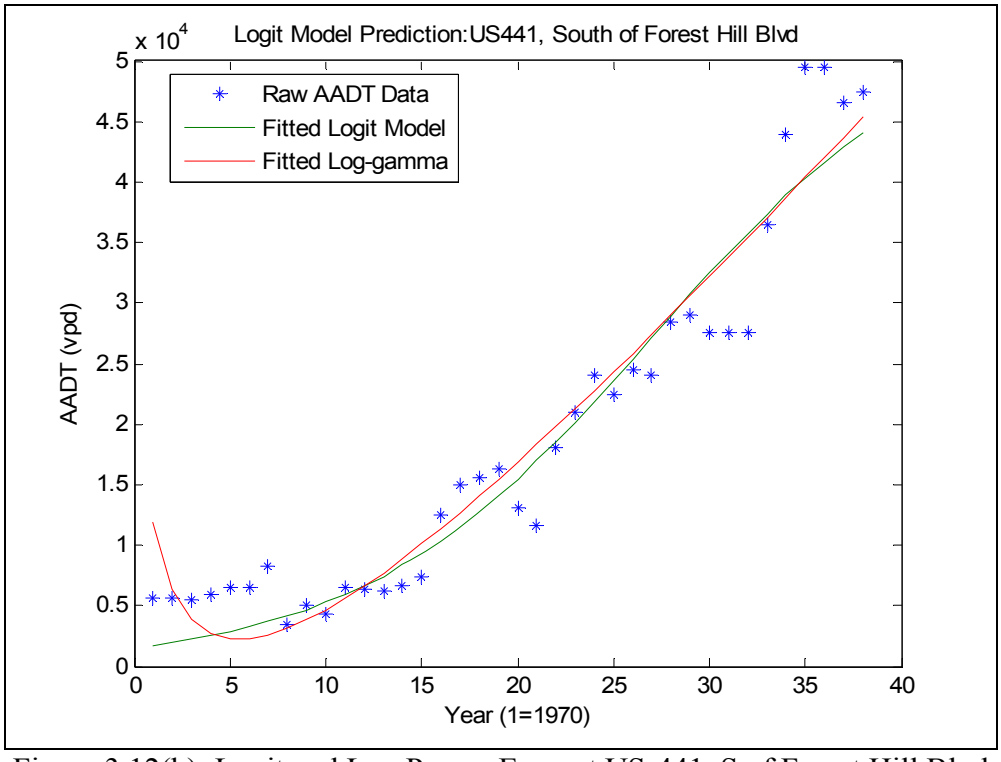


Figure 3.12(b): Logit and Log Power-Expo at US-441, S of Forest Hill Blvd

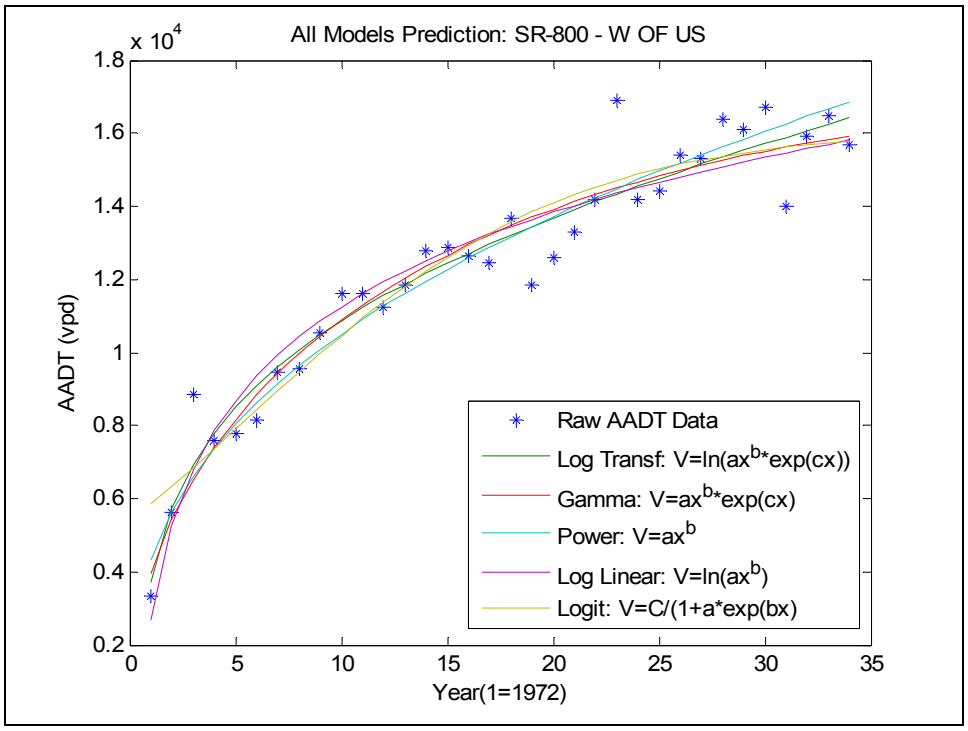


Figure 3.12(a): All Prediction at SR-800, W of US-1

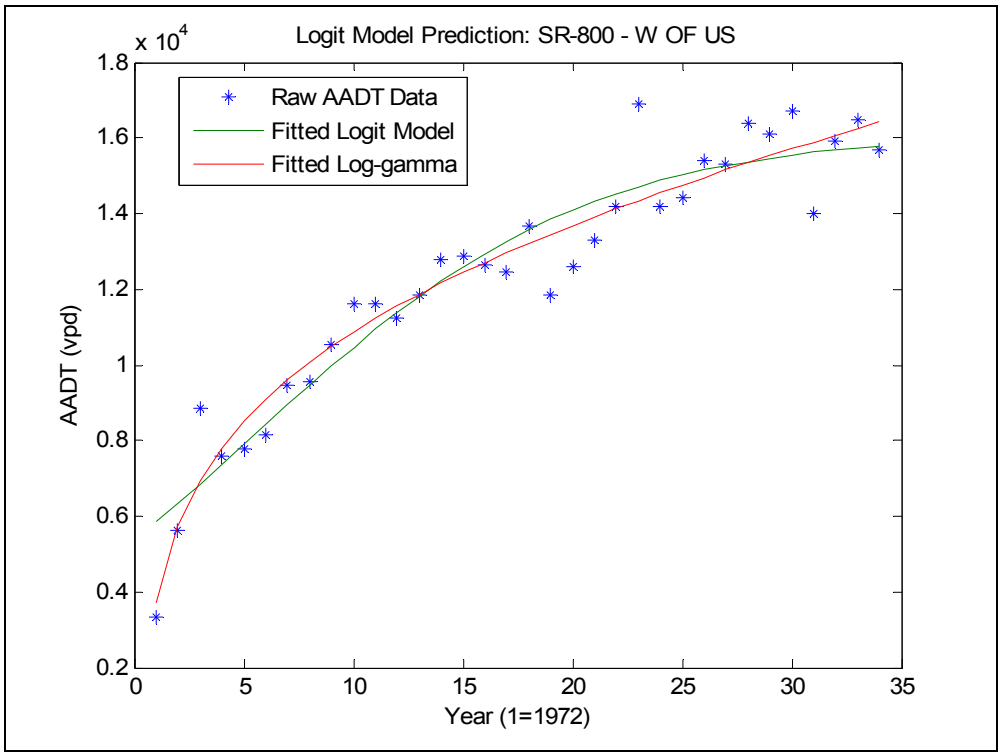


Figure 3.12(b): Logit and Log Power-Expo Prediction at SR-800, W of US-1

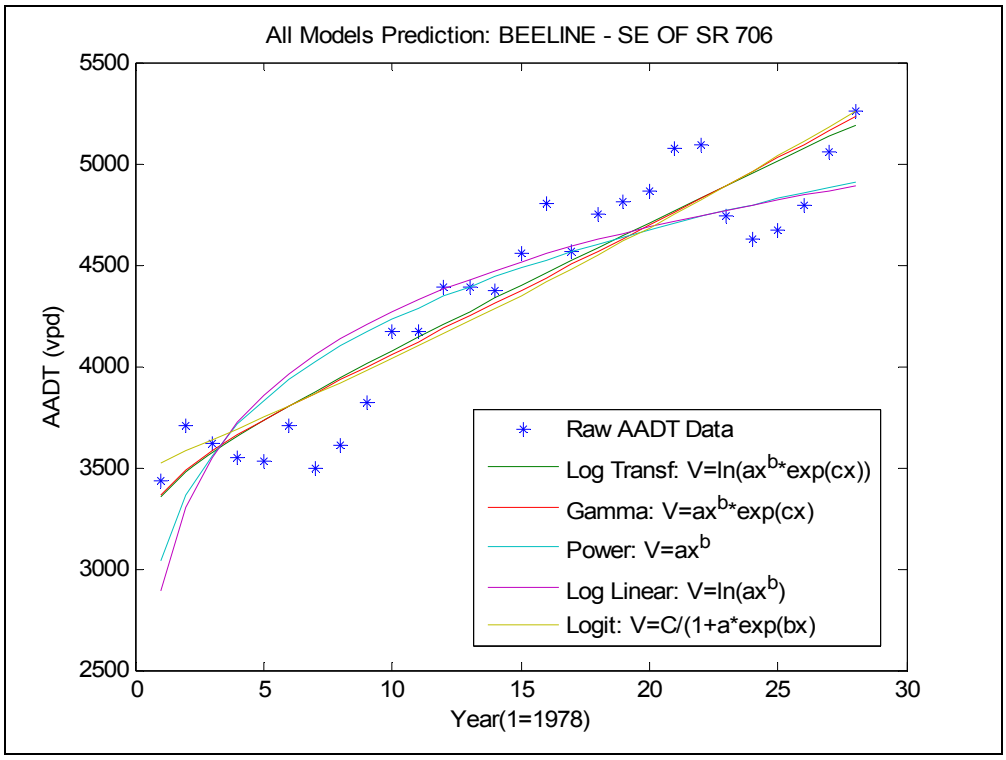


Figure 3.13(a): All Prediction at Beeline Hwy, SE of SR-706

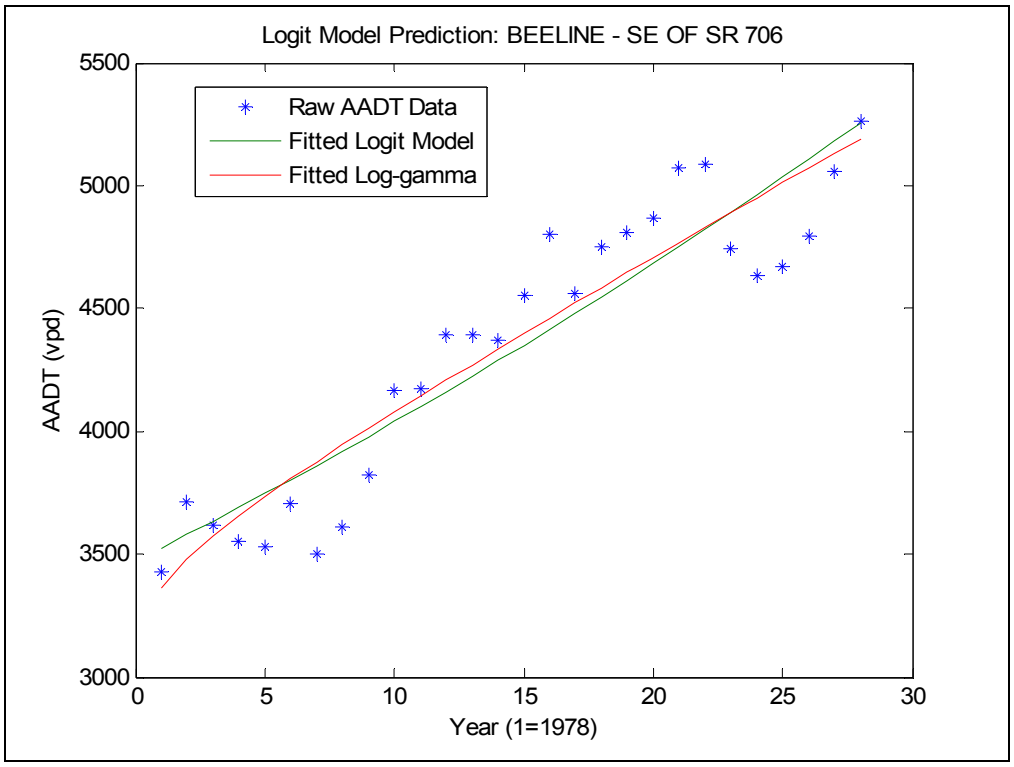


Figure 3.13(b): Logit and Log Power-Expo Prediction at Beeline Hwy, SE of SR-706

3.5.1 Fitted Logistic Model Coefficients

As observed from Figures 3.3 to 3.13, logistic function gave good prediction compared to all other models. Table 3.1 shows the coefficients of fitted Logistic Model at three locations;

Table 3.1: Fitted Logistic Model Coefficients, t-values

Location	Coefficient	$\frac{C}{(1+A*\exp(-BX))}$	t-values
Indiantown Road East of Center St	B	0.2004	13.677
	C	55275	34.766
	A	20.7061	5.800
Indiantown Road East of Turnpike	B	0.1113	12.970
	C	52000	22.961
	A	20.4464	6.694
Indiantown Road West of A1A	B	0.1399	16.704
	C	43644	28.654
	A	17.6708	10.696

3.5.2 Calibration of Logistic Function Formulation

The optimal traffic volume estimates are obtained by fitting the proposed formulation with the analytical optimal traffic volume by utilizing the nonlinear regression method described below. The optimal volume is treated as a random variable with mean estimate denoted as $\hat{V}$. The estimation equation can be written as $V = \hat{V} + \varepsilon$, where, without loss of generality, the estimation error is assumed normally distributed, i.e. $\varepsilon \approx N(0, \sigma^2)$. Under the normality assumption, data fitting can be easily carried out using the nonlinear regression method based on the following quadratic criterion:

$$\min \sum \left[V_k - \hat{V}_k (\beta, X_k) \right]^2 \qquad (3.8)$$

Where V_k = optimal traffic volume observation obtained from the historical data; and X_k is the k[th] year of the data observation. With given X_k, the best parameter estimate $\hat{\beta} = \left[\hat{C}\ \hat{A}\ \hat{B}\right]$ are determined from the nonlinear regression method, the estimated optimal volume $\hat{V}$, can be obtained from $V = \frac{C}{(1 + A * \exp(-BX))}$. The adequacy of data fitting can be evaluated using the coefficient of determination, *R*2.

Another important statistical test of the model adequacy is to look at the significance of the model parameters. This could be done using the information matrix approach which requires the log-likelihood function to be established. Based on the normality assumption, the log-likelihood function can be written as

$$L = \sum_k \left[-\frac{1}{2}\log(2\Pi\sigma^2) - \frac{(V_k - \hat{V}_k)^2}{2\sigma^2} \right] \tag{3.9}$$

The asymptotic covariance matrix of the parameter estimates can be obtained by inversing the negative Fisher information matrix (67), i.e.

$$Cov(\beta) = \left(-\frac{\partial L^2}{\partial\beta\partial\beta'} \right)^{-1} \tag{3.10}$$

Where $\beta = (C\ A\ B)'$ and the Fisher information matrix

$$\frac{\partial L^2}{\partial\beta\partial\beta'} = \sum_k \frac{1}{\sigma^2} \left[(V_k - \hat{V}_k) \frac{\partial^2 \hat{V}_k}{\partial\beta\partial\beta'} - \frac{\partial \hat{V}_k}{\partial\beta} \frac{\partial \hat{V}_k}{\partial\beta'} \right] \tag{3.11}$$

Given $V_k = \dfrac{C}{1 + A * \exp(-BX_k)}$

Then;

$$\frac{\partial \hat{V_k}}{\partial C} = \frac{1}{1 + A * \exp(-BX)} \tag{3.12}$$

$$\frac{\partial \hat{V_k}}{\partial A} = \frac{-C * \exp(-BX)}{(1 + A * \exp(-BX))^2} \tag{3.13}$$

$$\frac{\partial \hat{V_k}}{\partial B} = \frac{A * X * C * \exp(-BX)}{(1 + A * \exp(-BX))^2} \tag{3.14}$$

Let H be donated as,

$$H = \begin{bmatrix} \dfrac{\partial^2 \hat{V_k}}{\partial A^2} & \dfrac{\partial^2 \hat{V_k}}{\partial A \delta B} & \dfrac{\partial^2 \hat{V_k}}{\partial A \partial C} \\ \dfrac{\partial^2 \hat{V_k}}{\partial B \partial A} & \dfrac{\partial^2 \hat{V_k}}{\partial B^2} & \dfrac{\partial^2 \hat{V_k}}{\partial B \partial C} \\ \dfrac{\partial^2 \hat{V_k}}{\partial C \partial A} & \dfrac{\partial^2 \hat{V_k}}{\partial C \partial B} & \dfrac{\partial^2 \hat{V_k}}{\partial C^2} \end{bmatrix} \tag{3.15}$$

Then

$$H = \begin{bmatrix} \dfrac{2 * C * \exp^{-2BX}}{(1 + A * \exp^{-BX})^3} & \dfrac{X * C * \exp^{-BX}(1 - A * \exp^{-BX})}{(1 + A * \exp^{-BX})^3} & \dfrac{-\exp^{-BX}}{(1 + A * \exp^{-BX})^2} \\ \dfrac{X * C * \exp^{-BX}(1 - A * \exp^{-BX})}{(1 + A * \exp^{-BX})^3} & \dfrac{X^2 * C * A * \exp^{-BX}(A * \exp^{-BX} - 1)}{(1 + A * \exp^{-BX})^3} & \dfrac{X * A * \exp^{-BX}}{(1 + A * \exp^{-BX})^2} \\ \dfrac{-\exp^{BX}}{(1 + A * \exp^{-BX})^2} & \dfrac{X * A * \exp^{-BX}}{(1 + A * \exp^{-BX})^2} & 0 \end{bmatrix}$$

The following *t* test can be performed on the parameter estimates to determine their statistical significance (67):

$$t_{cal}(\beta) = \frac{|\beta|}{diag(\sqrt{Cov(\beta)})} \tag{3.16}$$

Where diag( ) denotes the diagonal element of square root of the covariance matrix. The calculated *t* values can be compared to the tabulated *t* values at a certain level of significance. For example, $t_{calc,0.025}$=2.045 at the 95% level of significance. Utilizing the coded matlab program, the estimated t-values for A, B and C at three different locations are summarized in Table 3.1.

3.6 Projection Confidence Intervals

The logistic function developed will project traffic based on the estimated parameters in the equation. In this case, the actual prediction might not be achieved. To take into consideration the deviation from the actual number of traffic expected for particular year using logistic function, reliability analysis has been performed. This reliability is presented in terms of confidence intervals, in which the lower and upper boundaries of the projection are presented. In this case, the range of expected traffic variation can be calculated by utilizing the confidence intervals. A confidence interval is an interval estimate of the data, whereby, an interval likely to include the parameter is given. The likelihood of the interval to contain the desired value is determined by the confidence level (68).

3.6.1 Delta method

Delta method is utilized for interval estimation and allows obtaining an approximate standard error of the function parameters that are estimated in a given model. The method

requires that the variances and covariance of the parameters are available or can be calculated. Consider the logistic function;

$$V = \frac{C}{(1 + A * \exp(-BX))}$$

We compute V as a nonlinear function of β=(A,B,C). We have the 3 Maximum likelihood estimation (MLEs) and their three variance-covariance matrix "Cov". To obtain its approximate variance, we can apply the formula for variance of a sum of random variables and obtain the following (69):

$$var(V) = \left(\frac{\partial V}{\partial \beta}\right) * Cov(\beta) * \left(\frac{\partial V}{\partial \beta}\right)' \tag{3.7}$$

Where

$$Cov(\beta) = \left(-\frac{\partial L^2}{\partial \beta \partial \beta'}\right)^{-1}$$

and

$$\frac{\partial V}{\partial \beta} = \left[\frac{\partial V}{\partial A} \quad \frac{\partial V}{\partial B} \quad \frac{\partial V}{\partial C}\right]$$

The elements in $\frac{\partial V}{\partial \beta}$ and $\frac{\partial L^2}{\partial \beta \partial \beta}$ can be found in Equations (3.12) through (3.15).

3.6.2 Confidence Intervals

The confidence intervals are given by general formula as;

$$\mathrm{CI} = \hat{\mathrm{V}}(x) \pm t_{\alpha/2, n-2} * \sqrt{\mathrm{var}(V)} \tag{3.8}$$

Where $\hat{\mathrm{V}}(x)$ is the predicted traffic volume at year x;

$t_{(\alpha/2,n-2)}$ is the t-value at alpha/2 significance level and degree of freedom of n-2;

$\text{var}(V)$ is the variance calculated through the delta method

The confidence intervals calculated based on equation 3.8 at three locations, Indiantown Rd between Center St and Central St, Indiantown Rd West of A1A and Okeechobee Rd East of 441 are shown in Figures 3.14(a), Figures 3.14(b) and Figures 3.14(c) respectively. Any data projected outside the confidence interval should be considered not accurate and can lead to erroneous conclusion if 95% confidence level is the target.

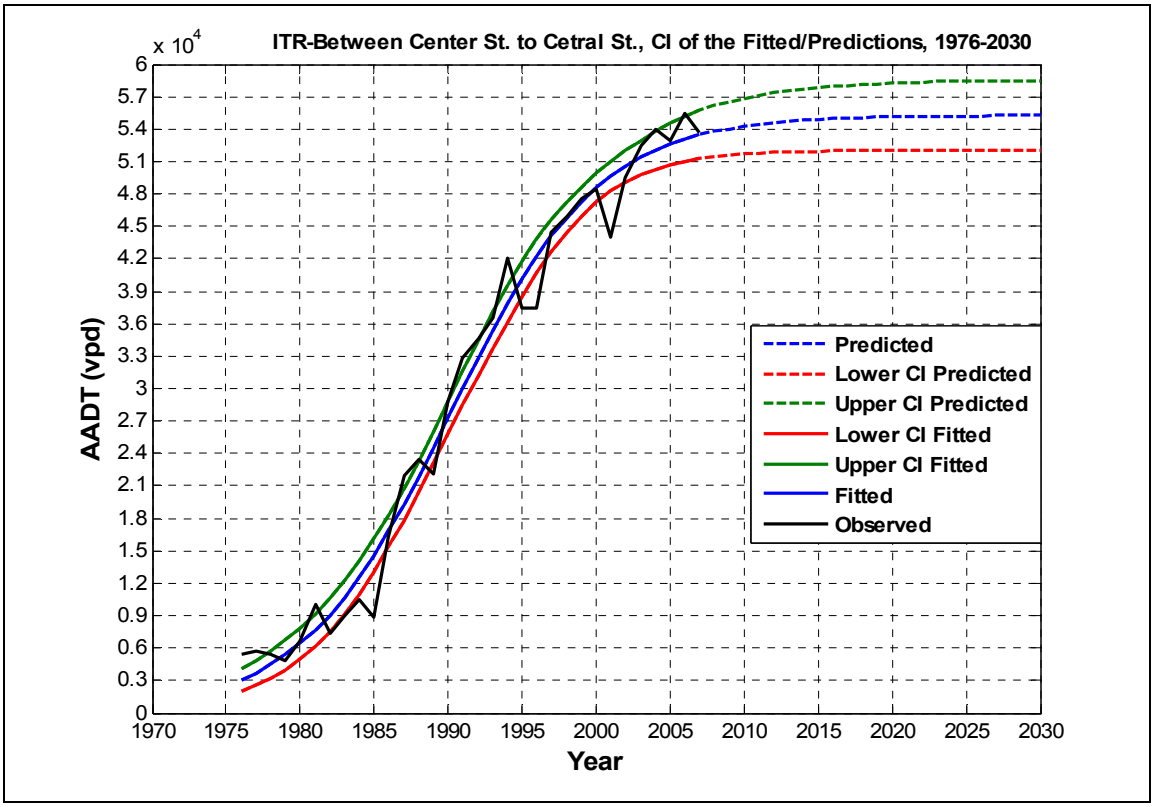


Figure 3.14(a): Confidence Intervals along ITR between Center St. and Central St.

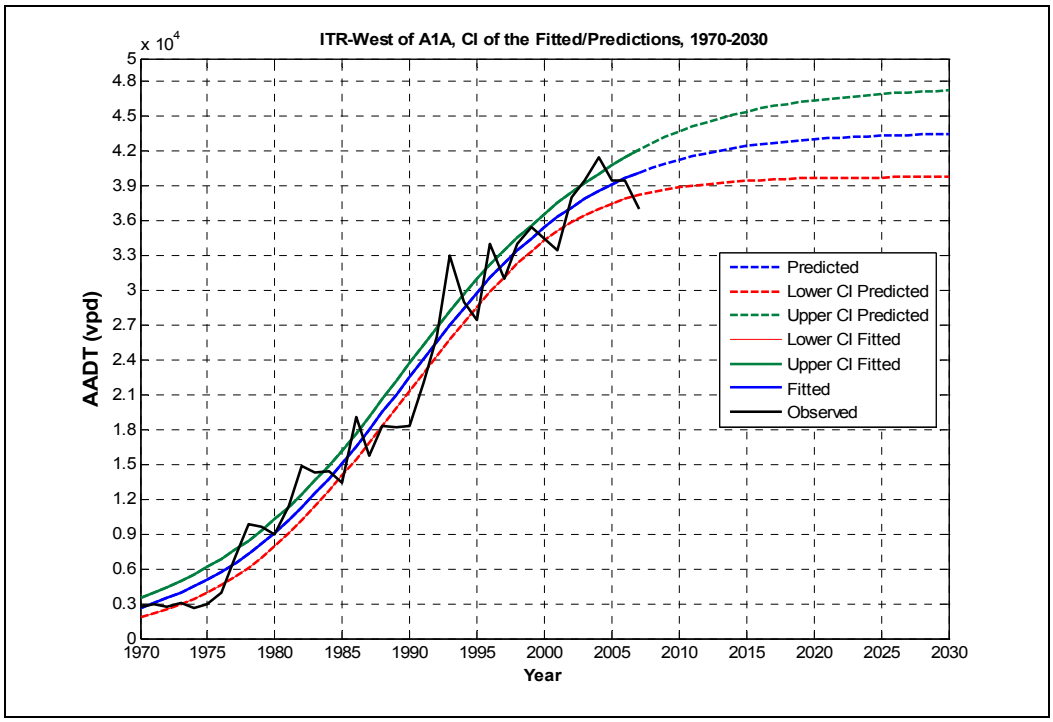


Figure 3.14(b): Confidence Intervals along ITR West of A1A.

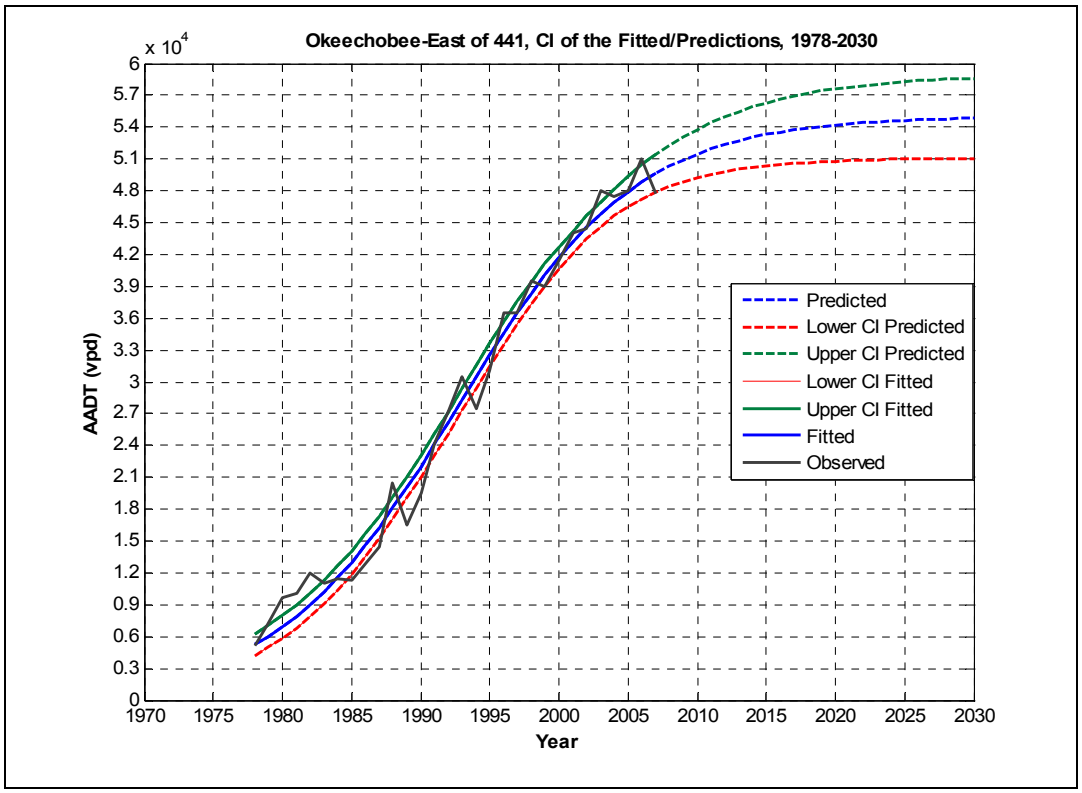


Figure 3.14(c): Confidence Intervals along Okeechobee Rd, East of US 441.

3.7 Validation

The logistic function coefficients shown in Table 3.1 were used for validation, the following coefficient of correlations (R-square) was calculated when linear comparison was made between the existing and the validated projection by logistic function. Table 3.2 summarizes the calculated R-squared after validation at seven locations.

Table 3.2: R-Squared for the Validated Logistic Model

Location	R-Squared
Indiantown Road East of Center St	98.24%
Indiantown Road East of Turnpike	87.79%
Indiantown Road West of Dixie Hwy	97.38%
Okeechobee Road, East of Florida Turnpike	94.89%
Okeechobee Road, West of Florida Turnpike	95.95%
Okeechobee Road, East of US-441	98.57%

The R-squared shown in Table 3.2 verified that logistic function prediction was highly correlated to the existing traffic volumes. The fitted logistic functions showing combined training and validated traffic volumes are shown Figure 3.15 to Figure 3.21.

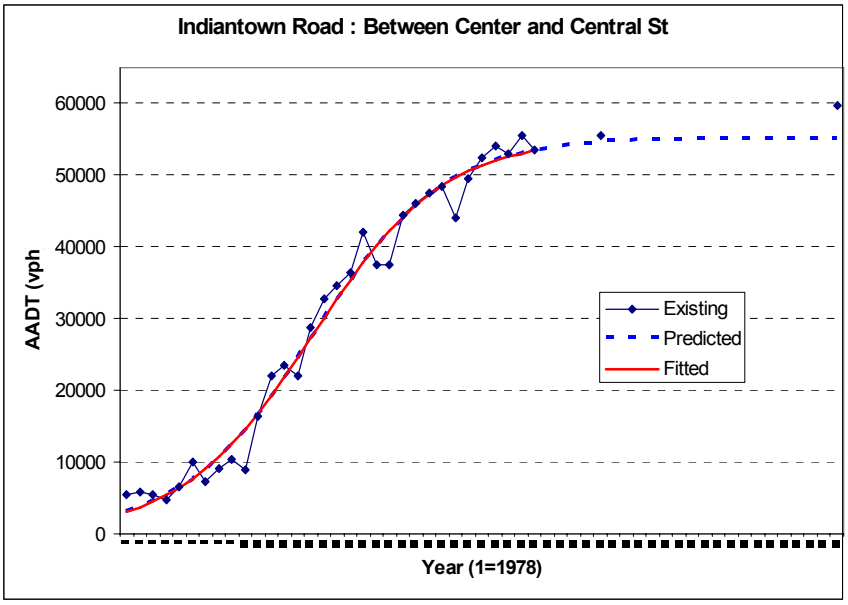


Figure 3.15: Predictions from the Validate Logistic Model at Indiantown E of Center St.

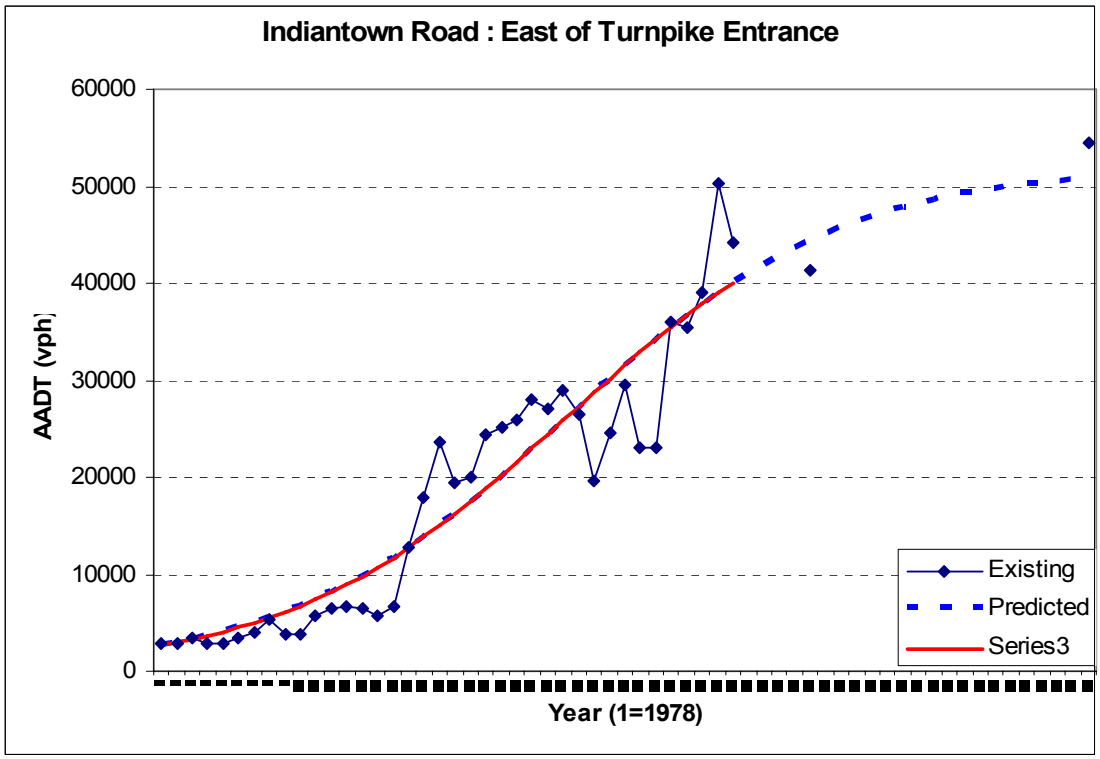


Figure 3.16: Predictions from the Validate Logistic Model at Indiantown E of Turnpike

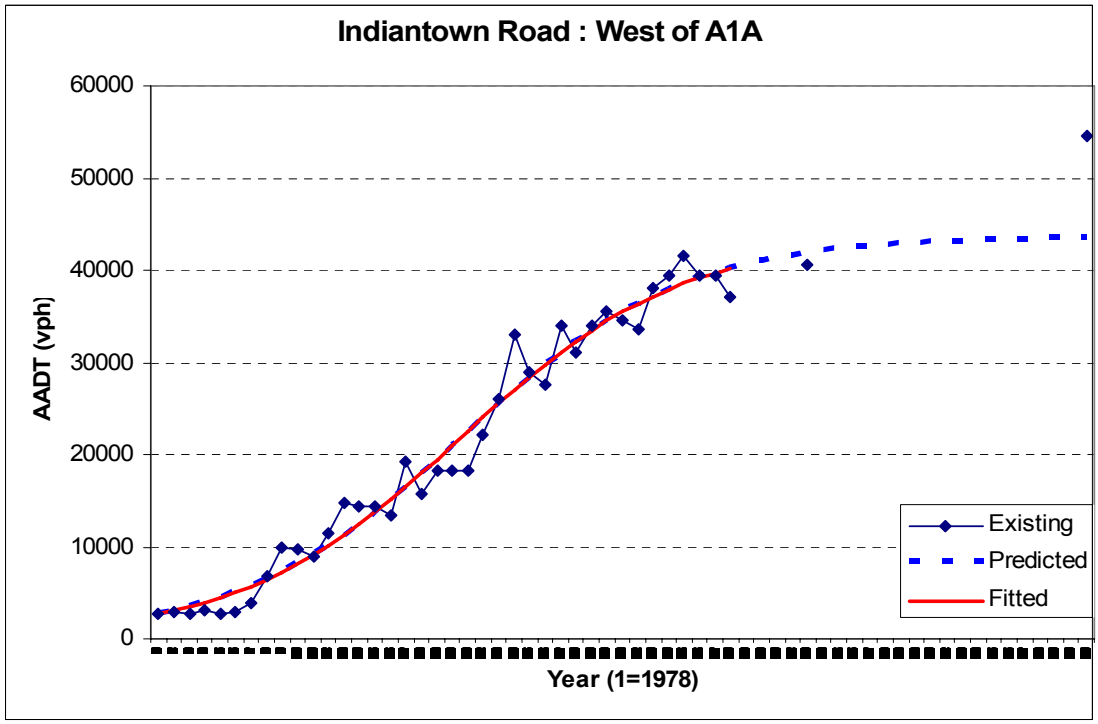


Figure 3.17: Predictions from the Validate Logistic Model at Indiantown W of A1A.

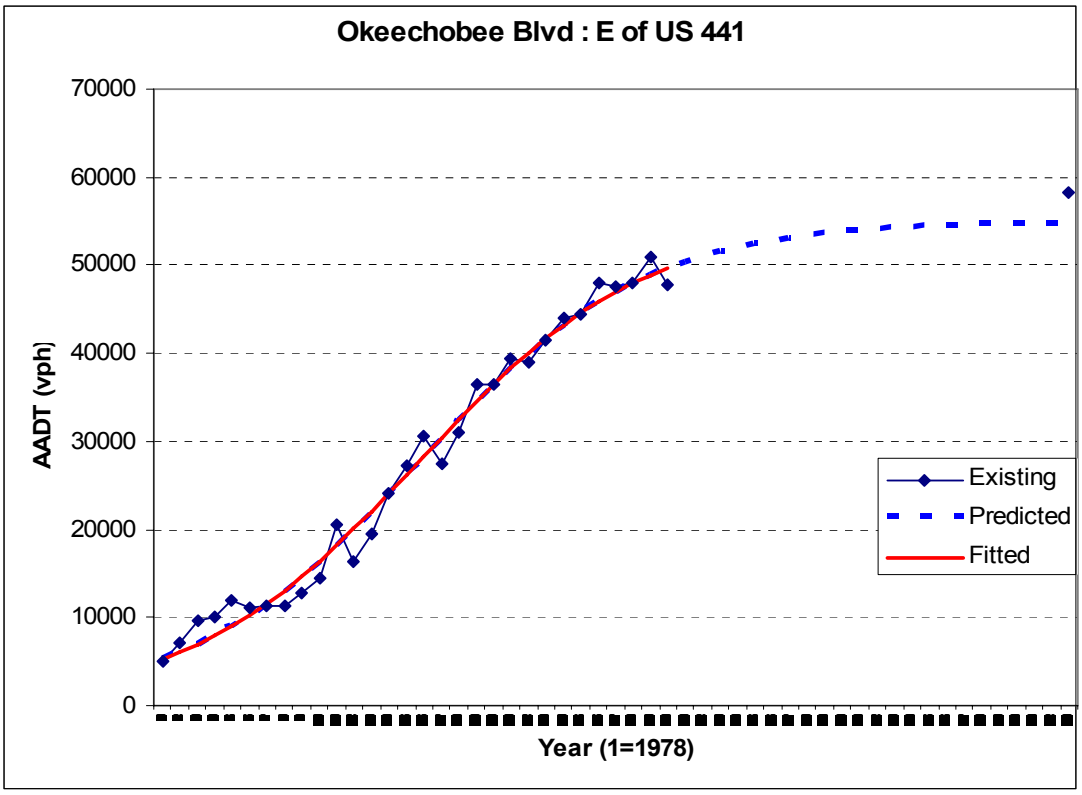


Figure 3.18: Predictions from the Validate Logistic Model at Okeechobee E of US441

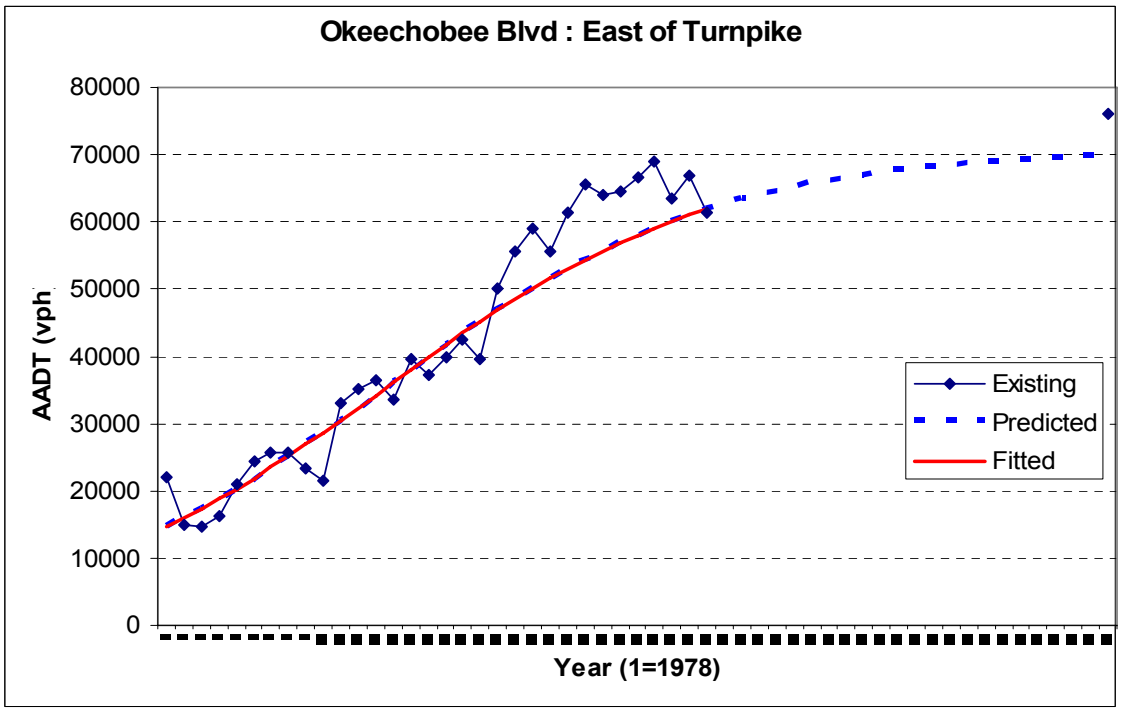


Figure 3.19: Predictions from the Validate Logistic Model at Okeechobee E of Turnpike

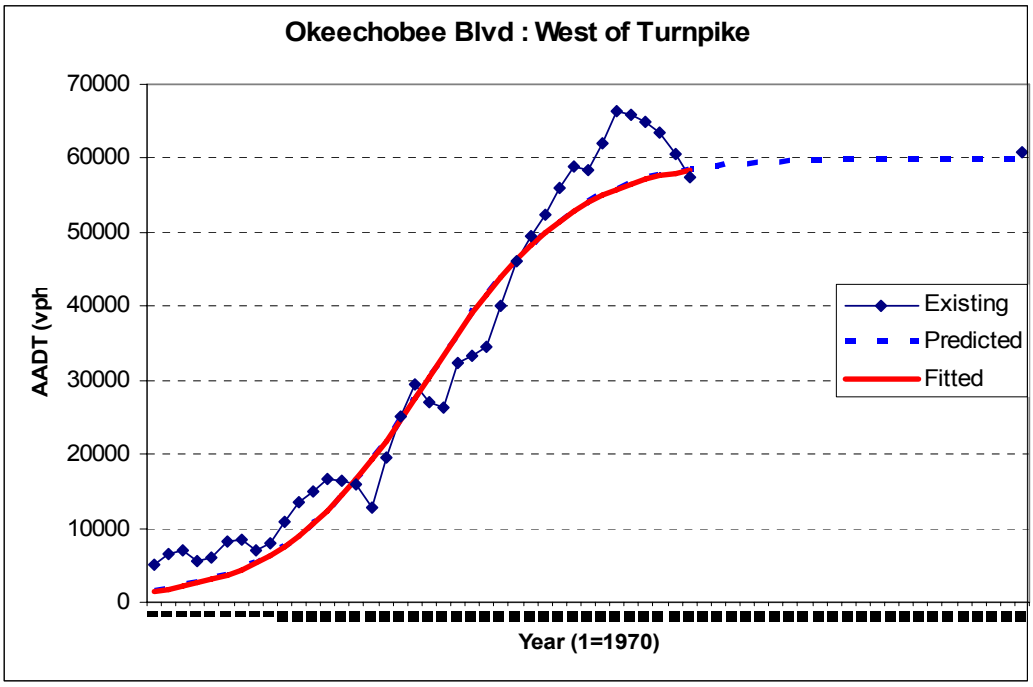


Figure 3.20: Predictions from the Validate Logistic Model at Okeechobee W of Turnpike

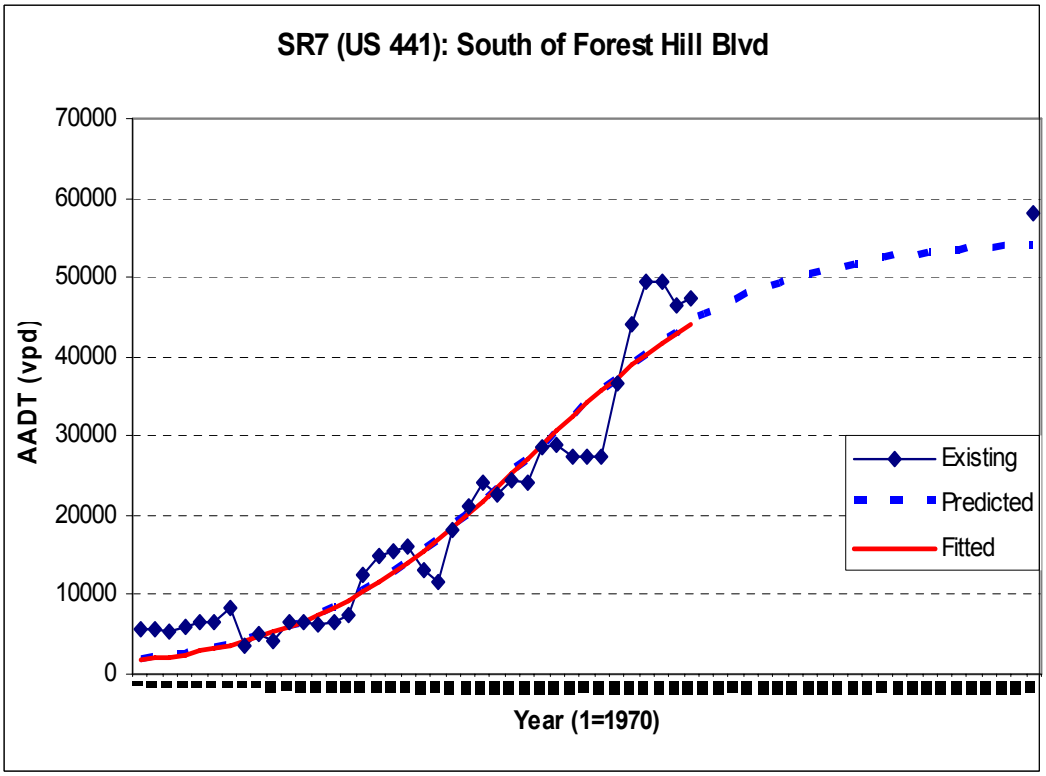


Figure 3.21: Predictions from the Validate Logistic Model at US441 S.of Forest Hill Blvd

3.8 Comparing Prediction with FSUTMS Model Forecast

As shown in Figures 3.15 to 3.21, the validated prediction was prolonged to year 2030. This was done so that to compared logistic model 2030 AADT prediction with those projected by FSUTMS for the same year at those locations. Table 3.3 compares year 2030 AADT from projected by Logistic model and FSUTMS.

Table 3.3: 2030 AADT Predicted by Logistic Model and FSUTMS

Location	FSUTMS	Logistic
Indiantown Road East of Center St	59798	55256
Indiantown Road East of Turnpike	54382	50830
Indiantown Road West of Dixie Hwy	54570	43493
Okeechobee Road, East of Florida Turnpike	76154	69825
Okeechobee Road, West of Florida Turnpike	60726	59982
Okeechobee Road, East of US-441	58226	54830
US-441 (SR 7), South of Forest Hill Blvd	58094	53966
Coefficient of Correlation (R-Squared)	**84.21%**	

Table 3.3 shows very close values when 2030 projection by FSUTMS and Logistic function are compared with linear R-squared of 84.21% and t-value of 5.16. From these projections and comparisons, it can be concluded that, logistic function can predict accurate future traffic volumes in the absence of forecasting models.

3.9 Chapter Summary

Traffic volume forecasting is an important step in any traffic related study involving analysis of future condition. Projected traffic is used to determine important highway design elements, urban planning and economic decisions. The accurately projected traffic will lead to a better and more appropriate designed transportation facility, while

inaccurate projected traffic may lead to under or over-design of transportation facility. This chapter introduced the simplified approach for small scale traffic projection by evaluating different functions with capability of forecasting. Among the five different functions tested, logistic functions was found to be more applicable due to its shape pattern which convey traffic growth patter, and its ability to limit growth to certain level which resemble capacity of the highway. The use of logistic functions as a simplified alternative approach for small scale traffic projection has shown practical usefulness which can utilized and generate accurate forecast. The study utilized historical traffic data ranging from 1970 to 2007 at 12 different locations to develop the three parameter logistic function. The fitted traffic volumes projected using the tree parameter logistic function showed close trend when compared to the observed data. The t-values for the three parameters showed high significance indicating strong correlation. The study also calculated the confidence intervals by using delta method. The confidence intervals express the reliability and range in which the projection can be certain at 95% confidence level. The usefulness of power function can be applied in traffic impact studies for new developments where background traffic are supposed to be projected to the design year and at the locations where demand models are not readily available.

CHAPTER 4

CRASH MODELING

4.1 Overview of Crash Modeling

Much of the research in highway safety has focused on different factors which affect roadway safety. The factors are categorized as traffic characteristics, road geometrics, roadway surface condition, weather and human factors. Previous research has shown that geometric design inconsistencies, operations (traffic mix, volume, and speed), environment, and driver behavior are the common causes of accidents. Environmental conditions and driver behavior can seldom be foreseen. They are specific to case, time, and driver; they are also influenced by geometric inconsistencies. Most of the studies have shown the influence of various geometric design variables on the occurrence of accidents and have concluded that not all variables have the same level of influence in all places. This uncertainty in the influence of geometric variables on accidents has prompted researchers to develop mathematical models to better understand the relationship.

Mathematical models enable highway agencies to select design standards that are essential to highway safety and to allow comparisons among alternative designs that can optimize the overall safety of the highway system under limited resources and other constraints. These models can also be used to test the sensitivity of accident rates to changes in specific geometric variables. From the relation of factors mentioned above,

different researchers have developed the relationship of roadway safety in terms of crash frequency and crash rates, fatality and injury rates and the roadway elements, traffic characteristics, and pavement conditions. Many of these previous studies investigated the relationship of crash rates or frequency in terms of number of lanes, lane width, presence of median, median width, type of median, shoulder width, AADT, access density, number of signalized intersections per road segment, speed limit, vertical grade, horizontal curvature, length of roadway segment, weather condition, time of the day and day of the week. The relationship between safety on the highway and factors mentioned above is the primary focus in crash reduction and predictions. Below are brief discussions about different studies on variables which affect crash rates and frequency for different highway types.

The review of crash models revealed that the crash rate and crash frequency are commonly used as dependent (or response) variables. The crash rate is a measure of exposure as it is related to the vehicle miles of travel. Since the number of crashes is generally low on highway sections, the crash rate is calculated per million or 100 million vehicle miles of travel. The use of the crash rate as the response variable causes the volume of traffic and section length not to be treated as independent variables. If volume and section length have to be considered as independent variables, the crash frequency should then be considered as a response variable and not a crash rate. Furthermore, the literature review revealed that even when the crash frequency is used as a dependent variable, the crashes are disaggregated into injury category (i.e., fatal crashes, injury

crashes, and property damage only [PDO] crashes) and modeled separately. Generally the disaggregating is done by researchers when building models designed to investigate the influence of operating speed and other traffic variables on safety. Another important issue in deciding on the response variable is the time frame of the analysis. To avoid regression-to-the-mean phenomenon, the use of a multi-year crash data is suggested. However, the modeler has to be careful that most independent variables discussed below must have remained the same during those years; otherwise, the modeling should consider different years independently.

4.2 Highway and Traffic Variables Influencing Crash Occurrence

Different variables have been discussed and their effects included in crash models. The effect of lane width is one of the variables which have been discussed in various studies. The link between lane width and safety stands on two principles. The first is that the wider the lanes, the larger the average separation will be between vehicles moving in adjacent lanes. The second strand in the link between safety and lane width is that a wider lane may provide more room for correction in near-accident circumstances. Hence, for a narrow lane, a moment's inattention may lead a vehicle off the edge-drop and onto a shoulder; however, if the lane is wider and the shoulder paved, the same inattention will still leave the vehicle on the paved surface. In these near-accident circumstances, it was difficult to distinguish the effects of lane width, shoulder width, shoulder paving, edge-drops etc. Contradictory conclusions from different studies have been drawn on the effect of lane width. Noland and Oh (12) found that the increase in lane width had no

statistically significant effect on the crash rate, but Abdel-Aty and Radwan (13) found that narrow lane width, narrow shoulder width and reduced median width had positive and significant coefficients in the crash rate. On the other hand, Hadi *et al*.(23) found that increasing the lane width to 12-13ft depending on the highway type is estimated to reduce crash for urban freeways and undivided highways while Karlaftis *et al.*(18) found that lane width, pavement condition, pavement type and friction are the most important variables affecting crash rates on two-lane facilities. In another study done by Harwood *et al* (35), they developed base models and accident modification factors (AMF). One of the factors was on lane width in which a factor of 1.15 was used to project the accident rate on roadway with 11ft lane width compared to 12ft lane roadways. This meant that crash rate on the highway with 11ft was higher by 15% compared to 12ft lane width highways.

The number of lanes is another variable which has been discussed in detail by various researchers. Almost all studies do conclude that the higher the number of lanes, the higher the crash rate. In their research, Noland and Oh (12) found that increasing the number of lanes was associated with increased traffic crashes. In another study, Abdel-Aty and Radwan (13) found that more lanes in urban roadway sections are associated with higher crash rates. Garber (14), considered flow per lane and found that there was an increase in the crash rate as the flow per lane increased. Evidence of the effect of the number of lanes can be seen when a study is done on the conversion of a two-lane, two-way roadway to four or six lanes. With such studies, most have shown an increase in the crash rate. A study by Hadi *et al.* (23) developed negative binomial regression models to

estimate influence of cross-sectional elements on different highway types including freeways, two-lane highways, and multi-lane highways. Of interest in this review were the model result differences between four-lane urban divided roadways and six-lane urban divided roadways. The general comparison of the models indicated that higher AADT levels resulted in higher crash rates for urban divided highways. The models suggested that the safety benefits of increasing median width were more on six-lane urban highways than on four-lane urban highways. In addition, the models showed that the effect of intersection density on crash rates was more pronounced on four-lane divided highways than on six-lane divided highways.

The primary function of the median is to separate the opposing traffic streams. It also provides a recovery area for out-of-control vehicles, a place where vehicles can stop in emergencies, and it allows for the accommodation of left turning lanes and of openings for left or U-turn maneuvers. One study which evaluated median types found that the safety of the median type decreased in the following order: flush unpaved, raised curb, crossover resistance, and TWLTL, Hadi *et al*.(23). Wider medians also seem superior to narrow medians plus a physical barrier, since these can only be effective if vehicles actually collide with them. Another study (36) found that type of median and nature of land use affect crash rate significantly. Harwood (37) evaluated various design alternatives including the following: 2-lane undivided; 2-lane with continuous two way left turn (TWLT) lane; 4-lane undivided; 4-lane with raised median; 4-lane with continuous TWLT lane; 4-lane with continuous alternating left turn lane; 6-lane with

raised median; and 6-lane with continuous TWLT lane. Harwood indicated that one advantage of the 6-lane with raised median design over the 4-lane design is that the additional roadway width provides a more generous turning radius for vehicles to make U-turns at signalized intersections to complete midblock left-turn maneuvers that are prevented by the median. Abdel-Aty and Radwan (13) found that narrow lane width, narrow shoulder width, and reduced median width had positive and significant influence on crash frequency.

There are several purposes in providing shoulders along the highway. These include accommodating stopped vehicles so that they do not encroach on the traveled lane, to make maintenance work easy, to facilitate access by emergency vehicles, and to protect the structural integrity of the pavement. In general, the main purpose of paving shoulders is to protect the road structure from being weakened by water, to protect the shoulder from erosion by stray vehicles and to enhance controllability of stray vehicles. The shoulder also provides a fairly even and obstacle free surface where drivers of stray vehicles can regain control, recover from error, and resume normal travel. The effect of shoulder width and type has been pointed out by different studies as an important aspect in crash frequency. The effect of shoulder width and shoulder paving material goes hand-in hand with lane width, and road side events. Literature generally agrees that the effect of shoulder width on safety is confounded with the effect of lane width and thus these two variables are generally modeled together. Zegeer *et al*.(21) found that the presence of a shoulder is associated with a significant crash reduction for lane widths of 10 ft or

wider while for 10-ft lanes, a shoulder of 5 ft or greater was found to affect accident rate significantly. For 11 and 12-ft lanes, shoulders of 3 ft or greater were associated with significant crash reductions. Another significant result was reported by Ivan *et al.* (38) in which the shoulder width model coefficient was negative for predicting single vehicle crashes but was positive for predicting multi-vehicle crashes. In another study (13), it was found that narrow shoulder width increases the fatality and injury rate compared to wider shoulder width. Harwood *et al*.(35) introduced the accident modification factor, which is based on the shoulder width to predict the crash rate at roadways with different shoulder widths.

Access density refers mainly to the number of driveways within a roadway segment. This term can also be linked with the number of signalization intersections within a specified roadway section. Consideration of intersection spacing is traditionally governed by considerations of delay, signal timing, and signal co-ordination. The safety impact of increased traffic signal spacing is obscured by the traffic volume on intersecting roadways and by vehicle miles of travel. Access density is one of the factors which have been pointed out as the determinant of accident rates on the highways. One study done in New Jersey (22) on the impact of access driveways on accident rates for multilane highways found that approximately 30% of the reported crashes were in mid-block sections and were caused by the presence of access points. Another finding in this study was that approximately 25% of the entering/exiting vehicles from/to access points have impact on mainline traffic. Karlaftis *et al* (18) found that for rural multilane roads,

median width and access control were the most important factors followed by the influence of pavement conditions in the crash. Some empirical evidence suggests that the accident rate increases linearly with access density, but some find that the increase is more than linear. Mouskos *et al*.(22) found that access density and intersection spacing had positive and significant coefficients. In another study (38), it was found that for multi-vehicle crashes, the most important predictor variable was the class of roadway, number of signals and daily single unit truck percentage. Collectively, these studies suggest that frequent access connections, median openings, and closely spaced traffic signals are a recipe for congestion on major roadways with its attendant consequences on safety. Research results deviate from each other on the level of impact of the number of access points on crash rates. The model developed by Gluck *et al.* (28) suggests that an increase from 10 access points to 20 access points per mile would increase crash rates by roughly 30 percent. Papayannoulis *et al.* (29) related traffic safety to access spacing, and presented results from eight states. They found that most literatures show an increase in accidents as a result of the increase in number of driveways. The study suggested that a road with 60 access points per mile would have triple the accident rate compared to 10 access points per mile.

Previous studies have taken account of the speed variable in crash modeling in various forms including posted speed limit, design speed, speed variance, 85th percentile speed, average speed, and actual involvement speed. In analyzing crashes in Virginia, Garber and Gadiraju (15) reported that crash rates increased with increasing speed variance on

all types of roadways and that speeds were higher on roads with higher design speeds, irrespective of the posted speed limits. The authors reported minimal variance when the posted speed limit was less than 10 mph below the design speed of the road. The limitation of the study is that the researchers combined data from different road types—e.g., rural two-lane, urban freeway, and rural freeway—the results of which might not necessarily be replicated when considering six-lane urban roadways only. In another study (14), it was found that the crash rate increases as the mean speed deviates from the posted speed limit. The crash rates were higher when the mean speed was less than the posted speed. The crash rates decreased to a minimum when the means were approximately equal to the posted speed limit; crash rates then continued to increase significantly as the speed increased above the posted speed limit. For a given standard deviation of speed, the crash rate decreased as the flow per lane increased to approximately 1200vph, after which the crash rate began to increase with the flow rate.

The importance of section length in a crash prediction model is generally revealed when the crash rate or crash frequency per mile is calculated. Shorter section lengths can sometimes result in higher crash rates that might affect the validity of crash prediction models. On the other hand, longer section lengths can lead to unrealistic prediction of crashes especially if the uniformity of the sections in geometrics and other variables is not controlled. The literature review has revealed some suggestions of reasonable section lengths for use in modeling. Tarso & Benekohal (39), for example, suggested section lengths of at least 0.5 miles in modeling crash rates in rural interstate highways and two-

lane rural highways. Furthermore, some researchers argue that if standardization of section lengths cannot be achieved, then separate models should be built in groups of similar section lengths. Qin *et al.* (20) found a positive coefficient to section length when they modeled single-vehicle and multi-vehicle accidents. The positive coefficient signifies an increase in the number of crashes as the section length increases. Milton and Mannering (24) found the coefficient of length as a variable to be positive which suggested short sections to be less likely to experience crashes than longer sections because of decreased exposure in terms of vehicle mile of travel (VMT).

Several studies have attempted to determine the variation in crash rates as they relate to hourly traffic volumes and traffic congestion. Traffic congestion occurs when the number of vehicles exceeds the capacity of a highway or road. In some literature, the effect of volume is associated with other aspects of traffic flow like speed, density, and flow. Literature indicates that traffic volume is positively correlated with incidences of traffic crashes. As the number of vehicles on a highway increases, the potential for conflicts within a traffic stream also increases. In addition, previous research has tended to quantify the influence of volume on multi-crashes and on severity of crashes. Qin *et al.* (20) found that for single-vehicle crashes the marginal crash rate is high at low traffic volumes and low at high traffic volumes, probably because crashes are more likely to involve multiple vehicles at high traffic volumes. Zeeger *et al*. (21) found that low-volume road accidents are affected primarily by roadway width, roadside hazard, terrain, and driveways per mile. Martin (40) found that incidence rates involving property

damage only crashes and injury crashes in France are highest when traffic is lightest (under 400 vph) and the incidence rates are at their lowest when traffic flows at a rate of 1,000 to 1,500 vph. Hadi *et al.* (23) found that sections with higher AADT levels are associated with higher crash frequencies for all highway types. Garber (14) found that there is an increase in the crash rate as the flow per lane increased. Mouskos *et al.* (22) found that as AADT increases the crash rate also increases. Milton and Mannering (24) found the positive coefficients of AADT in the model indicating that as the number of vehicles through a section increases, so does the number of accidents. He explained that as the number of vehicles increases through a section, the exposure to potential accidents and number of conflicts increases. The same finding about the effect of AADT on crash rates was also found by Aruldhas (25), Sawalha (26) and Poch and Mannering (27).

Apart from general independent variables, traffic mix has been studied in terms of percentage of certain type of vehicles on the roadway and their effect on the crash rate. In one study by Hiselius (41), he estimated the relationship between accident frequency and the traffic flow by empirically treating the hourly traffic flow in two different ways: consisting of homogenous vehicles and consisting of cars and lorries (trucks). He studied rural roads in Sweden using Poisson and Negative Binominal regression models. He found that important information is lost if no consideration is taken to differentiate between vehicle types when estimating the marginal effect of the traffic flow. The accident rate decreases when the traffic flow is treated as if it were homogeneous. However, when cars are studied separately the result suggests that the accident rate is

constant or increases. The result with respect to lorries (trucks) is reversed, indicating a decreasing number of accidents as the number of lorries increases. Miaou (42) evaluated the performance of Poisson and negative binomial (NB) regression models in establishing the relationship between truck accidents and geometric design of road sections. He used the percentage of trucks as an independent variable in building the model. In all models he developed, the trucks' percentage had a negative coefficient, meaning that as the percentage of trucks increased, there was a reduction in the number of crashes. Milton and Mannering (24) used the percentage of single-unit trucks and the percentage of trucks as the variables in the accident prediction model. They found that an increase in percentage of single-unit trucks tends to decrease accident frequency in Western Washington. Concerning the percentage of trucks, he found that it tends to decrease accident frequency in Eastern Washington.

Location of the roadway has been considered separately in different studies. Various studies considered suburban, urban or rural areas separately and few of them investigated the three situations in the same model. Retting *et al.* (16) studied a simple method for identifying and correcting crash problems on urban arterial streets in Washington DC. They found that urban crashes are often concentrated at specific locations and occur in patterns that can be mitigated through appropriate engineering countermeasures. In another study (17), they considered safety in rural and small urbanized areas. Comparative risk assessment showed village sites to be less hazardous than residential and shopping sites. Karlaftis and Golias (18) investigated effects of road geometry and

traffic volumes on rural roadway accident rates. They developed a methodology which allows for the explicit prediction of accident rates for given highway sections, as soon as a profile of a road is given. Greibe (19) created accident prediction models for urban roads in which he found shopping streets and city center roads having significantly higher accident risk than, for example, residential roads in less densely built-up areas. He concluded, the lower the building density, the lower the accident risk.

Other variables which are found in different literature include sidewalks, grades, horizontal curvature, superelevation, pavement condition, and parking type. Miaou (42) used horizontal degree of curvature, length of horizontal curvature and vertical grade as independent variables in truck accident prediction. He found that both horizontal curvature and percentage grade have positive coefficients. Greibe (19) found that roads linked with parked motor vehicles along the roadside (at the curb) or in marked parking bays have the highest accident risk, particularly for accidents involving pedestrians, motor vehicles from access roads or minor side roads, and for parked vehicles. He also found that the road environment (type and function of buildings along the road) has a considerable influence on the accident risk with shopping streets and city center roads have significantly higher accident risk than residential roads in less densely built-up areas. Milton and Mannering (24) found that large horizontal curves tend to decrease accident frequency.

It is clear that roadways of different functional classes will have different crash experiences with the experiences also being different between rural and urban areas for the same functional class. Similarly, it is evident that crashes occurring at intersections are influenced by independent variables which are mostly different from variables influencing crashes in sections or midblock. Some researchers developed separate models for highways of different functional classes and for intersections and sections. Some studies combine all roadways in a single model. As explained before, some studies use a dummy variable to indicate whether the section was in a rural or urban environment or whether the crash occurred at an intersection or midblock. Poch and Mannering (27) used a negative binomial to model only intersection related accidents in which the independent variables were left-turn and right-turn volumes, phase signals and intersection approach speeds. Greibe (19) modeled only urban accidents. Harwood *et al.* (35) did research on the safety performance of rural two-lane highways in which they developed base models and accident modification factors to account for different roadway geometrics. Persaud *et al*. (43) studied the effect of crash reduction related traffic signal removal in Philadelphia.

Intersections have been modeled separately or with particular attention compared to those which are non-intersection related. Greibe (19) evaluated the influence of signal control on the total number of observed accidents. He found that the signal control variable was not significant in the model, which indicates that the expected total number of accidents is very similar for signalized and non-signalized junctions with the same flow function. With respect to different accident types, Greibe found that rear-end accidents are

significantly higher in signalized junctions than in non-signalized junctions. Turner (44) studied the role of intersection location and non-collision flows on intersection accident estimation. They found that intersection location affects the number of different accident types and that it is important to consider the interactions between turning flows. Retting *et al.* (16) developed a countermeasure for individual intersection-based collisions. They proposed the implementation of safety-related operational and design changes along entire stretches of urban arterials, which include roadway widening, installation of two-way left-turn lanes, driveway elimination, street lightning improvements, installation of raised medians, and improved traffic signal coordination.

4.3 Criteria for Modeling Crash Data

There are two conditions that should be satisfied when developing accident prediction models. The first condition is that the model must yield logical results, which means it must not lead to the prediction of a negative number of crashes and it must ensure a prediction of zero crash frequency for zero values. The second condition is that there must exist a known link function that can linearize this form for the purpose of coefficient estimation. The literature review has revealed that Poisson and negative binomial distributions are often more appropriate for modeling discrete counts of events such as crashes which are likely to be zero or a small integer during a given time period. However, the Poisson distribution is more appropriate for modeling cross-sectional crash data that has equality between mean and variance, a phenomenon called equidispersion. In many crash modeling situations the data generally exhibits extra variation, resulting in

variance being greater than the mean, a phenomenon known as overdispersion. A negative binomial model is well suited for this case.

4.4 Poisson, Negative Binomial and Zero-Inflated Distributions

4.4.1 Poisson and Zero-Inflated Poisson (ZIP)

Miaou and Lump (45) suggested the use of Poisson regression as an initial step in the modeling effort, with the negative binomial model then being applied where appropriate. For the Poisson regression model, the probability of section $_i$ having y_i crashes per year (where y_i is a non-negative integer) is taken in the following form (Cameroon and Trivedi (46), Washington *et al*. (47))

$$P(y_i) = \frac{e^{-\mu_i}\mu_i^{y_i}}{y_i!} \qquad (4.1)$$

$y_i = 0,1,2....$

Where $P(y_i)$ is the probability of having y_i crashes per certain period of time

$\mu = \exp(\beta x_i)$ is the expected (mean) number of crashes

The most common relationship between explanatory variables and the Poisson parameters is the log-linear model,

$\mu = \exp(\beta x_i)$ or

$LN(\mu) = \beta x_i$

x_i = Parameter which is related to the occurrence of crash (Vector of explanatory variable)

β = the coefficient of the corresponding factor (vector of estimable parameter).

The model is estimated by the likelihood function

$$L(\beta) = \prod_i \frac{\exp[-\exp(\beta x_i)][\exp(\beta x_i)]^{yi}}{y_i}$$

The log likelihood function can be expanded as;

$$LL(\beta) = \sum_{i=1}^{n} -\exp(\beta x_i) + y_i \beta x_i - LN(y_i!)$$

Zero-Inflated Poisson is always tested after Poisson distribution is fitted to test which among the two fits the data so closely. ZIP model assumes that the events y_i=(y_1, y_2…..y_N) are independent and the model is $\Pr[y_i = 0] = \varphi_i + (1-\varphi_i)e^{-\mu_i}$

$$\Pr[y_i = r] = (1-\varphi)\frac{e^{-\mu_i}\mu_i^r}{r!}, \text{ r=1, 2… n} \qquad (4.2)$$

Where φ=proportion of zeros.

Maximum likelihood estimates are used to estimate the parameters of the ZIP regression model and confidence intervals are constructed by likelihood ratio tests.

4.4.2 Negative Binomial and Zero-Inflated Negative Binomial (ZINB)

The p.m.f. of the Negative Binomial (NB) model is expressed as:

$$p(y) = \frac{\Gamma(y+\alpha^{-1})}{\Gamma(\alpha^{-1})\Gamma(y+1)}\left(\frac{1}{1+\alpha\mu}\right)^{1/\alpha}\left(\frac{\alpha\mu}{1+\alpha\mu}\right)^{y} \qquad (4.3)$$

where the mean $\mu = E(y) = v\exp(X\beta)$. The corresponding variance is $Var(y) = \mu + \alpha\mu^2$.

Similar extensions to the NB model are considered, including the zero-inflated NB (ZINB) model with constant and mean-dependent split parameters, and the mean-

dependent over-dispersion factor. The ZINB model with constant split parameter, Washington et al., (47) can be expressed as:

$$\text{With } P(y_i=0) = \varphi_i + (1-\varphi_i)\left[\frac{\frac{1}{\alpha}}{(\frac{1}{\alpha})+\mu_i}\right]^{\frac{1}{\alpha}} \tag{4.4}$$

$$\text{With } P(y_i=r) = (1-\varphi_i)\left[\frac{\Gamma((\frac{1}{\alpha})+r)\lambda_i^{\frac{1}{\alpha}}(1-\lambda_i)^r}{\Gamma(\frac{1}{\alpha})r!}\right], \text{ r=1, 2, 3…, n, Where } \lambda_i = \frac{(1/\alpha)}{[(1/\alpha)+\mu_i]}.$$

Maximum likelihood methods are again used to estimate the parameters of ZINB regression model

4.4.3 Testing between Standard and Zero-Inflated Models

Vuong's tests, is the known approach for testing the appropriateness of using the zero inflated model rather than the traditional model, Poisson or Negative Binomial.

The Vuong's test statistic is calculated as follows.

$$m_i = \ln\left\{\frac{f_1(y_i/X_i)}{f_2(y_i/X_i)}\right\}; \tag{4.5}$$

Where "ln" is a natural logarithm

$f_1(yi/Xi)$ is the probability density function for model #1 e.g. ZINB

$f_2(y_i/Xi)$ is the probability density for model #2, e.g. NB

Then, Vuong's value 'V' is given as

$$V = \frac{\sqrt{n}(\overline{m})}{S_m}, \text{ where } \overline{m} = Mean = \left[(1/n) \sum_{i=1}^{n} m_i \right],$$

S_m=Standard Deviation, n=Sample Size

If absolute(V)<$V_{critical}$(1.96 for 95% Confidence Interval), the test does not support the selection of one model over the other.

Large positive values of V greater than $V_{critical,}$ e.g. V> $V_{critical}$ favor model #1 over model #2 whereas large negative values support model #2.

4.5 Study Crash Data

Data for this study originated from Florida Department of Transportation (FDOT) safety crash database. This database includes crash data and its attributes, traffic characteristics and geometric characteristics of roadway by section. Within this database, crashes are differentiated and stored according to the type of roadway, e.g., state-maintained and non-state maintained roadways. For the purpose of modeling, augmented crash data, including the exact milepost and roadway attributes where the crash occurred, were used. These attributes include the location of the crash, nature of the crash, contributing causes, surface condition, lightning and other environmental conditions.

From the database, 2005 crashes on state maintained roads for 2-lane, 4-lane and 6-lane roadway facilities in Palm Beach County were downloaded. Apart from crash data, Roadway Characteristics Inventory (RCI) data for the corresponding roadways was also downloaded. These data provide physical and administrative information relative to

roadway networks that are either maintained by or are of special interests to FDOT. The data in RCI comes from different departments, including safety, maintenance, access management, outdoor advertising, right of way, system planning, public transportation operations and general counsel. The RCI represents Florida's roadway network indexed by data segment. Each segment presents elements that describe each portion of the roadways in physical and jurisdictional terms. Not all features in RCI are of interest for modeling. The features in RCI which were considered for modeling included:

- Length features - characteristics that have "end mile" points that are different from the "begin mile" points.
- Point features - characteristics that occur at a point on roadway. Point features (like intersections) may be composed of several characteristics.
- Physical features - quantified components including type of roadway, shoulder type and median type.

The distribution of crashes is as shown in Figure 4.1

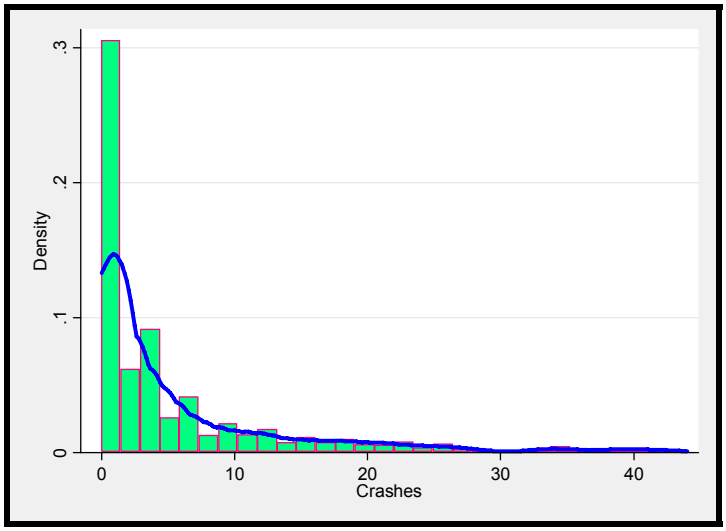


Figure 4.1: Distribution of Crashes

Based on the abovementioned features in RCI, many of the original segments were very short (e.g., 0.0016 miles) because many features were considered in segmentation. For regression purposes, re-segmentation based on roadway characteristics important to the regression model might be necessary to avoid equal covariate values on different roadway segments. Although it has been argued that the parameter estimates would not make a difference even when segments are not aggregated (45), re-segmentation is performed to avoid any possible complication. Based on various studies, 15 features found to be influential to crash occurrence were chosen as the starting variable set for modeling and re-segmentation. As a result, a total of 775 segments were generated, ranging from 0.1 of a mile (528 ft) to 2.00 miles with an average length equal to 0.37 miles as shown in Table 4.1.

For estimation purposes, the indicator variables listed in Table 4.1—such as area type (Urban, Suburban, and Rural) and median type (paved/raised)—were converted to binary variables as follows:

- The area type: 1 = urban and suburban areas, and 0 = rural area.
- The median type: 1 = two way left turn median (TWLT), and 0 = raised median.
- The access class: 1 = locations with unlimited access, and 0 = locations with limited access.
- The speed: 1 = posted speed limit >45 mph, and 0 = posted speed limits <50 mph

In addition, a new variable called vehicle mile of travel (VMT) was created as a product of AADT and length of corresponding segments. The variable was generated in order to

consider length as part of variables. Length of the segments was used as exposure (offset) variable. The average annual daily traffic (AADT) was used as one of the variables in the model. The former characterizes the effect of congestion on crashes, while the latter represents an exposure measure for crashes. Number of lanes for each segment was also used as one of the modeling variables. Other variables under consideration include the following: percentage of trucks, horizontal degree of curve, median width, shoulder width, inside shoulder width and pavement condition.

Table 4.1 Summary of Modeled Variables

Variable	Mean	Min.	Max.
AADT	21,934	1976	67000
MEDWIDTH	18.5	0	
LANEWIDTH	11	9	12
AVG D FACTOR	62%	55%	100%
T FACTOR	6.60%	0%	37.94%
AVG K FACTOR	9.85%	9.1%	12.09%
SHLDWIDTH	4.2	0	16
CRASH FREQ	3.7	0	116
LANES	2	1	4
LENGTH	0.28	0	4.0
SPEED	Indicator variable, 1 (>45 mph), 0 (15-45 mph)		
AREA TYPE	Indicator variable (1-(sub)urban, 0-rural)		
MEDIAN TYPE	Indicator Variable (1-TWLT, 0-undivided)		
ACCESS	Indicator Variable, 1-unlimited access, 0-limited access		

4.6 Selecting Well Fitted Model

Goodness-of-fit was used to determine the best Poisson model(Hardin and Hilbe (48)) . Goodness-of-fit is the measure of the difference between the behavior predicted model to

the observed data. The model with the least difference is the best. Though goodness-of-fit is the best test, but sometimes it has a tendency to favor overfitting models, matching the individual variations in data that may be due to noise or sampling artifacts rather than the actual underlying system. Goodness-of-fit favor models with many parameters, as those models will always provide a better fit. The following terms are used in this analysis to select well fitted model over others.

Deviance Information Criteria (DIC) was one of the approaches used to determine the well fitted model. When fitting a model with certain number of variables, we seek to minimize the deviation between the model and the "perfect fit" model. The overall goal of calculating DIC is to determine the model with fewest variables and yet have small deviance from the "perfect fit" model. Hardin and Hilbe (48) suggested the deviance, *D,* be calculated as $D = 2\phi\{\ln(y;y) - \ln(y;\mu)\}$ where *ln(y:y)* is the natural logarithm of the likelihood of full model and *ln(y:μ)* is the natural logarithm of likelihood of the fitted model. When fitting a particular model, we seek the values of the parameter that minimize the deviance. The optimal model is achieved when the difference in deviance calculations between successive iteration is very small. The values of the parameters which minimize the deviance are the same as the values of the parameter which maximize the likelihood.

Apart from DIC, Akaike Information Criteria (AIC) was also used in best model selection. The Akaike Information Criteria (AIC) offers a way of selecting models with a

lot of explanatory power but without excessive parameters. It does this by penalizing the log likelihood of the models with a term that somehow captures model complexity. Akaike Information Criteria is given by $AIC = \frac{-2L(M_k) + 2p}{n}$ where $L(M_k)$ is the log likelihood for model k, p is number of variables, and n is number of observations. The criterion is such that the lower the AIC, the better the fitted model.

Furthermore, Bayesian Information Criteria (BIC) was another alternative used in model fitness selection. Bayesian Information Criteria is defined as $BIC = D(M_k) - (df)\ln(n)$ where df is degree of freedom $D(M_k)$ is the likelihood of model and n is the number of observations. Again the smaller the BIC, the better the model. If the BIC is positive, then the saturated model is preferred. The difference in the BICs from two models indicates which model is more likely to have generated the observed data. The more negative of BIC the better the fit.

4.7 Selection of Significant Variables

P-value is the primary decision criteria for variables which are significant in the crash model. It is a statistical test associated to null hypothesis. The null hypothesis used in this analysis is that a variable has zero coefficients; that is, it doesn't affect crash frequency much compared to other variables. In general the *p*-value is the probability that the sample could have been drawn from the population(s) being tested (or that a more improbable sample could be drawn) given the assumption that the null hypothesis is true. In this analysis a tolerance of p-value up to 0.15 is accepted since we are modeling

crashes in which being 85% confidence for the crash to happen makes sense. In most analysis a *p*-value of 0.05 is preferred; the significance level depends on the accuracy needed. A *p*-value close to 0.15 signifies that the null hypothesis is false, and the variable has effect to crash frequency. Large *p*-values >0.15 imply that the null hypothesis is failed to be rejected meaning the variable has no effect to the crash frequency. All *p*-values for this analysis are calculated by Chi-Square and Z-statistic. Most of the variables dropped in this analysis were based on not meeting *p*-value level. Apart from testing individual variables, *p*-value is also used to test the whole model together. Selection of one model from another is also done in this analysis basing on general *p*-value.

Significant variables were determined using various modeling techniques. The first technique was by reviewing the correlation between the variable coefficients. The correlation coefficient measures the degree of the linear relationship between two variables. This value can also be viewed as the strength of the linear relationship. It takes values from -1 to 1. A value of 0 means there is completely no linear relationship between the two variables. Positive values of the correlation coefficient indicate that the two variables tend to be both large or both small at the same time. Negative values indicate an inverse relationship. The coefficient of correlation is a very useful measure of weather to include both variables in the same model or not. If the coefficient is approaching 1.0, e.g., 0.5-1.0, we may say there is high co-linearity meaning the variables are highly correlated and inclusion of both variables in the same model will result in unreliable fitness and prediction.

Sequential forward and backward selection was another two techniques used for screening non-significant variables. In forward selection, during first step, all the variables that have not yet been selected are considered for selection, and their *p*-values are recorded. In the end of the step, the variable whose inclusion resulted in the best *p*-value is included in the set. Then, a new step is started, and the remaining variables are considered. This is repeated until a pre-specified number of variables have been included. When using forward selection, first estimates parameters for variables forced into the model. These variables are the intercepts and the first explanatory variables in the model. Next compute the adjusted chi-square statistics for each variable in the model and examines the largest of these statistics. If it is significant at the specified *p*-value, the corresponding variable is added to the model. Once a variable is entered in the model, it is never removed from the model. The process is repeated until none of the remaining variables meet the specified level for entry or until the final model is reached (when there is no change in AIC, BIC, Deviance or Model *p*-Value). The drawback of forward variable selection is that once a variable is in the list of explanatory variables, it cannot be excluded.

In backward variable elimination there is removal of variables, one at a time, from the list of explanatory variables. The model starts with all explanatory variables included and eliminates those which do not seem to improve on the explanation provided by the other variables. At any step, the variable to be removed is determined by the *p*-values. The removed variable is the one with corresponding largest *p*-value. At any time when the

variable is removed, AIC, BIC, Deviance and *p*-value are checked. The procedure stops when eliminating another term cannot reduce those fitness measures any more. The problem with this procedure, is that once a variable is removed, it cannot come back.

Stepwise selection is a modified approach for forward and backward selection, where the variables are just removed and added randomly. The exception is that variables already in the model do not necessarily remain. Variables are entered into and removed from the model in such a way that each forward selection step can be followed by one or more backward elimination steps. The stepwise selection process terminates if no further variable can be added to the model or if the variable just entered into the model is the only variable removed in the subsequent backward elimination. Stepwise variable selection method is the most appropriate one since different variable combinations can be modeled at random selection. This methodology is the one which have been applied in this study. Variables are entered into the model in forward selection, then removed by backward elimination and then some variables added at random while recording vehicle fitness values.

The sign on the coefficient of the variable in the model can sometimes cause it to be omitted in the model. This happens if the effect of the variable is known in advance and the magnitude is the one which is tested. For instance, if the model comes with variable section length having negative coefficient, which mean the longer the roadway section the lower the crashes, which is absolutely wrong, then the variable can be dropped. This

means unexpected coefficients sometimes can result due to unreliable data from that variable or other source of errors, which means it has to be dropped or transformed. For unfamiliar variables (tested for the first time), the sign might not be of much interest for dropping it, but the sign applies much to the variables whose effect are known in advance either from other literature or from reality.

4.8 Selecting Modeling Distribution

As discussed in above sections, Poisson and negative binomial are the best distributions for modeling crash data. Furthermore, the extensions of these distributions, ZIP and ZINB are appropriate for crash data with excessive zero counts. All testing procedures were followed to determine which distribution among the four was appropriate for the available crash data.

4.8.1 Mean and Variance

Poisson distribution assumes mean of the data is equal to its variance. The mean of the crash data is 5.26 while the variance is 96.33. This shows variance is greater than mean which violate the Poisson distribution assumption. Under this test, NB will be preferred compared to Poisson

4.8.2 Overdispersion test

Overdispersion is a phenomenon which occurs when the model is fitted by using Poisson or negative binomial. Hardin and Hilbe (48) listed the following as the source of overdispersion in the data or model.

- When some important independent variables are omitted from the model
- When the data contains a lot of outliers resulting either from unreliable data collection or mistake and errors during data recording
- When the model fails to include sufficient number of interaction terms
- When the variable by itself is not appropriate and it needs transformation
- If the distribution assumed is quite different from the real distribution which relates the data e.g. using linear model instead of quadratic.

In this analysis, an overdispersion test was conducted based on scaled Pearson chi-square and scaled deviance. Scaled Pearson chi-square overdispersion factor σ_d is computed as follows $\sigma_d = \frac{Pearson\chi^2}{n-p}$ where n is the number of observations, p is the number of model parameters, and Pearson Chi square χ^2 is defined as the sum squares of the residuals weighted by the variance function and computed as follows

$$Pearson\chi^2 = \sum_{i=1}^{n} \frac{\left[y_i - \hat{E}(y_i)\right]^2}{Var(y_i)} \qquad (4.6)$$

Where y_i is the observed number of crashes on section "i", E(Yi) is the predicted crash frequency for section "i", and Var(y_i) is the variance of crash frequency for sections i.

Large values (in absolute) of the residuals indicate a failure on the part of the model to fit a particular observation. If σ_d turns to be significantly greater than 1.0, then the data have greater dispersion than is explained by Poisson distribution, and Negative Binomial regression model is fitted to the data, Sawalha (26). Figure 4.2 shows the graphs comparing between the proportions of the observed, Poisson probability and NB probability.

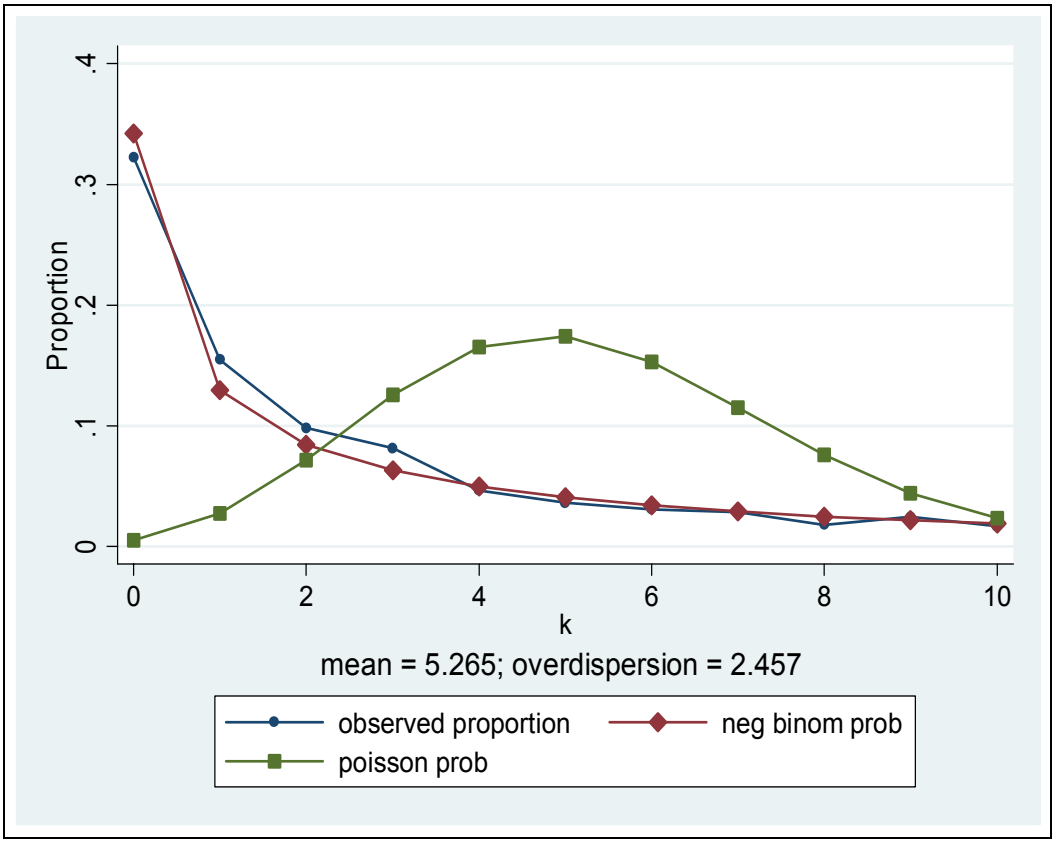


Figure 4.2: Comparison of Observed, Poisson, NB and Overdispersion

As shown in Figure 4.2, the overdispersion parameter is 2.457. According to Hardin and Hilbe (48), if the overdispersion is significantly greater than one, then the data are overdispersed. The values of 2.457 shows the crash data is overdispersed and hence can

be modeled by negative binomial (NB) and not Poisson regression. Furthermore, as observed in Figure 4.2, negative binomial fits well with the observed data compared to Poisson distribution. Observed data and negative binomial are observed to be skewed to the left while Poisson looks like normally distributed. Both overdispersion and shape of the plot favor NB over Poisson.

4.8.3 Test of alpha

Alpha test checks whether there is a statistical significant amount of overdispersion in the data by analyzing the alpha coefficient. Alpha is known as shape factor which indicates numerically the actual amount of overdispersion in the data set. The test follows the assumption that Poisson distribution assume mean equal to variance, given that in the NB the variance is given by $Var(y_i)=\mu_i+\alpha\mu_i^2$. The appropriateness of negative binomial relative to the Poisson model is determined by the statistical significance of the estimated coefficient α. If α is not significantly different from zero, the negative binomial model simply reduces to Poisson. If α is significantly different from zero, the negative binomial is the correct choice and Poisson becomes inappropriate, Poch and Mannering (27). After running NB regression the value of α (alpha) was found to be 1.97, likelihood-ratio test of alpha=0 gave χ^2= 62000 with a *p*-value =0.000. This nullifies the hypothesis that alpha is zero and conclude that alpha is not equal to zero. Then there is statistically significant amounts of overdispersion in crashes favoring NB model over Poisson regression.

4.8.4 Vuong's Test

This test is used whenever there is large number of zeros in the count data, in which Poisson and NB distribution are thought not satisfactory. Zero-inflated count models respond to the failure of the Poisson and the NB to account for the excess zeros in the data. Vuong statistic has an asymptotic normal distribution; if V > 1.96 (large positive value than 1.96) the ZINB/ZIP model is preferred. If V is large negative value, then normal Poisson or NB is preferred. After running ZINB, Vuong's test statistic was found to be -0.25 corresponding with *p*-value= 0.600. This Vuong's value is less than 1.96, indicating that NB distribution is favored over ZINB.

4.8.5 Summary of distribution selection

Based on the findings from the above four selecting criteria, it is concluded that the available crash data follows Negative Binomial (NB) distribution over other distributions. Based on this conclusion, crash modeling is done using NB.

4.9 Model Estimation Results

As discussed in section 4.8, negative binomial distribution was selected as the best modeling tools for these crash data. The response variable is number of crashes per lane-mile. The final fitted NB model retained only three independent variables; AADT, average directional split, median width, lane width, shoulder width, number of lanes and median type indicator as the significant variables in the model. The generated equation

for calculating number of crashes per lane-mile with relation to new development is shown below:

$$CRASH/_{mile} = e^{(3.95E5*ADT + 0.036*D - 0.013*MW - 0.15*LW - 0.07*SW + 0.35*LANES + 1.26*TWLT)}$$

Where ADT = link AADT (Traffic Volume),

D = traffic directional split,

MW = Medium width,

LW = Lane width,

SW = Shoulder width,

Lanes = number of lanes and

TWLT = Two way Left turn Median.

Table 4.2 shows the details of the generated negative model. All the variables are highly significant as seen from Z-value and P-value.

Table 4.2: Negative Binomial Model Results

Negative Binomial				Number of obs = 1328 Prob > chi2 = 0.000 Log likelihood = -4108		
CRASH PER MILE	Coef.	Std. Err.	Z-Value	P-Value	95% Conf. Level	
ADT	0.00004	7.4E-06	5.38	0.000	2.51E-05	5.39E-05
D-FACT	0.0363	0.0105	3.47	0.001	0.0158	0.057
MEDWIDTH	-0.0132	0.0036	-3.68	0.000	-0.020	-0.006
LANEWIDTH	-0.146	0.0558	-2.62	0.009	-0.256	-0.037
SLDWIDTH	-0.0714	0.0239	-2.99	0.003	-0.118	-0.025
LANES	0.3465	0.1057	3.28	0.001	0.139	0.554
TWTL	1.256	0.206	6.09	0.000	0.85	1.661
ALPHA	4.63	0.21			4.24	5.07
Likelihood-ratio test of alpha=0: chibar2(01) = 62000 Prob>=chibar2 = 0.000						

4.9.1 Median Type Indicator

The median type indicator was modeled as a categorical variable with two levels—TWLT (coded as 1) and undivided medians (coded as 0). The zero '0' will be entered in the equation if the link impacted by the development is at location where the roadway segment is undivided. The value of one '1' will be entered at segment with TWLT median. The interpretation from the coefficient of indicator is that, along segments with TWLT median will expect more number of crashes compared undivided segments.

4.9.2 ADT and Directional Split

The coefficient of ADT and directional split are positive indicating as traffic volume increases or directional split increases, the probability of crash to happen increases too.

4.9.3 Medium Width, Lane Width and Shoulder Width

The coefficients of these variables are negative indicating that segments with wider lanes, wider median and wider shoulders will have lower crash frequency compared to those with narrow ones. Roadway improvements which will narrow these variable widths will increase the likelihood of the crash to occur.

4.9.4 Number of Lanes

The coefficients of number of lanes is positive indicating that segments with more number of lanes are expected to have higher probability of crashes.

4.10 Chapter summary

This chapter used negative binomial (NB) distribution to model crashes and determines factors which influence crash occurrences. The NB distribution was chosen due to its capability of modeling count dispersed data after passing a series of tests. Based on this crash prediction model that was developed, the following conclusions were drawn:

- Increase in traffic volume and directional split will increase the probability of crashes.
- Segments with TWLT medians have high probability of increasing crash frequency compared to undivided segments.
- Medium, shoulder and lane widths have negative coefficient, showing the wider are these variables, the less the crash frequency compared to narrow widths.

CHAPTER 5

PROPOSED DELAY AND SAFETY MITIGATION FEES

5.1 Current Road Impact Fee Practice

The introduction and literature review discussed the current road impact practices. It has been found that, currently congestion mitigation is the major component in the impact fee calculation. Currently in many jurisdictions, the developers are only charged road impact fee which is related to capacity deterioration as a result of new trips generated. While some local governments have constant impact fee, some jurisdictions have developed the equations for determining what the developer is supposed to pay. In current congestion type of mitigation, the amount of impact fees paid by development is determined by the amount and type of impact typically generated by the particular use of the property. For instance, for residential units, the impact is measured by the type of dwelling unit (e.g. single-family, town houses, apartment etc.) and the amount of trips generated by those units. The transportation costs like right-of-way and construction fee are based on the typical number of vehicle trips and average vehicle trip lengths by the use of the new development. The variables currently used for impact fee calculations include;

- Construction cost per lane mile
- Number of trips generated
- Trip rates
- Trip length
- Percentage of new trips with respect to the existing capacity

- Adjustment factor (in some counties)
- Gasoline and ad valorem tax credit
- Expected life of the improved facility
- Fuel efficiency
- Net present value factor and
- Average lane (highway) capacity.

5.1.1 Construction Cost

Construction cost refers to the total cost to be incurred when constructing the improvement proposed to maintain traffic within desirable level of service. This cost includes engineering design, rights of way acquisitions, construction, MOT, mobilization, scope contingency, and CEI. The construction per lane mile ranging from new constructions to minor improvement in Florida is attached in the Appendix. For instance, the total construction cost for adding 300 ft exclusive left turn and right turn lane in urban arterial is $185,614 and $409,395 per centerline mile respectively. Adding 2 lane to 4-lane with 5 ft sidewalk and curb & gutter cost $13,965,570 per centerline mile. Generally this cost will depend on what type of improvements has been recommended in order to maintain traffic to operate within acceptable level of service.

5.1.2 Gasoline Credit

Most states return a portion of gas tax revenue to local governments. However, these funds tend to be used for street reconstruction and maintenance, unless earmarked for

infrastructure expansion by the state. The benefit generated by gasoline taxes is always credited from the total cost of the impact fee which the developer is supposed to pay. This is because travel from new development generates gasoline tax revenues, a portion of which is typically allocated to expansion of the transportation system. Only revenue sources that are required to be credited are those which are used by the jurisdiction for road improvements. In Florida local taxes for gasoline and gasohol vary from 10.2 cents to 18.2 cents per gallon with an average of 15.3 cents per gallon. In Hillsborough County for instance, the procedures to determine gasoline credit have been laid out. First is determination of the total gasoline tax paid to each jurisdiction like Federal or State which may be available for construction of new capacity. Then the determination of what portions of gasoline taxes paid to each jurisdiction are used to provide new roadway capacity as opposed to maintenance functions, and then further subdivided between funding for construction of roads as existing deficiencies and construction of roads for growth related to capacity. This is followed by determination of the gasoline tax credit based on estimated annual consumption for new development generated traffic.

5.1.3 Lane Capacity per Lane Mile

This variable determines the highway capacity based on number of vehicles per lane. For instance if the highway is 4-lanes with directional LOS D capacity of 1510 vph, the lane capacity will be 755 vphpl (vehicle per hour per lane). This capacity is divided by the total length of the highway segments impacted by the new developments. It is an additional component of the impact fee equation based on the capacity added per lane

mile of roadway constructed. The most current adopted capacity levels within the jurisdiction where the impact fee model is generated are used. Table 4 of FDOT Quality/Level of Service Handbook gives the list of generalized level of service volumes for different roadway classes. These volumes are applied in the impact fee equation depending on the location of the project. If the proposed improvement involves addition of more number of lanes, then the capacity will be based on the combined number of lanes after improvement. The cost per lane mile of construction is always divided by lane capacity per mile. This means the unit cost multiplied to the trips generated is per available capacity (e.g. cost/lane capacity).

5.1.4 Rate of Return

The rate of return included in the impact fee equation is related gasoline revenue discount. This is the discount rate at which gasoline tax revenues are bonded. It is used to compute the present value of the gasoline taxes generated by new development. The discount rate percent is determined based on the jurisdictions policy and standards.

5.1.5 Trip Length

Trip length is the average length of a trip on the major roadway system impacted by the development. The length is determined by summation of all major segments to be used by the project trips within the radius of influence. Some Counties has developed trip lengths based on different land uses, in this case one needs to use these values in

calculating impact fee instead of new calculations. For instance, Palm Beach County has developed the following trip lengths with corresponding land uses.

General Category	Trip Length (miles)
Residential	6.0
Non Residential	2.0
Non Residential, Short trips	1.0

Source: Palm Beach County, Article 13 page 41

5.1.6 Percentage of New Trips

Percentage of new trips is used to consider only project committed traffic and not pass-by trips. Pass-by trips are trips made as intermediate stops on the way from an origin to a primary destination. Pass-by trips are attracted from traffic passing on an adjacent street that contains direct access to the generator. These trips do not require a diversion from another roadway and are not new trips added to the system. They are involved in either a trip chain or destination with neither the origin nor the final destination of the primary trip being in the development. The percentage of trips that can be classified as pass-by for a site will vary by the type of land use, time of day, type and volume of traffic carried on the adjacent street, and the size of development. Credit for pass-by trips is usually only allowed for retail and some commercial land uses such as fast-food restaurants with drive-through windows, service stations, and drive-in banks. Exclusion of these trips from the trip equation is targeted to ensure the developer pay only for the impact caused by the project generated trips. Florida Department of Transportation recommends the number of pass-by trips not to exceed 10 percent of the adjacent street traffic during the peak hour or 25 percent of the project's external trip generating potential.

5.1.7 Trip Rate, Project Trips, ADT, VMT

Trip rate are obtained from the trip generation which is always determined by use of the current edition of Trip Generation by Institute of Traffic Engineers (ITE), previous studies of comparable sites, or standards adopted by the County. These rates will depend on the type of the land use. The total number of units times the trip rates give the total project trips. The total project trips, in some equations, are used as ADT. The product of project trips and trip lengths give vehicle mile of travel (VMT). This means the use of trip rate, project trips, ADT, VMT in the impact fee equation will depend on how they are related. If the interest in the unit cost, then trip rate becomes a relevant parameter to use in the equation. If the interest is the total cost, then total trips of ADT becomes the relevant parameter.

5.1.8 Current Road Congestion Impact Fee Equation

Based on the above descriptions and concept obtained from the literature review (Palm Beach County, Collier County, Lee County, Hillsborough County and Orange County), the first part of the current impact fee component is shown in equation 5.1

$$CMIF = TR * TTL * \%CT * \left(COST \,/\, LC\right) * 0.5 - GRC \qquad (5.1)$$

Where

CMIR = Congestion Mitigation Impact Fee per Unit

TR = Trip Rate

0.5 = considering Traffic only in one direction (Avoid double counting)

TTL = Total Trip Length

%CT = Percentage Committed Trips (primary trips not pass-by or diverted trips)

COST = Total Construction Cost per Lane Mile

LC = Average Lane Desired Level of Service Capacity

$/Gallon = The amount of gas tax revenue per gallon of fuel that is used for capital improvements

FE = Fuel Efficiency

CRF = Capital Recovery Factor, converts present value to the uniform annual worth

$= \frac{i(1+i)^N}{(1+i)^N - 1}$, where i is the interest rate and N is the life cycle of improvement

GRC = Gas Revenue Credit, which can be calculated as:

$$GRC = TR * TTL * \%CT * \left(DAYS/YEAR\right) * \left(\$/GALLON\right) * 0.5 / FE / RF \quad (5.2)$$

5.1.9 Congestion Impact Fee Illustration Example Data

The following information is considered for the impact fee calculation illustration example.

Development Type

300 Single Family Units (Dwelling Units)

1.5 miles radius of influence

Generated Trips = 300 trips/hour (3,000 trips/day) (in Palm Beach County)

Unit Trip Rate = 10.00 vehicles/dwelling unit (in Palm Beach County)

Unit additional Trip Length = 6.0 vehicle miles (in Palm Beach County)

Unit additional VMT = 60 vehicle miles/Unit

%Committed Trips = 100% (No pass-by trips)

Existing Condition

Assume an existing 6-lane section (3-lane each side)

2 ft traffic separator (median)

Background Traffic 51,374 vpd

2 ft Shoulder

Lane Width = 11 ft

Highest directional split = 47%

Proposed Improvement

Cost per Lane Mile = $2,878,462/lane mile

Additional of one lane on one side (7-lanes)

Widening the lane from 11 ft to 12 ft

Construction of median (6 ft)

Construction of shoulder (6 ft)

Roadway Gasoline Revenue

$0.153/Gallon (In Palm Beach County)

Days per Year
365 days/year

Fuel Efficiency
Fuel efficiency = 18 mpg

CRF: CRF = 0.1233 (i=4% for 10 years)

Then for congestion mitigation fee;

Future traffic

Background + Project Trips = $50000*(1+0.01)^{10} + 3000 = 58231$ vpd

=8319 vpd/lane (with proposed lane addition)

Then

$$GRC = 10*6*1.0*(365)*(0.153)*0.5/(0.1233*18) = \$755$$

$$CMIF = 10 * 6 * 1.0 * \left({}^{2,878,462}\!\!\diagup\!\!_{8319} \right) * 0.5 - \$755 = \$9625 \ / Unit$$

Therefore Congestion Mitigation Impact Fee (CMIF) per Single family Unit = **$9625**

5.2 Proposed Safety Mitigation Fee

In developing safety mitigation fee with respect to crash reduction, the following considerations are taken into account:

- Crash frequency prediction
- The difference between before and after development crash frequency
- Cost per crash (the dollar value adopted by respective jurisdictions)
- Number of development Units
- Accident Reduction Factors (ARF)
- Capital Recovery Factor (CRF)

5.2.1 Crash Frequency per mile

The crash frequency per mile is determined by the crash prediction model developed in chapter 4. This model contains significant roadway features found to be impacting crash

occurrence. Roadway features before the implementation of the new development are used to determine crash frequency before, while features after the development are used to calculate crash frequency after. The developed crash frequency prediction model is shown below.

$$CRASH/_{mile} = e^{(3.95E5*ADT + 0.036*D - 0.013*MW - 0.15*LW - 0.07*SW + 0.35*LANES + 1.26*TWLT)}$$

5.2.2 Increase in Crashes due to new Development

The increase in crashes generated by new development with respect to the original condition is an important factor in safety mitigation fee calculation. The new development should only pay for extra crashes generated due to the impact it imposed on the roadway network. In this case, the proposed formulation determines what the developer should pay with respect to crash increase due to new developments. The difference of crash after and before development is multiplied by the total crash cost.

5.2.3 Cost per Crash (CPC)

The cost per crash is the researched average dollar amount found to be tentatively worth for damage caused by crash and cleanup services. Different states have their specified average cost per crash. NHTSA has assessed the overall economic costs of motor vehicle crashes (65, 66). These studies focused on direct economic losses, and provided estimates of the monetary value society places on the human consequences of crashes, including functional impairment due to injury, "pain and suffering," and even loss of life. According to Blincoe (65), in 1994, the average economic cost of a police-reported (PR)

crash was approximately $12,360**,** and the total economic cost of U.S. motor vehicle crashes (PR plus nonpolice-reported (NPR)) was $150.5 billion. On a comprehensive scale incorporating derived valuations for life and pain and suffering (65, 66), in addition to direct economic loss, the estimates were $34,490 per PR crash and $379.5 billion (PR and NPR) for the national total. Take this resource as an example, the difference is $34,490-$12,360=$22,130 in 1999. Factoring in the annual CPI=3%, the crash cost pertaining to human's desires is about $22,130*$(1.03)^8$ = **$28,000**. It should be noted that some crashes are minor which generate negligible damage cost while some crashes are severe and fatal which involve death. In this case, **$28,000** is an average value to compensate minor and severe crashes. No dollar value can be equated to human life; hence this amount is direct related to property damage and sympathy compensation. It should be noted that, this study didn't take into consideration cost per crash as a related to study crash data. It is recommended in the conclusion for future studies to consider crash cost as a density function due to crash distribution.

5.2.4 Accident Reduction Factors (ARF)

Though the new developments may create more room for vehicular conflicts, hence alleviate probability of more crashes, the proposed improvements can also reduce some crashes. In this case, the crash reduction factor is introduced as an approach to give credit to the developers for the crashes expected to be reduced due to the improvements they are implementing. Crash reduction factors are given to roadway improvements which are thought to result in crash reduction. The exact percentage reduction will depend on state

or jurisdiction laid values or engineering judgments. The possible crash countermeasures listed in Table 2.1 can be used in estimating the number of crashes to be reduced based on the improved facility. Gan et al (63) studied the use of crash reduction factors in safety benefit cost analysis. Apart from developing the procedures for crash reduction estimation, they also conducted a survey country wide of some of the reduction factors used by different States. Table 5.1 summarizes some of the reduction factors found in the literature from different States for a combination of various crash types. These reduction factors vary and sometimes engineering judgment is required to determine some of the reduction factors. The values of ARF's shown in Table 5.1 are used in an illustrative example.

Table 5.1 Selected Crash Reduction Factors from Different States

	Improvement			
	Shoulder Widening	Adding Lane	Widening Lane	TWLT
Average	**6%**	**4.5%**	**3.5%**	**2.5%**

In a situation that the proposed improvement results in multiple crash countermeasures, the overall crash reduction factor is obtained by utilizing the formula below (Roy Jorgensen and Associates).

$$\begin{aligned} ARF &= ARF_1 + (1 - ARF_1)\,ARF_2 + (1 - ARF_1)(1 - ARF_2)\,CRF_3 \\ &\quad + (1 - ARF_1)\Lambda\ (1 - ARF_{m-1})\,ARF_m \\ &= 1 - (1 - ARF_1)(1 - ARF_2)\Lambda\ (1 - ARF_m) \end{aligned} \tag{5.3}$$

Where ARF = Overall crash reduction factor for multiple mutually exclusive improvement at the improved location

ARF_1 = Highest crash reduction factor for a specific countermeasure

m = number of countermeasures at the improved location

To utilize the above equation, it is necessary to list all the individual countermeasures in order of importance, with the highest reduction designated ARF_1, second importance designated ARF_2 and so on.

5.2.5 Safety Mitigation Impact Fee Equation

The following equation is generated as part of impact fee taking care of safety:

$$CMF/_{Unit} = \frac{Max\left(CRASH_{after}(1 - ARF) - CRASH_{before}, 0\right) * CRASHCOST * TTL}{\text{UNITS} * \text{CRF}} \quad (5.4)$$

Where

CMF = Safety Mitigation Fee per Unit

$CRASH_{after}$ = Predicted number of crashes per mile after the development buildout

$CRASH_{before}$ = Predicted number of crashes per mile before the development

ARF = Overall crash reduction factor for the safety countermeasure

CRASHCOST = Adopted dollar value per crash (e.g. $28,000)

TTL = Total Trip Length

CRF = Capital Recovery Factor (converting present value to uniform annual worth)

In order to estimate mitigation fee per trip, then the equation 5.4 should be divided by the trip rate (TR), e.g.

$$CMF/_{Trip} = \frac{Max\left(CRASH_{after}(1 - ARF) - CRASH_{before}, 0\right) * CRASHCOST * TTL}{\text{UNITS} * \text{TR} * \text{CRF}}$$

5.2.6 Crash Mitigation Fee Illustrative Example

Utilizing corridor information example used in section 5.1.9, safety mitigation fee can be estimated as follows:

	ADT	MEDWIDTH	LANEWIDTH	SLDWIDTH	TWLT	AVGDFACT	Lane	crash/mile
Before	55231	2	11	2	0	47	6	65.96
After	58231	6	12	6	0	52	7	77.55

$CRASH_{before}$ = 65.96 per mile

$CRASH_{after}$ = 77.55 per mile

Years = 10, i=4%, then CRF=0.1233

CRASHCOST = $28,000/crash

Assuming there is crash reduction due to shoulder widening from 2 to 6ft (8%), lane widening from 11ft to 12 ft (3.5%) and adding number of lane (5%), then;

$$ARF = 0.06 + (1-0.06)0.045 + (1-0.06)(1-0.045)0.035 = 0.13$$

Then;

$$CMF = \frac{(77.55 * 0.87 - 65.96) * 28000}{300 * 0.1233} = \$1141 / length / Unit$$

Taking a 1.5 miles radius of influence,

Then total length = 1.5*2 = 3 miles (two directional)

The total cost per unit is 1141*3 = **$3,423/Unit**

The total cost per trip is = **$3423/10 = $342/trip**

5.3 Proposed Delay Mitigation Fee

Link travel time is expected to change when traffic volume increases or decreases. Increase in traffic volume decrease the link speed hence increase travel time. The same concept can be applied in the case on new developments along the highway corridors. Substantial increase in project traffic trips in combination with the background traffic is expected to increase the travel time. The difference in the expected travel time before the development trips and travel time after the development can be termed as the travel time delay. The magnitude of delay has been represented by different equations depending on the type of movement, roadway class and other related factors. Delay in seconds, minutes or hours is used to express the magnitude of congestion at the mainline or at the intersection. Different ranges of delay values are used to define level of service A to F with A having short delay while F represent long delay. With the known money value of time, one can calculate the cost incurred by the road user as a result of travel time delay. Marginal cost pricing methodology is introduced in delay cost calculation as part of the impact fee to be paid by the developer. A marginal cost-pricing strategy charges the user any difference between the average cost and the marginal costs.

5.3.1 Travel Time Equation

The difference between the expected and actual travel time within the roadway link can be defined as a delay. These are different travel time equations generated by different approaches for different purposes. This research utilizes the Bureau of Public Road

(BPR) travel time function with the parameter values recommended in 2000 HCM. The link average travel time (AT) is expressed as:

$$AT = t_0\left[1 + a\left(\frac{V}{C}\right)^b\right] \tag{5.5}$$

$$t_0 = \frac{L}{S_0} \tag{5.6}$$

Combining equation 1 and 2 gives;

$$AT = \frac{L}{S_0}\left[1 + a\left(\frac{V}{C}\right)^b\right] \tag{5.7}$$

Where t=Link average travel time (hr)

t_0=Free flow link traversal time (hr)

L=link length (mi)

S_0=link Free-Flow Speed (FFS) (mph)

V=link traffic volume in ADT (vph)

C=link ADT capacity (vph)

a and b = The BPR function parameters obtained from Exhibit C30-1 and C30-2 of 2000 HCM.

5.3.2 Principle of Congestion Pricing and Marginal Travel Time

Marginal cost refers to the change in total transport network costs for a single additional trip. To differentiate marginal cost with the total cost of congestion, the later gives the cost of congestion compared to a state of zero congestion. Furthermore, the marginal cost

can also be differentiated with average cost, the later which can be defined as total transport network cost divided by number of trips. Marginal cost of road travel typically increases with each additional unit of demand, as roads become more congested. Short and long run also can distinct the marginal cost with short run referring to marginal costs with the fixed capacity while long run referring to those with expanded capacity. The delay component of the marginal external cost to road users has been referred to as the marginal external cost of congestion in some publications. Estimating marginal cost with respect to travel time will need knowledge of link speed-flow relationships, area speed flow curves, network assignment models and time-traffic volume relationship. Furthermore, cost of travel in terms of value of time, demand function and traffic supply are needed for calculation of marginal cost due to congestion (60, 61).

To elaborate more, the principle concept of congestion pricing using marginal-cost road pricing is based on the requirement that road users to pay certain amount of money (toll), would equal the costs they impose on all other users by adding to congestion levels. In this case, it can be seen that marginal-cost charging helps in financing road investments. The concept of road users being charged by adding congestion on that section of the roadway can be applied to the developers who build their investments along the highway corridors then generate more trips increasing the congestion level.

Congestion imposes various costs on travelers: reduced speeds and increased travel times, a decrease in travel time reliability, greater fuel consumption and vehicle wear, inconvenience from rescheduling trips or using alternative travel modes, and the costs of

relocating residences and jobs (62). The originality of congestion pricing was based on the approach to control traffic congestion along very busy roadways. This approach was considered because in daily life people tend to choose the cheapest way of life, that is, by imposing some kind of cost to be incurred by the road users simply by using certain roads, then some of them will be discouraged and decide to use another road or means of transportation just to avoid the cost. In this way, congestion pricing is considered in improving capacity by reducing some traffic due to the imposed cost.

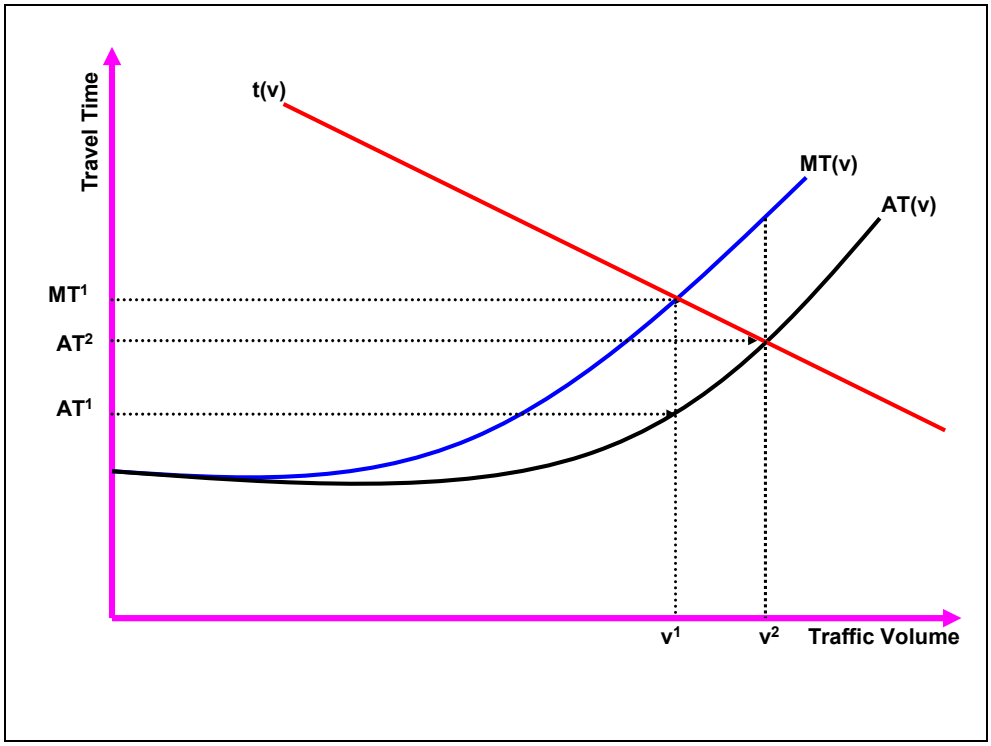


Figure 5.1: Principle of Road Congestion Pricing and Marginal Travel Time

Considering Figure 5.1 let the horizontal axis represent traffic volume along the segment of highway and the vertical axis represent the travel time elapsing for a vehicle to travel from the beginning point to the end of the segment. At low volumes, we expect a vehicle

to travel with higher speed approximately free flow and the travel time curve AT(v) is at a constant free-flow. At higher volumes, traveling speed is expected to fall and the slope for AT(v) turns upwards indicating increase in travel time. If the traffic flow can be interpreted as demand per hour or per day or any specifies time period, then the demand function t(v) can be added as shown in Figure 5.1 for the purpose of having a demand and supply associated with the travel time and volume. The demand function is sloping downward (negative slope) since many people will want to make trips when the travel time is short.

If there is no conditions or restriction on this segment of the highway, then normal equilibrium is expected to occur where t(v) intersects AT(v) resulting into equilibrium traffic volume v^2 and travel time AT^2. If there is no any external factors which affect the travel time apart from congestion, then AT(v) measures average travel time of the trip. The total travel time of trip "v" can be taken as;

$$TT = \sum_i AT_i(v_i) \cdot v_i \tag{5.8}$$

The marginal travel time of the additional trip is taken as

$$\text{MT(v)} = \frac{\partial TT(v_i)}{\partial v_i} = \sum_i AT(v_i) + v_i . \frac{\partial AT(v_i)}{\partial v_i} \tag{5.9}$$

The optimal travel time and traffic volume (MT^1, v^1) is obtained at the intersection of MT(v) and t(v) in Figure 5.1 where the marginal willingness to travel for trips is less than in regular equilibrium. Since the trip travel time is the sum of individual travel times, the requisite delta time $\nabla t = MT^1 - AT^1 = v1 * \frac{\partial AT(v^1)}{\partial v}$, where $v1 * \frac{\partial AT(v^1)}{\partial v}$ is the

marginal congestion travel time imposed by a traveller on others. From question 5.8 and 5.9, it is shown MT>AT

Using equation 5.5 to 5.7;

$$AT = \frac{L_i}{S_0}\left[1 + a\left(\frac{V_i}{C_i}\right)^b\right] \tag{5.10}$$

$$\begin{aligned} MT &= \frac{L_i}{S_0}\left[1 + a\left(\frac{V_i}{C_i}\right)^b\right] + V_i * \frac{L_i}{S_0} * ab\frac{(V_i)^{b-1}}{C_i^b} \\ &= \frac{L_i}{S_0}\left[1 + a(b+1)\left(\frac{V_i}{C_i}\right)^b\right] \end{aligned} \tag{5.11}$$

In summary, the following procedures are followed in calculating delay cost due to travel time per trip using average and marginal travel time concept:

- Plot Average Travel Time (AT) curve (Travel time vs. Volume)
- Plot Marginal Travel Time (MT) curve (Travel time vs. Volume)
- Plot the demand function and note the point where it crosses MT curve, record the travel time at the point where demand function crosses MTT curve, e.g. "t*"
- From the point where demand line meets MT curve, extend the vertical line until it meets the ATT curve. Record the travel time corresponding to this point, e.g. "t_1"
- Then unit change in travel time (day)= t^*-t_1

Let VOT = value of time ($/person-hr)

VHO = Vehicle occupancy

NTD=Number of travel days per year

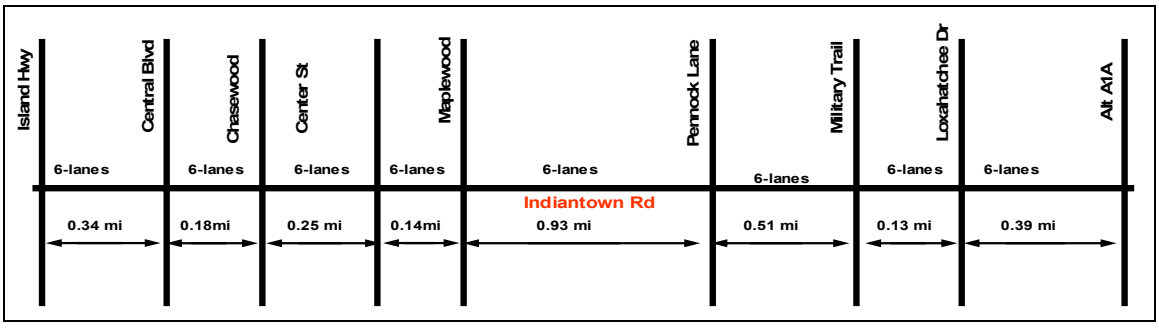
CRF = Capital Recovery Factor

Then, the delay mitigation fee per trip in is given by;

$$DMF = (t^{*} - t_1) * VOT * VHO * NTD / CRF \tag{5.12}$$

5.3.3 Travel Delay Study Corridor

The highway used in this study is a 2.87 miles corridor of Indiantown Rd in Jupiter (Palm Beach County) from Island Way to Alt A1A. This section is an arterial with a posted speed limit of 45 mph. The sketch of Indiantown Rd corridor is shown below.



As shown in the sketch above, there are 9 signalized intersections within this corridor, forming signal density to be 3.14/mile. Using exhibit C30-2 in 2000 HCM, the S_0=50 mph, a=0.99 and b=5.6. Suppose the analysis is performed for each link and by inserting these values in equation (3) above yields;

$$TT = \frac{L_i}{50}\left[1 + 0.99\left(\frac{V_i}{C_i}\right)^{5.6}\right] \tag{5.13}$$

$$MT = \frac{L_i}{50}\left[1 + 0.99(1+5.6)\left(\frac{V_i}{C_i}\right)^{5.6}\right] = \frac{L_i}{50}\left[1 + 6.534\left(\frac{V_i}{C_i}\right)^{5.6}\right] \tag{5.14}$$

The level of service E capacity of the 6-lane class II roadway sections was taken as 51800 vpd. This is the volume adopted by FDOT for urban Class II highway. Using the lengths for each section shown in the sketch above, the value of TT and MT were calculated using the AADT's for each link for existing year, partially buildout and fully buildout scenarios.

5.3.4 Formation of the demand function

The demand function is formed considering congestion and free-flow condition. As the traffic volume increases, travel time also increases but demand decreases. In this case, demand function will have a negative gradient when drawn along travel time vs. traffic volume axis. Lets denote the following key points expected on the demand curve:

V≈0, t≈maximum—low demand (V_{low}, T_{max})

V≈maximum (beyond capacity), t≈0—high demand (V_{max}, T_{low})

Suppose, the link has length L and free flow speed of 50 mph, then T_{low} cab be calculated as;

$$T_{low} = \frac{L}{FFS} \tag{5.15}$$

Maximum time when the demand is at the highest can be equated to 4*T_{low} hence

$$T_{\max} = 4 * T_{low} \tag{5.16}$$

The lowest traffic volume, e.g. V_{low} can be approximated to 0 for worst case scenario. The maximum traffic volume, e.g. V_{max} is assumed 20% of the year traffic is considered. This is based on the consideration of the latent volume beyond the capacity which can be absorbed by the facility before total failure. Therefore; $V_{\max} \approx 1.2 * V$

Let V and T represent traffic volume and travel time along any point on the demand function curve, e.g. (V, T), then;

$$\frac{T - T_{low}}{V - V_{\max}} = \frac{T_{\max} - T_{low}}{0 - V_{\max}}$$

$$T = T_{low} + \frac{(T_{\max} - T_{low}) * (V - V_{\max})}{-V_{\max}} \tag{5.17}$$

Considering $T_{\max} = 4T_{low}$ then

$$T_{(i)} = T_{low} + \frac{3 * T_{low} * (V_{(i)} - V_{\max(i)})}{-V_{\max(i)}} \tag{5.18}$$

5.3.5 Determination of the Equilibrium Points

The equilibrium point in which the demand function intersects the marginal time equation is determined by equating the two functions. This point was estimated by utilizing a scalar nonlinear zero finding "FZERO" in matlab. The MT and AT with respect to the demand function for the whole corridor is summarized Table 5.2.

Table 5.2: AT and MT Equilibrium Points

	2007	**2012**	**2030**
Equilibrium V (average)	37,040	37,863	39,137
MT (t*) (average)	0.1149	0.1215	0.1321
AT (t1) (average)	0.0661	0.0671	0.0687
t*-t1 (hr)	0.0488	0.0544	0.0634

Figures 5.2, 5.3 and 5.4 shows the plotted AT, MT and the demand function for the year 2007, 2012 and 2030. Shown in the figures are also the equilibrium travel time with respect to MT (t*) and AT (t1).

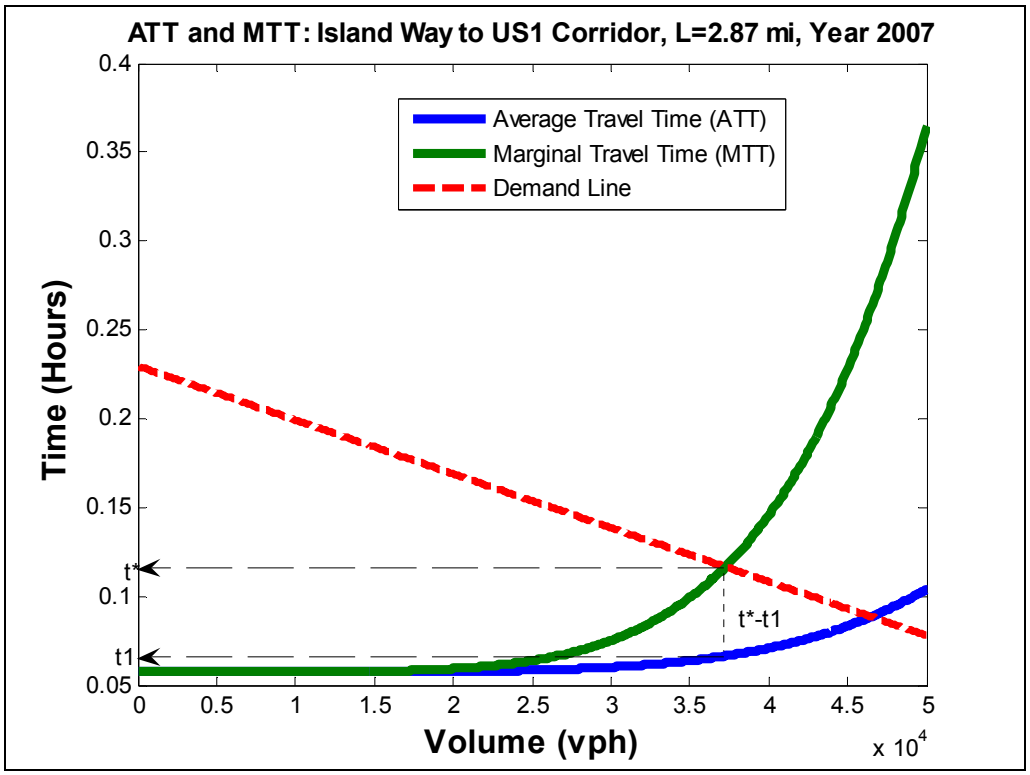


Figure 5.2: 2007 Marginal & Average Travel Time, Demand Function, Equilibrium Points

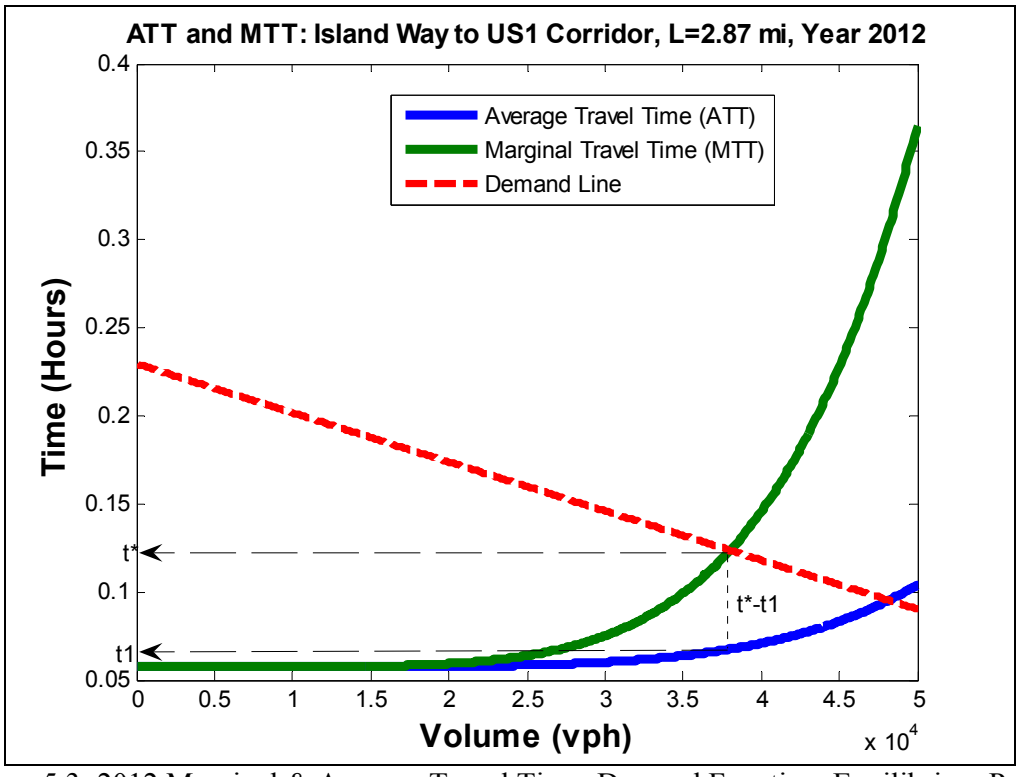


Figure 5.3: 2012 Marginal & Average Travel Time, Demand Function, Equilibrium Points

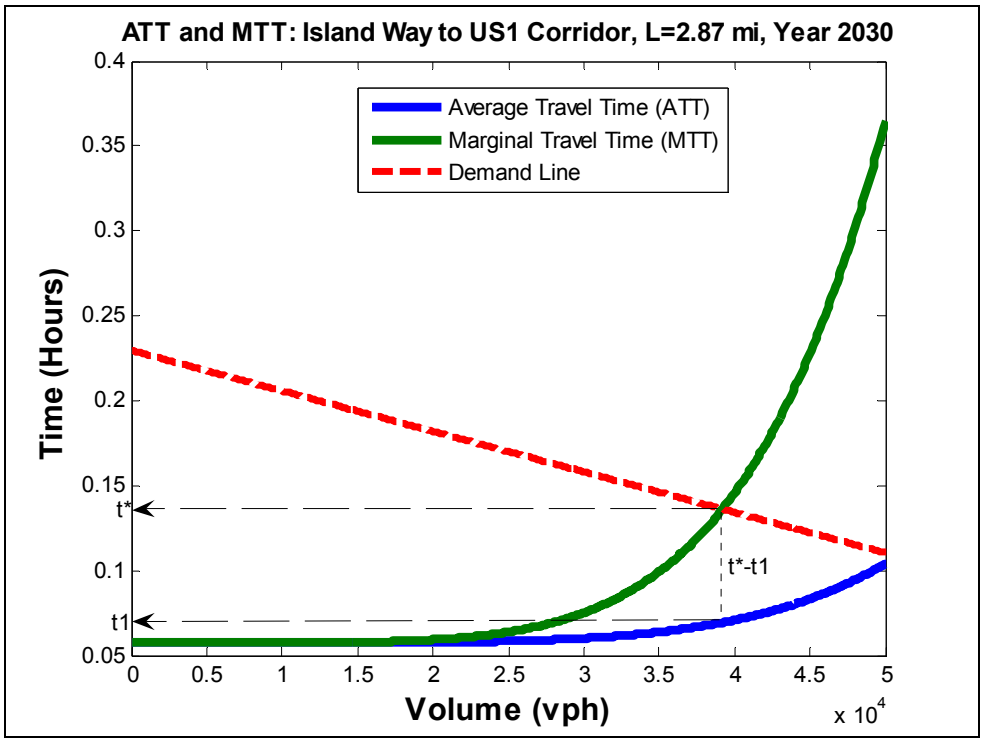


Figure 5.4: 2030 Marginal & Average Travel Time, Demand Function, Equilibrium Points

5.3.6 Delay Cost per Trip

In order to determine the delay cost per trip within this corridor, the following values were considered:

5.3.6.1 Value of Time

Cost associated with travel time are include job time, personal time and in general time for doing time. Value of time has been estimated based on vehicle type, occupancy, and trip purpose. According to Oregon Department of Transportation (64), the value of travel time is conventionally based on either wages or total compensation. Oregon Department of Transportation suggested the value of time for local personal travel to be between 35%

to 60% of the average wage of the region (64). This means for instance, for the State of Florida in which the average hourly wage for the year 2006 was $17.22 according to US Bureau of Labour Statistics (http://www.bls.gov/oes/current/oes_fl.htm) the travel time value can be ranged between $6.03/hr (35%) and $10.33/hr (60%). Some studies have related the value of time with the minimum wages of the corresponding regions. Table 5.3 summarize the minimum wages from each State as far as they were by August 2007. The value of time based 35% to 60% approach proposed by the Oregon Department of Transportation seems to match the minimum wages listed in Table 5.3. Since there is no established constant value of time, this study utilizes the minimum wage for the State where the study is performed as the value of time. The value of time used in study is taken as $7.5 per hour, originating from the minimum wage concept and 35% to 60% of the average wage published by Oregon Department of Transportation.

Table 5.3: Minimum Wages by States

State	Minimum Wage	State	Minimum Wage	State	Minimum Wage	State	Minimum Wage
Federal	5.85	Idaho	$5.85	Missouri	$5.50	Pennsylvania	$6.25
Alabama	None	Illinois	$7.50	Montana	$6.15	Rhode Island	$7.40
Alaska	7.15	Indiana	$5.86	Nebraska	$5.85	S. Carolina	None
Arizona	6.75	Iowa	$6.21	Nevada	$5.33	S. Dakota	$5.85
Arkansas	6.25	Kansas	$2.65	N. Hamp.	$6.85	Tennessee	None
California	7.50	Kentucky	$5.86	N. Jersey	$7.15	Texas	$5.85
Colorado	6.85	Louisiana	None	N.Mexico	$5.15	Utah	$5.15
Connecticut	7.65	Maine	$6.76	N. York	$7.15	Vermont	$7.53
Delaware	6.65	Maryland	$6.16	N.Carolina	$6.15	Virginia	$5.85
DC	7.00	Massachusetts	$7.50	N. Dakota	$5.85	Washington	$7.93
Florida	6.67	Michigan	$7.15	Ohio	$6.85	W. Virginia	$6.55
Georgia	5.15	Minnesota	$6.15	Oklahoma	$5.85	Wisconsin	$6.50
Hawaii	7.25	Mississippi	None	Oregon	$7.80	Wyoming	$5.15

5.3.6.2 Vehicle Occupancy

Vehicle occupancy is used in the delay cost estimation as an approach to calculate cost per person delayed on the corridor. The average number of persons in the private passenger cars is used to illustrate the average vehicle occupancy. The private car is such overwhelmingly dominant choice that travellers are willing to pay substantial capital and operating costs, just to have the flexibility of travel time and destination choices uniquely. The average number of person in private vehicles (vehicle occupancy) ranges between 1.1 to 1.3. This study utilized 1.2 persons per vehicle as the vehicle occupancy. Therefore *VHO* is assumed 1.2 persons/vehicle.

5.3.6.3 Number of Travel Days per Year

Number of travel days per year is taken as weekdays throughout the year which are assumed to be workdays. Working days are considered for value of time assuming delay is more significant during the working trips compared to weekends in which most trips are not work related. In other words, total number of days in a year minus weekends gives total number of travel days. With a year having 52 weeks, then weekend days (Saturday's and Sunday's) will be 2*52 = 104 days. Taking total number of days in a year as 365, then normal travel days = 365-104 = 261 days. Therefore NTD in equation 8 is 261 days per year.

5.3.6.4 Capital Recovery Factor (CRF)

A capital recovery factor is the ratio of constant annuity of the present value of receiving that annuity for a given length of time. Using an interest rate *i*, the capital recovery factor

can be determined as: $CRF = \frac{i(1+i)^N}{(1+i)^N - 1}$, where i is the interest rate and N is the life cycle of improvement. Assuming $i = 4\%$ and $N = 10$ years, then $CRF = 0.1233$.

5.3.7 Delay Mitigation Fee Illustrative Example

Using the above identified values with respect to travel time, the delay cost per trip was calculated for the years 2007, 2012 and 2030 as shown in Table 5.4.

Table 5.4: Delay Cost per Trip

	Scenarios		
	Existing Year	**Partially Buildout**	**Completely Buildout**
VOT ($/hr)	7.5	7.5	7.5
VHO (persons/vehicle)	1.2	1.2	1.2
NTD (days/year)	261	261	261
CRF	0.1233		
t*-t1 (hr)	0.0488	0.0544	0.0634
Delay Cost per Trip =VOT*VHO*NTD* (t*-t1)/CRF	**$930**	**$1,036**	**$1,207**

The result shown in Table 5.4 indicates the cost per trip to be paid by the developer as a result of the travel time delay will depend on the development stages up to complete buildout. For the development impacting for instance along Indiantown Rd, the developer will be expected to pay $930 per trip if consideration is on existing year only, $1036 per trip for the partially buildout condition and $1207 for completely buildout after 10 years. This consideration is based on 4% interest rate (for 10 years), 1.2persons/vehicle, 261 travel days per year and $7.5/hr value of time.

5.4 Combined Safety and Delay Mitigation Fee

The developed safety and delay mitigation fees can be termed collectively as special fees in relation to regular road impact fee. They can be combined together to produce special mitigation fee.

$$SMF = CMF + DMF \quad (5.19)$$

Where

$$CMF = \frac{Max\left(CRASH_{after}(1 - ARF) - CRASH_{before}, 0\right) * CRASHCOST * TTL}{\text{UNITS} * \text{TR} * \text{CRF}}$$

$$DMF = (t^{*} - t_1) * VOT * VHO * NTD / CRF$$

SMF = Special Mitigation Fee per Trip

CMF = Safety Mitigation Fee per Trip

DMF = Delay Mitigation Fee per Unit

TTL = Total Trip Length

TR = Trip Rate

VOT = value of time

VHO = Vehicle occupancy

NTD=Number of travel days per year

CRF = Capital Recovery Factor

5.5 Delay mitigation using Stochastic User Equilibrium Assignment

Section 5.3 discussed increase in delay as the impact of the new developments along the highway corridors. The section also developed mitigation fee to be paid by the developers

as a result to the delay they impacted. The mitigation fee paid by the developer can be used to support the project which can decrease the delay to the desired level. Though there are many kinds of improvements which can be implemented with the delay mitigation fee, re-routing (diverting traffic) can be one of the feasible and practical approach. In this thought, traffic is assigned to another route which runs parallel to the original one whose delay has increased due to the development. The fee charged from the developer can be used to improve the highway assigned to the diverted traffic. Traffic assignment is the distribution of traffic in a network considering a demand between locations and the transport supply of the network. Assignment methods are looking for a way to model the distribution of traffic in a network according to a set of constraints, in this case delay time cost. There are different traffic assignment methodologies used in assigning traffic in the roadway network. These methodologies include: all-or-nothing assignment, user equilibrium (EU), system optimum and stochastic user equilibrium (SUE).

5.5.1 Theory of User Equilibrium (UE) Stochastic User Equilibrium (SUE)

User Equilibrium can be derived from Wardrop's first principle which states that under equilibrium conditions traffic arranges itself in congested networks in such a way that no individual trip maker can reduce his travel cost by switching routes or all used routes between an origin and destination pair have equal and minimum costs while all unused routes have greater or equal costs. UE conditions can be written as a given O-D pair as:

$fi\ (ci - u) = 0$ for all i

$ci - u > 0$ for all i

$\Sigma\ fi = q$

$fi > 0$

Where, fi is the flow on path i, ci is the travel cost on path i, and u is the minimum cost. Solution to the above conditions are obtained by solving an equivalent optimization program

$$\text{Min } z = \sum_i \int_0^{v_i} t_i(v)dv$$

The assumptions regarding UE are:

- The user has perfect knowledge of the path cost
- Travel time on a given link is a function of the flow on that link only
- Travel time functions are positive and increasing

For the Stochastic User Equilibrium (SUE), the following assumptions and implications are considered:

- Implies traveler has perfect knowledge of the network and travel cost
- Assumes that a traveler will choose the perceived least cost path
- Implies different traveler perceives differently, thus introducing stochasticity

The route choice can then be analyzed using logit model which treats all alternatives statistically independent. The formulation of SUE can be summarized as shown below:

$$\text{Min } z(v) = -\sum qE[\min c(v)] + \sum v_i t_i(v_i) - \sum_i \int_0^{v_i} t_i(v)dv$$

Where,

- The first term represent expected minimum cost X demand summed over all links
- The second term represents expected total system travel time
- The third term represents User equilibrium (UE) formulation

SUE model utilizes multinomial models in stochastic assignment. Multinomial models use utilities which are independent and identically distributed mainly with a Gumbel distribution. They also have response homogeneity across individuals and there is error variance-covariance homogeneity across individuals.

5.5.2 Application of SUE on Indiantown Road (ITR) and Toney Penna Dr

Directional hourly volumes along ITR and Toney Penna Dr are shown in Figure 5.5. The travel times were formulated using posted speed limits, link lengths and signal density along each segment utilizing Bureau of Public Roads (BPR).

$$t = \frac{L}{S_0}\left[1 + a\left(\frac{V}{C}\right)^b\right]$$

The traffic flow for each link v_1 and v_2 were based on the percentage of the desired movements over the combined turning movement counts for each intersection. Figure 5.5 summarize the total flow v_1+v_2

Scenarios shown in Figure 5.5 can be summarized as follows;

Scenario 1: Eastbound Movement—AM

ITR $t_1 = 0.047 + 0.05\left[\frac{v_1}{2570}\right]^{5.75}, \quad v_1 = 793$

Toney Penna Dr. $t_1 = 0.071 + 0.048\left[\frac{v_1}{760}\right]^{5}, \quad v_2 = 62$

Therefore $q = v_1 + v_2 = 855$

Scenario 2: Westbound Movement—AM

ITR $t_1 = 0.047 + 0.05\left[\frac{v_1}{2570}\right]^{5.75}, \quad v_1 = 334$

Toney Penna Dr. $t_1 = 0.071 + 0.048\left[\frac{v_1}{760}\right]^{5}, \quad v_2 = 59$

Therefore $q = v_1 + v_2 = 393$

Scenario 3: Eastbound Movement—PM

ITR $t_1 = 0.047 + 0.05\left[\frac{v_1}{2570}\right]^{5.75}, \quad v_1 = 475$

Toney Penna Dr. $t_1 = 0.071 + 0.048\left[\frac{v_1}{760}\right]^{5}, \quad v_2 = 52$

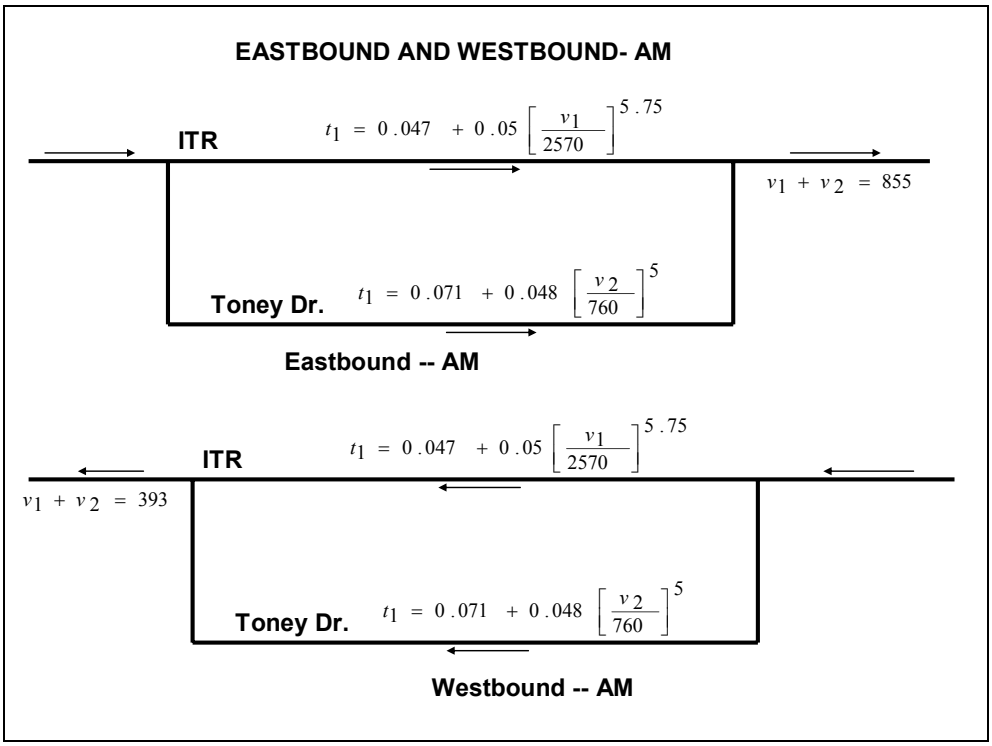
Therefore $q = v_1 + v_2 = 527$

Scenario 4: Westbound Movement—PM

ITR $t_1 = 0.047 + 0.05\left[\frac{v_1}{2570}\right]^{5.75}, \quad v_1 = 643$

Toney Penna Dr. $t_1 = 0.071 + 0.048\left[\frac{v_1}{760}\right]^{5}, \quad v_2 = 46$

Therefore $q = v_1 + v_2 = 689$



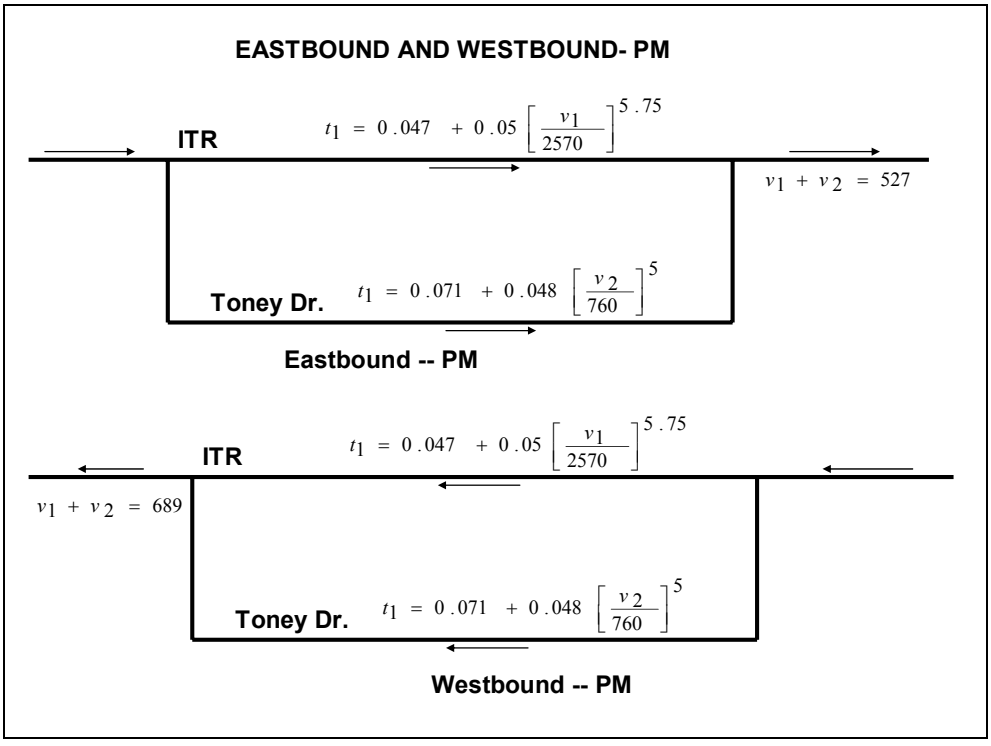


Figure 5.5: Link travel time and flow formulation along ITR and Toney Penna Dr

5.5.3 Illustrative Example of Traffic Assignment using SUE

This illustrative example utilizes variable estimation based on Cost Utility Logit function.

The utility function which is formulated as;

$$GC_1 = \theta_1 * t_1 + \theta_2 * Cost + \theta_3 * \left(\frac{v_1}{C_1}\right)^2 * t_1$$

$$GC_2 = \theta_1 * t_2 + \theta_2 * Cost + \theta_3 * \left(\frac{v_2}{C_2}\right)^2 * t_2$$

Where t_1= travel time along ITR

t_2= travel time along Toney Penna

θ_1= In vehicle time coefficient, -0.094

θ_2= Congestion Index Coefficient, -0.009

θ_3=Total Cost coefficient, -0.002

$\left(\frac{v_i}{Ci}\right)^2 * ti$ = Congestion Index

Gasoline Cost:

Average fuel efficiency = 20 miles/gallon, gasoline price = $3.15/gallon

$1.00 = 107.86 Yen

For ITR Link: length = 2.35 miles

Gasoline cost = 3.15*2.35/20= $0.37 = 40 yen

For Toney Penna Dr Link: length = 3.55 miles

Gasoline cost = 3.15*3.55/20= $0.56 = 60 yen

Note: the coefficient values showed for θ_1, θ_2 and θ_3 where based on study which utilized Japanese "Yen" as the currency. Therefore gasoline cost in US Dollars is converted to Yen for compatibility with coefficients.

Therefore

$$P_1 = \frac{e^{-GC_1}}{e^{-GC_1} + e^{-GC_2}}$$

For optimal solution

$q * P_1 - v_1 \approx 0.0$
$q * P_2 - v_2 \approx 0.0$

Optimizing these equations using Matlab yielded;

	GC1	GC2	t1 (min)	t2(min)	v1(vph)	v2(vph)
Eastbound/AM	-0.6252	-0.9786	2.8200	4.6231	353	502
Westbound/AM	-0.6251	-0.9419	2.8200	4.2669	166	227
Eastbound/PM	-0.6251	-0.9447	2.8200	4.2901	222	305
Westbound/PM	-0.3644	-0.5578	0.0470	0.1882	288	401

Therefore, as a result of Stochastic User Equilibrium (SUE), some of the traffic currently using Indiantown Road will be assigned or diverted to Toney Penna Dr. The delay mitigation fee charged to the developer can be used to improve Toney Penna Dr to accommodate the diverted traffic.

5.6 Chapter Summary

This chapter developed a special mitigation fee which is currently not considered in the impact fee estimations. The current road impact fee estimation in many jurisdictions considers mainly the impact imposed on the congestion and the proposed improvements to alleviate that congestion. Literature reviews have found other two transportation problems which can be worsened by trips generated by new developments. These are

increase in travel time (delay) and creation of more vehicle conflicting scenarios causing safety concerns (increase in crash). These two parameters, increase in delay and probability of increase in crash can be determined and priced, the cost which can be paid by the developers. The chapter therefore highlighted with an illustrative example all parameters and calculations involved in the current practice for impact fee estimation. Using the developed crash prediction model, the chapter introduced the safety impact fee estimation equation which considers number of crashes predicted before the development and those after the implementation of the new development. With the fact that proposed improvements after implementation of the new development may reduce some of the previously expected crashes, the accident reduction factors have been applied for improvements which are applicable, e.g. lane, median, and shoulder widening. Using the average crash cost, the difference between the crash after with reduction factor applied and crash before is multiplied with the crash cost then divided by the number of units to give cost per unit. Illustrative example is associated with the developed safety mitigation to elaborate the procedures to be used in estimation safety impact fee.

Delay mitigation fee estimation is introduced considering the increase in travel time caused by increase in trips from the new developments. Using congestion pricing and marginal cost methodologies, the increase in travel time per unit trip is determined. The value of time is taken as the minimum wage at the location where the development is implemented. The choice of the value of time considered different researches which found the justifiable value may range from 35% to 60% of the average hourly wage.

Taking average hourly wage to be between $15/hr to $18/hr, the value of time was taken as $7.5/hr which is about the minimum wage in many states. The delay calculated from the marginal effect analysis is multiplied by the value of time, vehicle occupancy and number of working travel days in the year to estimate delay cost per trip.

To avoid double charging of the developers, the charged congestion impact fee should be deducted as a credit from the combined delay and safety mitigation fee. The fee from delay mitigation can be used to improve another corridor parallel which traffic diverts due to increased delay cost on the original route. Stochastic user equilibrium has been found to be appropriate methodology in determine traffic assignment between the two routes. Stochastic model take account of variations in drivers' perceptions of travel times or costs. This is done by means of a probability distribution for perceived link costs.

The findings from this chapter are major contribution which adds planning, safety and operation elements in improving safety and traffic operations. It has added important components (safety and delay) which are missing in the current mitigation estimation practices.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Overview

The main objectives of this research were to evaluate a simplified approach for small-scale traffic projection and evaluate how transportation problems, congestion, safety, and delay can be incorporated in the mitigation fee estimate. Figure 6.1 summarizes the complete steps, which incorporate the developed logistic function for traffic projection as well as safety and delay special impact fees.

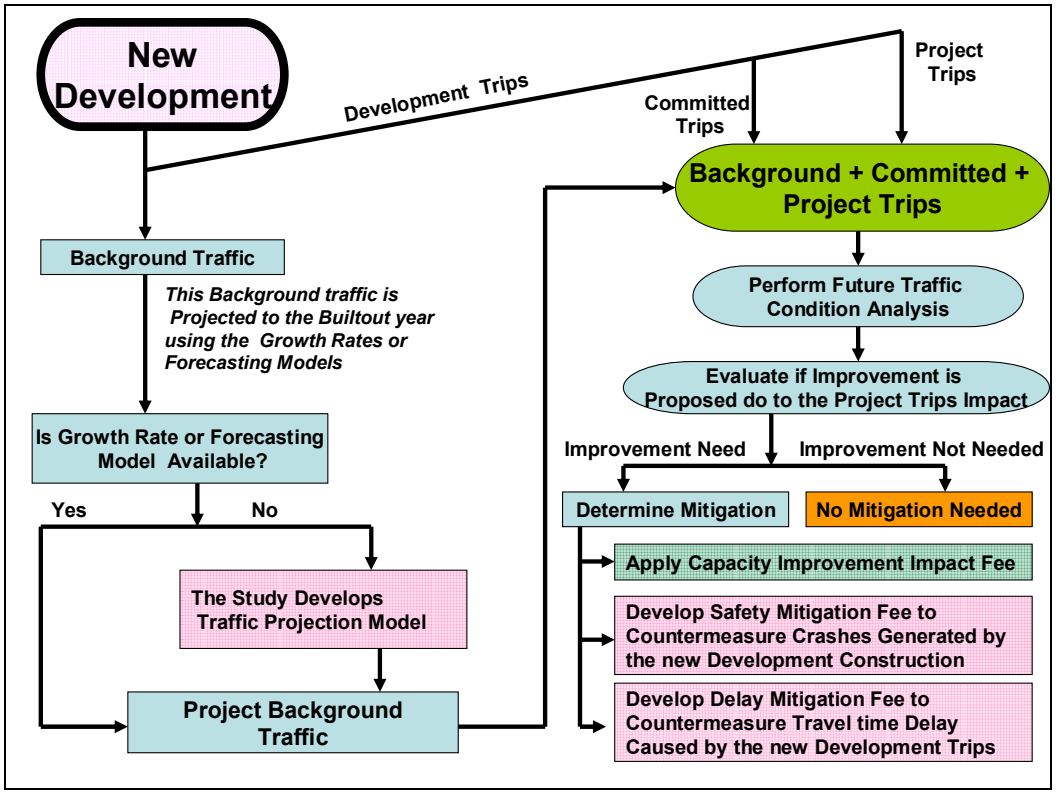


Figure 6.1: Complete flow chart incorporating the objectives of the study

Considering the impact generated by new developments along the highway corridors as the system case study, and utilizing Figure 6.1, this research had the following objectives:

(i) Develop a simplified traffic projection methodology, which can be easily applied at the locations without pre-determined growth rates

(ii) Develop a crash model which can be used to predict probability of future crashes on the highway segment

(iii) Develop Safety Mitigation Fee by utilizing the developed crash model

(iv) Develop Delay Mitigation Fee by utilizing Congestion Pricing approach

The first objective was based on the fact that new developments always generate traffic, which impacts the traffic patterns within the specified radius of the influence. The new development generated trips are added to the background traffic, which is also forecasted to the buildout (design) year. The accuracy of the projected traffic is very important to the recommended improvements. If the background traffic is under projected (erroneously projected below expected value), then the impact of the new development will be seen as inconsequential and this will lead to proposing improvements which do not reflect the real traffic pattern at that year. The developer will pay less for mitigation than expected since the impact is minor. On the other end, if the traffic is over projected (erroneously projected beyond the expected value), then the recommended improvements will cause over design, and the developer will be required to pay more for mitigation than what he or she would have been required to pay if the projection was accurate. Considering these issues, this study developed a logistic function as a simplified projection tool at the locations without available traffic growth rates.

The second objective was aimed in developing a model which can quantify how roadway features influence safety in terms of crash occurrences. New developments generate new traffic which leads to the need of geometric changes in order to accommodate these new trips. With the disturbance of the existing traffic patterns and geometrics, safety along these segments is also disturbed. The improvements made due to new developments can increase or reduce crash risk or can do both at the same time. In order to evaluate how much change in traffic and geometry affect the safety, crash model using generalized count data linear models are applied to model crashes with roadway features as independent variables. Using before and after roadway variables in the crash model help in determining how the safety in terms of crash frequency has been impacted. These before and after crash frequencies are then used in the safety mitigation fee calculation.

The third objective of this study developed a safety mitigation fee. Safety mitigation fee is developed based on the assumption that, new developments create safety problems and the developers should pay for the crash risk they impose. With the known cost of crashes and by using the developed crash model which calculate crash frequency before and after the development, the safety mitigation fee is calculated. The difference between before and after predicted crashes is multiplied by the cost of crashes and then converted to one payment using capital recovery factor. Development of safety mitigation is taken as a major contribution over the current practice which considers congestion only in the impact fee calculation

The fourth objective of this study developed delay mitigation fee by considering the delay per trip imposed on the road user by the new developments. Travel time is directly proportional to traffic volumes. At lower traffic volumes the travel speed is high, causing travel time to be a short time from origin to the destination. As the traffic volume increases, travel speed decreases due to congestion causing travel time to be longer. Theoretically, new developments generate new traffic which increase congestion and lower travel speed, hence longer travel time. In other words, the new generated traffic increases travel time. The difference between the original travel time before the development and travel time after the increase in traffic can be termed as travel time delay. Using congestion pricing methodology, travel time delay per trip can be calculated. Furthermore, with known value of time, vehicle occupancy and number of travel time days, the delay cost per trip can calculated. This study therefore utilizes marginal cost analysis utilized in congestion pricing to calculate travel delay cost per unit trip. Development of delay mitigation is a major contribution over the current road mitigation determination which considers congestion only in the impact fee calculation and neglect impacted delay. It should be noted that in order to avoid double payment by the developer, the calculated congestion impact fee is credited in the delay mitigation fee. In other words, congestion impact fee is deducted from the delay mitigation fee as a credit. The last objective combined the developed safety mitigation fee with congestion impact fee to form an integrated congestion-safety impact fee equation.

6.2 Conclusions

Based on the objectives and findings of this research, conclusions were made as follows.

6.2.1 Simplified Traffic Projection

Logistic function in the form of $V = \frac{C}{(1 + A * \exp(-BX))}$ has been developed and found to be well fitted for traffic projection. The variables in this function are X, representing number of years from the base year, and V, which is the projected traffic. The parameters A, B and C are the constants to be determined and which regulate the final fitted outcome. The final values of the constants A, B and C can be found by optimization or any modeling software, these values are site specific based on traffic trend. The nature of this form of logistic function constrains the projection not to go beyond the constant value C, which can be taken as the maximum allowable capacity within that link of the road. The trend and prediction shape of this logistic function reflect the normal traffic growth. As shown in Figure 6.2, the traffic projected by logistic function is S-shaped with gradual growth during the initial years after opening, then rapid growth at the mid years, and then gradual growth at final stages approaching design year.

Figure 6.2 shows the fitted and predicted logistic function at one of the road segments used in this study. As shown in the figure, the fitted logistic function generated traffic volumes very close to the existing ones, and the produced predicted reasonable results.

The use of logistic function is therefore considered adequate for small scale traffic projection at the locations without reliable pre-determined growth rates.

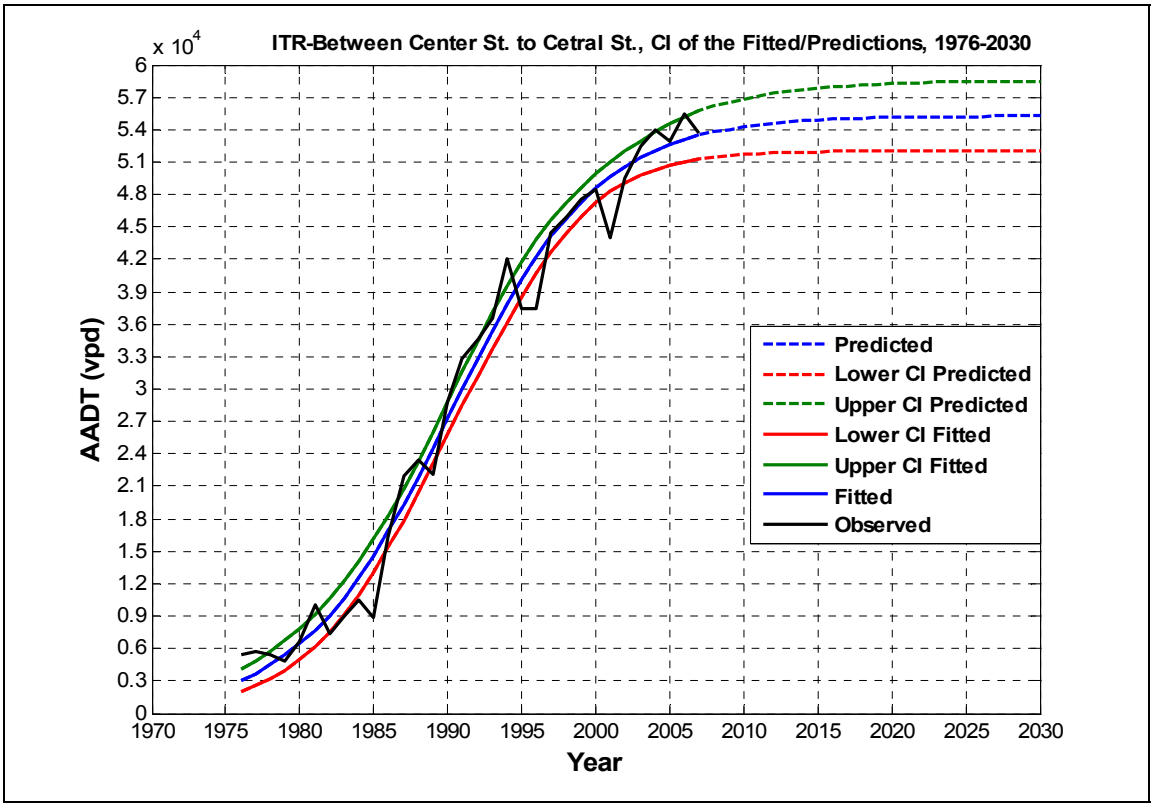


Figure 6.2: Illustration of Developed Logistic Function

6.2.2 Crash Prediction Model

Different generalized linear models have been tested for crash data modeling. These models included negative binomial (NB), Poisson, zero inflated negative binomial, ad zero inflated Poisson. The generalized linear models are preferred since they are the most suitable for count data modeling, like crashes, and whose predictions are limited to positive outcomes. The series of tests was performed with the available crash data to determine which model was suitable for modeling. These tests included test of the mean

and variance, over-dispersion test, test of alpha and Vuong's test for zero inflated. After going through all the tests, Negative Binomial (NB) was found to be the most suitable and then used for final crash model. To determine which variables were significant, the t-values (*p*-values) and coefficient signs for each variable was examined. The variable whose coefficient sign reflected expected impact to the crash occurrence and whose t-value showed more than 85% significant level was selected for the final model. After all modeling procedures, seven final variables were selected. These variables included annual daily traffic volume (ADT), directional split, median width, lane width, shoulder width, number of lanes, and two way left turn (TWLT) median indicator. The developed crash prediction model is shown below.

$$CRASH/_{mile} = e^{(3.95E5*ADT + 0.036*D - 0.013*MW - 0.15*LW - 0.07*SW + 0.35*LANES + 1.26*TWLT)}$$

The signs of these variables in the models indicate the increase in traffic volume and directional split will increase the probability of crashes. Also segments with TWLT medians have a high probability of increasing crash frequency compared to undivided segments. For the roadway cross-sections with wider median, shoulder and lanes lowers crash frequency compared to narrow widths. More number of lanes increases the likelihood of crashes compared to locations with fewer number of lanes. This developed crash prediction model is used in the safety mitigation fee calculation by predicting before and after crash frequencies.

6.2.3 Safety Mitigation Fee Estimation

In order to develop safety mitigation fee, crash frequency before and after the development are calculated utilizing the developed crash prediction model. The predicted crash can be per unit or per trip. To evaluate the impact of the new development to the safety, the crash frequency before the new development is deducted from those after the development. The difference between the two is assumed to reflect the magnitude of the how the development impacted the safety. The developed safety (crash) mitigation fee is shown below; the initials are detailed in chapter five.

$$CMF = \frac{Max\left(CRASH_{after}(1-ARF) - CRASH_{before}, 0\right) * CRASHCOST * TTL}{\text{UNITS} * \text{TR} * \text{CRF}}$$

The crash cost is taken as $28,000/crash, the value extracted from different insurance companies and FHWA.

6.2.4 Delay Mitigation Fee Estimation

The delay mitigation fee estimation applied the principle of congestion pricing. The different between the marginal and total travel time is used as delay. This difference is multiplied with the value of time, vehicle occupancy, and number of travel days in the year. The developed delay mitigation fee estimation is show below.

$$DMF = (t^{*} - t_1) * VOT * VHO * NTD \Big/ CRF$$

Where "t*" is the marginal travel time and "t" is the average travel time. The value of time (VOT) is used as $7.5/hr (or approximately minimum wage). Vehicle occupancy

(VHO) is used as 1.2persons/vehicle. Number of working travel days (NTD) is used as 261/year.

6.2.5 Special Combined Safety-Delay Mitigation Fee Estimation

The combined safety and delay mitigation fee estimation considered these two parameters in one equation. It is simple the combination of safety mitigation equation and delay equation. The equation developed in chapter five is shown below.

$$SMF = CMF + DMF$$

Where

$$CMF = \frac{Max\left(CRASH_{after}(1-ARF) - CRASH_{before}, 0\right) * CRASHCOST * TTL}{\text{UNITS} * \text{TR} * \text{CRF}}$$

$$DMF = (t^{*} - t_1) * VOT * VHO * NTD \Big/ CRF$$

SMF = Special Mitigation Fee per Trip

CMF = Safety Mitigation Fee per Trip

DMF = Delay Mitigation Fee per Unit

6.3 Research Contributions

Through the completion of this study, the following are new contributions developed.

(i) The study introduced a reasonably accurate yet tractable traffic projection approach utilizing logistic function;

(ii) The study first introduces and develops safety mitigation fee as a special impact fee; and

(iii) The study first introduced and developed delay mitigation fee as a special impact fee based on the congestion pricing concept.

6.4 Recommendations for Future Study

This study considered a small scale 1.5 miles radius of study. The variables and values used in the analysis and in illustrative examples reflected the impact of 300 units residential development. Future studies should consider large and mixed-use developments. The derivation of delay mitigation fee used Bureau of Public Roads (BPR) link travel time function with model parameters calibrated to different free-flow speeds and traffic signal density for surface street conditions. Future studies may consider the use of other more macroscopic travel time functions specifically developed for surface streets and signalized intersections if higher level of accuracy is desired. Furthermore, the safety mitigation fee considered simple before and after crash frequency in determining the safety impact created by the development. Future studies may consider the use of risk analysis in determining in detail, the crash risk imposed by the new developments.

REFERENCES

1. Palm Beach County. Comprehensive Plan Ordinances 2006-20 & 22-27. Transportation Element, 2006

2. Broward County. Broward County Land Development Code. http://www.broward.org/development/landdevcode.htm . Accessed on 9/21/2007.

3. Karen, E. S., Kristine M, W. Pei-Sung Lin and Fatih P. Transportation Concurrency Requirements and Best Practices: Guidelines for Developing and Maintaining; An Effective Transportation Concurrency Management System. *Center for Urban Transportation Research*, September 2006.

4. Sarasota County. Land Development Regulations. www.scgov.net. Accessed September 2007

5. Miami Dade County. Comprehensive Development Master Plan (CDMP) http://www.miamidade.gov/planzone/CDMP.asp. Accessed October , 2007

6. City of Tallahassee. City Comprehensive Plan. Transportation Goals, Objectives and policies. http://www.talgov.com/planning/pdf/compln/thecompplan/transp.pdf. Accessed September 21, 2007

7. Kristine M., W, Edward, A. M and Karen E. S: Current Practices and Regulatory Concepts, Model Fair Share Ordinance for Transportation Concurrency. *Center for Urban Transportation Research*, September 2005

8. Nicholas, J. C., and Arthur, N. "Determining the Appropriate Development Impact Fee Using the Rational Nexus Test." Journal of the American Planning Association. Chicago: Winter 1988, Vol. 54, Iss. 1, pp. 56-67

9. Duncan Associates. Impact Fee Study for Transportation, Parks, Library and Police Facilities. City of Destin, Florida, July 2007

10. Duncan Associates. Impact Fee Study for Roads and Parks. Town of Farragut, Tennessee, August 2007

11. Duncan Associates. Road Impact Fee Study. Lee County, Florida, October 2006

12. Noland, R.B., and Oh. L. The Effect of Infrastructure and Demographic Change on Traffic-Related Fatalities and Crashes: A Case Study of Illinois County-Level Data. *Accident Analysis and Prevention 36,* 2004, pp 525–532

13. Abdel-Aty, M. A., and Radwan, A. E. Modeling Traffic Accident Occurrence and Involvement. *Accident Analysis and Prevention 34,* 2000, Pp.633-642

14. Garber, N. J. *The Effect of Speed, Flow, and Geometric Characteristics on Crash Rates for Different Types of Virginia Highways*. Virginia Transportation Council, 2000

15. Garber, J. N., and Gadiraju, R. (1989) "Factors affecting speed variance and its influence on accidents." *Transportation Research Record No. 1213*, pp. 64-71

16. Retting, R. A., Weinstein, H. B., Williams, A. F., and Preusser, D. F. A Simple Method for Identifying and Correcting Crash Problems on Urban Arterial Streets. *Accident Analysis and Prevention 33*, 2001, pp.723-734

17. Ossenbruggen, P. J., Pendharkar, J., and Ivan, J. Roadway Safety in Rural and Small Urbanized Areas. *Accident Analysis and Prevention 33*, 2001, pp.485-495

18. Karlaftis, M. G., and Golias. I. Effect of Road Geometry and Traffic Volumes on Rural Roadway Accident Rates. *Accident Analysis and Prevention 34,* 2002, pp.357-365

19. Greibe, P. Accident Models for Urban Roads. *Accident Analysis and Prevention Vol.35 December 2001, pp.273-285*

20. Qin, X., John, N., Ivan and Ravishanker. Selecting Exposure Measures in Crash Rate Prediction for Two-Lane Highway Segments. *Accident Analysis and Prevention 36,* 2004, pp.183-191

21. Zegeer, C.V., Stewart, R. and Council, F. *NCHRP Report 362: Roadway Width For Low-Traffic-Volume Roads. National Academy Press,* Washington, DC, 1994

22. Mouskos, K. C., Sun, W., and Qu. T. *Impact of Access Driveways on Accident Rates At Multilane Highways*. National Center for Transportation and Industrial Productivity, New Jersey Institute of Technology, 1999

23. Hadi, M.A., Aruldhas. J., Chow. L. F., And Wattleworth, J. A. "Estimating Safety Effects of Cross-Section Design For Various Highway Types Using Negative Binomial Regression". *Transportation Research Center,* University Of Florida, 1995

24. Milton, J., and Mannering, F. The Relationship among Highway Geometrics, Traffic-Related Elements and Motor-Vehicle Accident Frequencies. *Transportation 25*, 1998, pp. 395-413

25. Aruldhas, J. *Examination of Statistical Relationships between Highway Crashes and Highway Geometric and Operational Characteristics of Two-Lane Urban Highways*. PhD Thesis, University of Florida, 1998

26. Sawalha, Z. *Statistical Issues in Traffic Accident Modeling*. In Proceedings of the 82th Annual Meeting of the Transportation Research Board, January 12-16, Washington, D.C, 2003

27. Poch, M., and Mannering, F. *Negative Binomial Analysis of Intersection-Accident Frequencies*. Journal of Transportation Engineering, 1996

28. Gluck, J., Levinson H. S., and Stover, V. G. (1999). "Impacts of access management techniques." *NCHRP Report 420,* National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C

29. Papayannoulis, V., Gluck, J S., Feeney, K. *Access Spacing and Traffic Safety*. TRB Circular EC019: Urban Street Symposium.

30. Florida Department of Transportation. Site Impact Handbook, April 1997

31. Florida Department of Transportation (FDOT) Safety Office. Strategic Highway Safety Plan, February 2003

32. National Cooperative Highway Research Program (NCHRP). Incorporating Safety into Long-Range Transportation Planning. Report # 546, 2006

33. U.S. Department of Transportation. Considering Safety in the Transportation Planning Process. http://tmip.fhwa.dot.gov/clearinghouse/docs/safety/. Accessed October 2007

34. Kelvin, R. The Development of the Safer Transportation Network Planning Process. Safety Conscious Planning. Institute of Transportation Engineers, January 2001

35. Harwood, D.W., Council, F.M., Hauer, E., Hughes, W.E., and Vogt, A. Prediction Of The Expected Safety Performance Of Rural Two-Lane Highways, September 2000.

36. Sawalha, Z., and Sayed, T. (2001). *Evaluating Safety of Urban Arterial Roadways*. Journal of Transportation Engineering .

37. Harwood, D. W. *Multilane Design Alternatives for Improving Suburban Highways*. NCHRP Report 282, National Cooperative Research Program, Transportation Research Board, Washington, D.C, 1986

38. Ivan, J. N., Pasupathy, R. K., and Ossenbruggen, P. J. (1999). "Differences in Causality Factors for Single and Multi-Vehicle Crashes on Two-Lane Roads." *Accident Analysis and Prevention 31,* pp.695-704

39. Tarso, P., Resende, V., and Benekohal, R.F. *Effects of Roadway Section Length on Accident Modeling.* Traffic Congestion and Traffic Safety in the 21st Century. Proceedings of the ASCE Conference, Chicago, IL, June 8-11, 1997, pp. 403-409

40. Martin, J. L. Relationship between Crash Rate and Hourly Traffic Flow on Interurban Motorways. *Accident Analysis and Prevention 34,* 2002*,* pp.619-629

41. Hiselius, L. Wt. Estimating the Relationship between Accident Frequency and Homogeneous and Inhomogeneous Traffic Flows. *Accident Analysis and Prevention Article in Press, 2004*

42. Miaou, S. The Relationship between Truck Accidents and Geometric Design of Road Sections: Poisson versus Negative Binomial Regressions. *Accident Analysis and Prevention Vol.26 September 1993, pp.471-482*

43. Persaud, B., Hauer E., Retting, R., Vallurupalli, R., and Mucsi, K. *Crash Reductions Related to Traffic Signal Removal in Philadelphia*, 1997

44. Turner, S., and Nicholson, A. Intersection Accident Estimation. The Role of Intersection Location and Non-Collision Flows. *Accident Analysis and Prevention Vol.30 1998, pp.505-517*

45. Miaou, S. and Lump, H. Modeling Vehicle Accidents and Highway Geometric Design Relationships. *Accident Analysis and Prevention 25,* 1993, pp. 689-709

46. Cameron, A.C. And Trivedi, P.K. *Regression Analysis of Count Data.* Cambridge University Press, 1998

47. Washington. S. P., Karlaftis, M. G., and Mannering, F. L. *Statistical and Econometric Methods for Transportation Data Analysis*. Chapman & Hall/CRC, 2002

48. Hardin, J., Hilbe, J. *Generalized Linear Models and Extension*. Stata Corportation, 2001

49. Memmott, J. L., *Factors That Affect Traffic Growth Rates and Projection of Traffic Volumes for Use in Highway Economic Models*. Transportation Research Board, Issue Number: 912, 1981

50. Johnson, D, L., 20-Year Traffic Forecasting Factors. *South Dakota Department of Transportation Office of Research Final Report*, October 2000.

51. Lee, M., Chootinan, P., Chen A., and Recker, W., Forecasting Network Traffic for Small and Medium-sized Communities using Path Flow Estimator. *85th Annual Meeting of the Transportation Research Board Proceedings*, Washington D.C., 2006

52. URS Corporation., History and Projection of Traffic, Toll Revenues and Expenses and Review of Physical Conditions. *Triborough Bridge and Tunnel Authority*, April 2006.

53. Perone, S., Carr, T., and Upton, D., Forecasting Traffic Impacts on a City-Wide Street Network in Absence of a Travel Demand Model. *85th Annual Meeting of the Transportation Research Board Proceedings*, Washington D.C., 2006

54. Anderson, D. M., Sharfi, K., and Gholston, E. S., A Small Urban Community Direct Demand Forecasting Model Using Multiple Linear Regression. *85th Annual Meeting of the Transportation Research Board Proceedings*, Washington D.C., 2006

55. Kikuchi, S., Gupta, A., Felsen, M and Brockenbrough, C, W., Trip Attraction Characteristics of Neighborhood – and Community -Level Shopping Centers. *85th Annual Meeting of the Transportation Research Board Proceedings*, Washington D.C., 2006

56. U.S. Department of Transportation., Economic Analysis Primer. *Federal Highway Administration Office of Asset Management*, August 2003

57. United States General Accounting Office., Highway Infrastructure, FHWA's Model for Estimating Highway Needs Is Generally Reasonable, Despite Limitations. www.gao.gov/archive/2000/rc00133.pdf , accessed June 24, 2007.

58. Xiao-Yuan, W., Hai-hong, L., Zhi-ping, L. Short-Time Traffic Flow Forecasting Based on Projection Pursuit Auto-Regression," *ICICIC*, pp. 707-711, *First International Conference on Innovative Computing, Information and Control* - Volume I (ICICIC'06), 2006

59. Florida Department of Transportation. Project Traffic Forecasting Handbook. *Chapter 5, Forecasting without a Traffic Model*, October 2002.

60. Zhao, Y., and Kockelman, K, M. On-line Marginal-cost Pricing across Networks: Incorporating Heterogeneous users and Stochastic Equilibria. *Transportation Research Part B 40, pg (424–435)*, 2006.

61. Grant-Muller, S., and Laird, J. Costs of Congestion: Literature Based Review of Methodologies and Analytical Approaches. *Institute for Transport Studies, University of Leeds*. Scottish Executive Social Research, 2006

62. Lindsey, C., R and Erik, V., T. Traffic Congestion and Congestion Pricing. *Tinbergen Institute Discussion Paper*, 2000.

63. Gan, A., Shen, J., and Rodriguez, A. Update of Florida Crash Reduction Factors and Countermeasures to improve the Development of District Safety Improvement Projects. *Final Report, Safety Office, Florida Department of Transportation*, 2005.

64. Whitney, D. The Value of Travel-Time: Estimates of the Hourly Value of Time for Vehicles in Oregon. *Oregon Department of Transportation*, 2004.

65. Wang, J-S, R. R. Knipling, and L. J. Blincoe, The Dimensions of Motor Vehicle Crash Risk, *Journal of Transportation and Statistics*, MAY 1999, pp. 19-43.

66. Blincoe L, J., Faigin B, M. The Economic Cost of Motor Vehicle Crashes, 1990. Washington, DC: *US Department of Transportation, National Highway Traffic Safety Administration*, 1992; report no. DOT-HS-807-876.

67. Chang-Jen, L. New Optimal Cycle Length Formulation for Pretimed Signals at Isolated Intersections. J*ournal of Transportation Engineering, ASCE*, October 2004.

68. Beyene, J., and Moineddin, R. Methods for Confidence Interval Estimation of A Ratio Parameter with Application to Location Quotients. *BMC Medical Research Methodology*, October 2005.

69. Raykov, T., and Marcoulides, G. A. Using the Delta Method for Approximate Interval Estimation of Parameter Functions in SEM. *Structural Equation Modeling*, 2004, pg 621-637.

APPENDICES

I. Matlab Codes

Logistic Traffic Projection Code

```
clear all
clc;
%x=[1:16 18:32]';
%training data: AADT from 1976 to 1991, validating data 1991-2007
v=[5410 5767    5428    4795    6595    10009   7356    9016    10431
8836    16351   21994   23515   22073   28854   32836   34668   36500
42000   37500   37500   44500   46000   47500   48500   44000   49500
52500   54000   53000   55500   53600]';
[m,n]=size(v);
x=[1:m]';

% v=ln(ax^b*exp(cx))
xx=[ones(m,1) log(x) x];
b=inv(xx'*xx)*(xx'*v)
vh(:,1)=xx*b;

% v=ax^b*exp(cx)
xx=[ones(m,1) log(x) x];
b=inv(xx'*xx)*(xx'*log(v))
vh(:,2)=exp(xx*b);

% v=ax^b
xx=[ones(m,1) log(x)];
b=inv(xx'*xx)*(xx'*log(v));
vh(:,3)=exp(xx*b);

% v=ln(ax^b)
xx=[ones(m,1) log(x)];
b=inv(xx'*xx)*(xx'*v);
vh(:,4)=xx*b;

%logit - C/(1+a*exp(bx))
p0=[10 0.01 70000]';
options = optimset('Display','iter');
LB=[eps eps 51800]';
UB=[1e5 1e5 70200]';
p  =
fmincon(@(p)myfun(p,x,v),p0,[],[],[],[],LB,UB,@(p)mycon(p),options)
vh(:,5)=p(3)./(1+p(1)*exp(-p(2)*x));
x1=[1:m+16]';
vh1=p(3)./(1+p(1)*exp(-p(2)*x1));
x2=[1:m+23]';
vh2=p(3)./(1+p(1)*exp(-p(2)*x2));

a=p(1); b=p(2); C=p(3);
A = zeros(3,3);
dis = v-vh(:,5);
```

```
vr = var(v-vh(:,5))
for k=1:m
    dn3=(1+a*exp(-b*x(k))).^3;
    dn2=(1+a*exp(-b*x(k))).^2;
    dn1= 1+a*exp(-b*x(k));
    dif = dis(k);
        H(1,1)= 2*C*exp(-b*2*x(k))./dn3;
        H(1,2)= x(k)'*C*exp(-b*x(k))*(1-a*exp(-b*x(k)))./dn3;
        H(1,3)= (-1)*exp(-b*x(k))./dn2;
        H(2,1)= x(k)'*C*exp(-b*x(k))*(1-a*exp(-b*x(k)))./dn3;
        H(2,2)= (x(k)').^2*C*p(1)*exp(-b*x(k))*(a*exp(-b*x(k))-1)./dn3;
        H(2,3)= (x(k)')*a*exp(-b*x(k))./dn2;
        H(3,1)= (-1)*exp(-b*x(k))./dn2;
        H(3,2)= (x(k)')*a*exp(-b*x(k))./dn2;
        H(3,3)= 0;

        dvda = -C*exp(-b*x(k))./dn2;
        dvdb = x(k)'*a*C*exp(-b*x(k))./dn2;
        dvdc = 1./dn1;
        dv=[dvda dvdb dvdc];

      A = A+(dif*H-dv'*dv)./vr;

end
T =p./sqrt(diag(inv(-A')))

% Calculation of Cofidence Intervals
CO=inv(-A');
vht=vh(:,5);
for k=1:m
    dn3=(1+a*exp(-b*x(k))).^3;
    dn2=(1+a*exp(-b*x(k))).^2;
    dn1= 1+a*exp(-b*x(k));
    dif = dis(k);
        dvdat = -C*exp(-b*x(k))./dn2;
        dvdbt = x(k)'*a*C*exp(-b*x(k))./dn2;
        dvdct = 1./dn1;
    dvt=[dvdat dvdbt dvdct];
    variance=dvt*CO*dvt';
    CI_LB(k) =vht(k)-2.042*sqrt(variance);
    CI_UB(k) =vht(k)+2.042*sqrt(variance);
end
y1=[1976:2007];
y2=[1976:2030];

for k=1:m+23
    dn3=(1+a*exp(-b*x2(k))).^3;
    dn2=(1+a*exp(-b*x2(k))).^2;
    dn1= 1+a*exp(-b*x2(k));
        dvdat = -C*exp(-b*x2(k))./dn2;
        dvdbt = x2(k)'*a*C*exp(-b*x2(k))./dn2;
        dvdct = 1./dn1;
    dvt=[dvdat dvdbt dvdct];
```

```
    variance=dvt*CO*dvt';
    CI_LB2(k)= vh2(k)-2.042*sqrt(variance);
    CI_UB2(k)= vh2(k)+2.042*sqrt(variance);
end
figure;
plot(y2,vh2,'--',y2,CI_LB2,'--',y2,CI_UB2,'--
',y1,CI_LB,y1,CI_UB,y1,vht,y1,v)

title('ITR-Between Center St. to Cetral St., CI of the
Fitted/Predictions, 1976-2030')
legend('Predicted','Lower CI Predicted','Upper CI Predicted','Lower CI
Fitted','Upper CI Fitted','Fitted','Observed')
xlabel('Year')
ylabel('AADT (vpd)')
```

myfun.m

```
function f=myfun(p,x,v)
a=p(1); b=p(2); C=p(3);
vh=C./(1+a*exp(-b*x));
[m,n]=size(v);
f=sqrt((v-vh)'*(v-vh)/m);
```

mycon.m

```
function [c,ceq]=mycon(p)
c=[];
ceq=[];
```

Delay Mitigation Fee Code

```
clear all
clc;
L=[0.34 0.18 0.25 0.14 0.93 0.51 0.13 0.39]; %length of the links in
the corridor
V2030=[70166 63615 60633 61312 51454 54354 62019 58053] % ADT of the
links in the corridor at 2030
V=V2030;
vmax=[84199 76338   72760   73574   61745   65225   74423   69664]
a=0.99; b=5.6; C=51800; sf=50;
V0=40000; option = []; %optimset('disp','iter');
for i = 1:8
    Vo(i) = fzero(@(v) mcmyfun30(v,L(i),a,b,C,sf,vmax(i)), V0, option);
end
% calculate ATL and MTL at Vo
ATL = L/sf.*(1+a*(Vo/C).^b)
MTL = L/sf.*(1+a*(Vo/C).^b)+ L/sf.*(a*b).*(Vo/C).^b
Vo
CRF = 0.1172; %assuming i=3% and N=10 years
DF=MTL-ATL;
TCost=DF*7.5*260*1.2./CRF
Total = sum(TCost)
V1=0:100:50000;
L1=2.87;
t01=L1/sf;
vmax1=mean(vmax);
voave=mean(Vo)
atlave=mean(ATL)*8
mtlave=mean(MTL)*8
ATL1 = L1/sf.*(1+a*(V1/C).^b);
MTL1 = L1/sf.*(1+a*(V1/C).^b)+ L1/sf.*(a*b).*(V1/C).^b;
t1=t01+(3*t01*(V1-vmax1)./(-vmax1));
figure;
plot(V1,ATL1,V1,MTL1,V1,t1,'k--')
title('ATT and MTT: Island Way to US1 Corridor, L=2.87 mi, Year 2030')
legend('Average Travel Time (ATT)','Marginal Travel Time (MTT)','Demand
Line')
ylabel('Time (Hours)')
xlabel('Volume (vph)')
```

mcmyfun30.m

```
function f1 = mcmyfun(v,L,a,b,C,sf,vA)
t0=L/sf; tB=4*t0; vA=1.2*v; % define demand function
f1=L/sf.*(1+a*(b+1)*(v/C).^b) - (t0+(3*t0*(v-vA)./(-vA)));
```

Stochastic User Equilibrium Assignment Code

```
clear all
clc;
v=0:20:1200; % Initialize ranges for the traffic volumes
V0=1000; option = [];
% Finding the optimal traffic volume based on logit funtion
for i = 1:61
    V1 = fzero(@(v) mysue1(v), V0, option);
    V2 = fzero(@(v) mysue2(v), V0, option);
    V3 = fzero(@(v) mysue3(v), V0, option);
    V4 = fzero(@(v) mysue4(v), V0, option);
end
Vol_EB_AM_ITR=V1;
Vol_WB_AM_ITR=V2;
Vol_EB_PM_ITR=V3;
Vol_WB_PM_ITR=V4;

%Eastbound AM Results based on Logit Function
t11=(0.047+0.05*(V1./2570).^5.75)*60;
t12=(0.071+0.048*((855-V1)./760).^5)*60;
theta1=-0.094;
theta2=-0.009;
theta3=-0.002;
Cost1=40;
Cost2=60;
C11=(V1./2570).^2*t11;
C12=((855-V1)./760).^2*t12;
GC11=theta1*t11+theta2*Cost1+theta3*C11;
GC12=theta1*t12+theta2*Cost2+theta3*C12;
P11=exp(-GC11)./(exp(-GC11)+exp(-GC12));
P12=exp(-GC12)./(exp(-GC11)+exp(-GC12));
error1=855*P11-V1;

%Westbound AM Results based on Logit Function
t21=(0.047+0.05*(V2./2570).^5.75)*60;
t22=(0.071+0.048*((393-V2)./760).^5)*60;
theta1=-0.094;
theta2=-0.009;
theta3=-0.002;
Cost1=40;
Cost2=60;
C21=(V2./2570).^2*t21;
C22=((393-V2)./760).^2*t22;
GC21=theta1*t21+theta2*Cost1+theta3*C21;
GC22=theta1*t22+theta2*Cost2+theta3*C22;
P21=exp(-GC21)./(exp(-GC21)+exp(-GC22));
P22=exp(-GC22)./(exp(-GC21)+exp(-GC22));
error2=393*P21-V2;

%Eastbound PM Results based on Logit Function
```

```
t31=(0.047+0.05*(V3./2570).^5.75)*60;
t32=(0.071+0.048*((527-V3)./760).^5)*60;
theta1=-0.094;
theta2=-0.009;
theta3=-0.002;
Cost1=40;
Cost2=60;
C31=(V3./2570).^2*t31;
C32=((527-V3)./760).^2*t32;
GC31=theta1*t31+theta2*Cost1+theta3*C31;
GC32=theta1*t32+theta2*Cost2+theta3*C32;
P31=exp(-GC31)./(exp(-GC31)+exp(-GC32));
P32=exp(-GC32)./(exp(-GC31)+exp(-GC32));
error3=527*P31-V3;

%Westbound PM Results based on Logit Function
t41=(0.047+0.05*(V4./2570).^5.75)*60;
t42=(0.071+0.048*((689-V4)./760).^5)*60;
theta1=-0.094;
theta2=-0.009;
theta3=-0.002;
Cost1=40;
Cost2=60;
C41=(V4./2570).^2*t41;
C42=((689-V4)./760).^2*t42;
GC41=theta1*t41+theta2*Cost1+theta3*C41;
GC42=theta1*t42+theta2*Cost2+theta3*C42;
P41=exp(-GC41)./(exp(-GC41)+exp(-GC42));
P42=exp(-GC42)./(exp(-GC41)+exp(-GC42));
error4=689*P41-V4;

%Summar of Results based on Logit Function
Vol_EB_AM_ITR;
Vol_EB_AM_Toney=855-V1;
t11; t12; GC11; GC12; P11; P12; error1;

Vol_WB_AM_ITR;
Vol_WB_AM_Toney=393-V2;
t21; t22; GC21; GC22; P21; P22; error2;

Vol_EB_PM_ITR;
Vol_EB_PM_Toney=527-V3;
t31; t32; GC31; GC32; P31; P32; error3;

Vol_WB_PM_ITR;
Vol_WB_PM_Toney=689-V4;
t41; t42; GC41; GC42; P41; P42; error4;
 %SUE analysis based on Cost Utility Logit Function e.g q*P(i)-V(i)=0
for i = 1:61
    Vo1 = fzero(@(v) mysue5(v), V0, option);
    Vo2 = fzero(@(v) mysue6(v), V0, option);
    Vo3 = fzero(@(v) mysue7(v), V0, option);
    Vo4 = fzero(@(v) mysue8(v), V0, option);
```

```
end
V_EB_AM_ITR=Vo1          %Eastbound AM ITR
V_EB_AM_Toney=855-Vo1      %Eastbound AM Toney

V_WB_AM_ITR=Vo2            %Westbound AM ITR
V_WB_AM_Toney=393-Vo2       %Westbound AM Toney

V_EB_PM_ITR=Vo3          %Eastbound PM ITR
V_EB_PM_Toney=527-Vo3     %Eastbound PM Toney

V_WB_PM_ITR=Vo4         %Westbound PM ITR
V_WB_PM_Toney=689-Vo4    %Westbound PM Toney

t01=(0.047+0.05*(Vo1./2570).^5.75)*60  %Eastbound AM ITR
t02=(0.071+0.048*((855-Vo1)./760).^5)*60 %Eastbound AM Toney

t03=(0.047+0.05*(Vo2./2570).^5.75)*60 %Westbound AM ITR
t04=(0.071+0.048*((393-Vo2)./760).^5)*60 %Westbound AM Toney

t05=(0.047+0.05*(Vo3./2570).^5.75)*60 %Eastbound PM ITR
t06=(0.071+0.048*((527-Vo3)./760).^5)*60 %Eastbound PM Toney

t07=0.047+0.05*(Vo4./2570).^5.75*60 %Westbound PM ITR
t08=0.071+0.048*((689-Vo4)./760).^5*60 %Westbound PM Toney

C1=(Vo1./2570).^2*t01;
C2=((855-Vo1)./760).^2*t02;
GC1=theta1*t01+theta2*Cost1+theta3*C1 %Eastbound AM ITR
GC2=theta1*t02+theta2*Cost2+theta3*C2 %Eastbound AM Toney

C3=(Vo2./2570).^2*t03;
C4=((393-Vo2)./760).^2*t04;
GC3=theta1*t03+theta2*Cost1+theta3*C3 %Westbound AM ITR
GC4=theta1*t04+theta2*Cost2+theta3*C4 %Westbound AM Toney

C5=(Vo3./2570).^2*t05;
C6=((527-Vo3)./760).^2*t06;
GC5=theta1*t05+theta2*Cost1+theta3*C5  %Eastbound PM ITR
GC6=theta1*t06+theta2*Cost2+theta3*C6  %Eastbound PM Toney

C7=(Vo4./2570).^2*t07;
C8=((689-Vo4)./760).^2*t08;
GC7=theta1*t07+theta2*Cost1+theta3*C7  %Westbound PM ITR
GC8=theta1*t08+theta2*Cost2+theta3*C8  %Westbound PM Toney
```

II. Construction Costs

Roadway Cost Per Centerline Mile
Revised August 2007

	Construction Cost From LRE	MOT *	Mobilization *	Subtotal	Scope Contingency (25%)	Total Construction Cost	PE Design (15%)	CEI (15%)	Total Project Cost **
Rural Arterial									
New Construction (2-Lane Roadway) with 5' Paved Shoulders	$6,885,346	$688,535	$757,388	$8,331,268	$2,082,817	**$10,414,085**	$1,562,113	$1,562,113	**$13,538,311**
New Construction (4-Lane Roadway) with 5' Paved Shoulders	$10,435,915	$1,043,592	$1,147,951	$12,627,458	$3,156,864	**$15,784,322**	$2,367,648	$2,367,648	**$20,519,619**
New Construction (6-Lane Roadway) with 5' Paved Shoulders	$12,888,225	$1,288,823	$1,417,705	$15,594,752	$3,898,688	**$19,493,440**	$2,924,016	$2,924,016	**$25,341,472**
Milling and Resurfacing (4-Lane Roadway) with 5' Paved Shoulders	$1,407,175	$140,718	$154,789	$1,702,682	$425,670	**$2,128,352**	$319,253	$319,253	**$2,766,858**
Milling and Resurfacing (6-Lane Roadway) with 5' Paved Shoulders	$2,107,462	$210,746	$231,821	$2,550,029	$637,507	**$3,187,536**	$478,130	$478,130	**$4,143,797**
Add Lanes (2 to 4 Lanes) with 5' Paved Shoulders (Includes milling and resurfacing of existing pavement)	$6,273,712	$627,371	$690,108	$7,591,192	$1,897,798	**$9,488,990**	$1,423,348	$1,423,348	**$12,335,687**
Add Lanes (4 to 6 Lanes) with 5' Paved Shoulders (Includes milling and resurfacing of existing pavement)	$6,697,610	$669,761	$736,737	$8,104,108	$2,026,027	**$10,130,135**	$1,519,520	$1,519,520	**$13,169,176**
Add Lanes (4 to 8 Lanes) with 5' Paved Shoulders (Includes milling and resurfacing of existing pavement)	$8,977,014	$897,701	$987,472	$10,862,187	$2,715,547	**$13,577,734**	$2,036,660	$2,036,660	**$17,651,054**
Add Lanes (6 to 8 Lanes) with 5' Paved Shoulders (Includes milling and resurfacing of existing pavement)	$8,385,494	$838,549	$922,404	$10,146,448	$2,536,612	**$12,683,060**	$1,902,459	$1,902,459	**$16,487,978**
Add 1 Through Lane on Inside (To Existing) with 5' Paved Shoulders	$1,467,587	$146,759	$161,435	$1,775,780	$443,945	**$2,219,725**	$332,959	$332,959	**$2,885,643**
Add 1 Through Lane on Outside (To Existing) with 5' Paved Shoulders	$2,092,850	$209,285	$230,214	$2,532,349	$633,087	**$3,165,436**	$474,815	$474,815	**$4,115,067**
Add 300' Exclusive Left Turn Lane	$68,509	$10,276	$11,818	$90,603	$22,651	**$113,254**	$16,988	$16,988	**$147,230**
Add 300' Exclusive Right Turn Lane	$144,737	$21,711	$24,967	$191,415	$47,854	**$239,269**	$35,890	$35,890	**$311,050**
Urban Arterial									
New Construction (2-Lane Roadway) with 5' Sidewalk, and Curb & Gutter	$10,007,673	$1,000,767	$1,100,844	$12,109,284	$3,027,321	**$15,136,606**	$2,270,491	$2,270,491	**$19,677,587**
New Construction (4-Lane Roadway) with 5' Sidewalk, and Curb & Gutter	$13,241,675	$1,324,167	$1,456,584	$16,022,427	$4,005,607	**$20,028,033**	$3,004,205	$3,004,205	**$26,036,443**
New Construction (6-Lane Roadway) with 5' Sidewalk, and Curb & Gutter	$15,714,461	$1,571,446	$1,728,591	$19,014,497	$4,753,624	**$23,768,122**	$3,565,218	$3,565,218	**$30,898,558**
Milling and Resurfacing (4-Lane Roadway) with 5' Sidewalk, and Curb & Gutter	$1,550,212	$155,021	$170,523	$1,875,757	$468,939	**$2,344,696**	$351,704	$351,704	**$3,048,104**
Milling and Resurfacing (6-Lane Roadway) with 5' Sidewalk, and Curb & Gutter	$2,195,587	$219,559	$241,515	$2,656,660	$664,165	**$3,320,826**	$498,124	$498,124	**$4,317,073**
Add Lanes (2 to 4 Lanes) with 5' Sidewalk, and Curb & Gutter (Includes milling and resurfacing existing pavement)	$7,102,642	$710,264	$781,291	$8,594,197	$2,148,549	**$10,742,746**	$1,611,412	$1,611,412	**$13,965,570**
Add Lanes (4 to 6 Lanes) with 5' Sidewalk, and Curb & Gutter (Includes milling and resurfacing existing pavement)	$7,771,231	$777,123	$854,835	$9,403,189	$2,350,797	**$11,753,987**	$1,763,098	$1,763,098	**$15,280,183**
Add Lanes (4 to 8 Lanes) with 5' Sidewalk, and Curb & Gutter (Includes milling and resurfacing existing pavement)	$10,373,766	$1,037,377	$1,141,114	$12,552,257	$3,138,064	**$15,690,321**	$2,353,548	$2,353,548	**$20,397,418**
Add Lanes (6 to 8 Lanes) with 5' Sidewalk, and Curb & Gutter (Includes milling and resurfacing existing pavement)	$9,245,287	$924,529	$1,016,982	$11,186,797	$2,796,699	**$13,983,497**	$2,097,524	$2,097,524	**$18,178,546**
Add 1 Through Lane on Inside (To Existing) with 5' Sidewalk, and Curb & Gutter	$1,463,935	$146,393	$161,033	$1,771,361	$442,840	**$2,214,201**	$332,130	$332,130	**$2,878,462**
Add 1 Through Lane on Outside (To Existing) with 5' Sidewalk, and Curb & Gutter	$3,666,987	$366,699	$403,369	$4,437,054	$1,109,263	**$5,546,317**	$831,948	$831,948	**$7,210,212**
Add 300' Exclusive Left Turn Lane	$86,370	$12,955	$14,899	$114,224	$28,556	**$142,780**	$21,417	$21,417	**$185,614**
Add 300' Exclusive Right Turn Lane	$190,499	$28,575	$32,861	$251,936	$62,984	**$314,919**	$47,238	$47,238	**$409,395**

*** A 15% MOT and Mobilization factor was used for exclusive left and right turn lanes. A 10% factor was used for all other figures.**
**** Total cost shown is derived from a standard typical section. Costs will need to be adjusted to account for signals, bridges, or any additional item not deemed typical.**

Note:
1. **Estimates were derived from FDOT LRE system**
2. **These figures exclude costs for intersections/interchanges, improvements to cross streets, bridges over 20', right-of-way, landscaping, ITS, and traffic signals.**
3. **The figures are based on market costs for Hillsborough County.**
4. **Costs shown are present day costs.**
5. **The costs developed for this report are not project-specific and should be used for preliminary estimating purposes only.**

	Construction Cost From LRE	MOT *	Mobilization (15%)	Subtotal	Scope Contingency (25%)	Total Construction Cost	PE Design (15%)	CEI (15%)	Total Project Cost
Intersection Traffic Signalization (Mast Arm Assembly)**									
2-Lane Roadway Intersecting 2-Lane Roadway	$186,183	$27,928	$32,117	$246,228	$61,557	**$307,785**	$46,168	$46,168	**$400,120**
4-Lane Roadway Intersecting 4-Lane Roadway	$228,106	$34,216	$39,348	$301,670	$75,417	**$377,087**	$56,563	$56,563	**$490,214**
6-Lane Roadway Intersecting 6-Lane Roadway	$264,560	$39,684	$45,637	$349,881	$87,470	**$437,351**	$65,603	$65,603	**$568,556**
Bicycle and Pedestrian Facilities									
Sidewalks Per Mile (5' Width - 1 Side)	$186,765	$9,338	$29,416	$225,519	$56,380	**$281,899**	$42,285	$42,285	**$366,468**
Sidewalks Per Mile (6' Width - 1 Side)	$224,118	$11,206	$35,299	$270,623	$67,656	**$338,279**	$50,742	$50,742	**$439,762**
Multi-Use Trail Per Mile (12' Width - 1 Side)	$271,814	$13,591	$42,811	$328,216	$82,054	**$410,270**	$61,540	$61,540	**$533,351**
Stormwater Retention Facilities									
1 Acre Pond Site (6' Depth)	$395,634	$19,782	$62,312	$477,728	$119,432	**$597,160**	$89,574	$89,574	**$776,308**
Median Retrofit									
Convert 14' Center Turn Lane to 14' Raised Median (Per Mile)	$338,587	$50,788	$58,406	$447,781	$111,945	**$559,726**	$83,959	$83,959	**$727,644**
Cross Street Improvements									
Widen 1-Leg of Existing Rural 2-Lane Cross Street to Accommodate 2 Receiving Lanes, Dual Left Turn lanes, and Exclusive Right Turn Lane (Approximate Length of 0.25 Miles)	$1,724,941	$258,741	$297,552	$2,281,234	$570,308	**$2,851,542**	$427,731	$427,731	**$3,707,005**

* A 15% MOT factor was used for Traffic Signals, Median Retrofit, and Cross Street Improvements. A 5% factor was used for all other figures.

**The cost of traffic signalization assumes the installation of mast arms on all four legs of an intersection. To obtain the cost of signalizing a four-lane roadway intersecting a two-lane roadway, divide the signal cost of a four-lane roadway by two and add this figure to the signal cost of the two-lane roadway divided by two.

Notes:

1. Estimates were derived from FDOT LRE system
2. The figures are based on market costs for Hillsborough County.
3. Costs shown are present day costs.
4. The costs developed for this report are not site-specific and should be used for preliminary estimating purposes only.

Interchange Cost

Revised August 2007

	Construction Cost From LRE	MOT (10%)	Mobilization (10%)	Subtotal	Scope Contingency (25%)	Total Construction Cost	PE Design (15%)	CEI (15%)	Subtotal Project Cost
Single Point Urban Interchange (SPUI)	$ 36,383,575.03	$3,638,358	$4,002,193	$44,024,126	$11,006,031	**$55,030,157**	$8,254,524	$8,254,524	**$71,539,204**

Note:

1. Cost was derived from an LRE estimate to modify the existing diamond interchange at I-75/SR 54 to a single point urban interchange.
2. Cost shown is for construction only. Does not include Design, CEI, and right-of-way.

8/14/2007

Cat.	Landuse	ITE Code	Unit	Daily Rate/Equation	Pass-By %	AM Peak Hour		PM Peak Hour	
						In/Out	Rate/Equation	In/Out	Rate/Equation
Industrial	Light Industrial	110	1000 S.F.	6.97	5%	88/12	0.92	12/88	0.98
	Warehouse	150	1000 S.F.	4.96	5%	82/18	Ln(T) = 0.71 Ln(X) + 1.15	25/75	Ln(T) = 0.79 Ln(X) + 0.54
	Mini-Warehouse/SS	151	1000 S.F.	2.50	5%	59/41	0.15	51/49	0.26
Residential	Single Family Detached	210	Dwelling Unit	10	0%	25/75	T = 0.70(X) + 9.43	63/37	Ln(T) = 0.90 Ln(X) + 0.53
	Apartment	220	Dwelling Unit	7	0%	20/80	T = 0.49(X) + 3.73	65/35	0.62
	Condo/TH (Fee Simple)	230	Dwelling Unit	7	0%	17/83	Ln(T) = 0.80 Ln(X) + 0.26	67/33	Ln(T) = 0.82 Ln(X) + 0.32
	55+ SF Detached	251	Dwelling Unit	8	0%	38/62	0.41	61/39	0.51
	55+ SF Attached	252	Dwelling Unit	6	0%	38/62	0.28[a]	61/39	0.32[a]
	Congregate Care Facility	253	Beds	2.15	0%	61/39	0.06	56/44	0.17
	Assisted Living Facility	254	Beds	2.66	0%	65/35	0.14	44/56	0.22
Ldg	Hotel	310	Rooms	8.92	5%	58/42	0.67	49/51	0.7
Rec	Movie Theater	444/5	Seats	1.76	0%	N/A	0	36/64	0.08
	Health Club	492	1000 S.F.	32.93	5%	42/58	1.21	51/49	4.05
Institutional	Elementary School	520	Students	1.29	0%	55/45	0.42	45/55	0.28[b]
	Middle/Junior School	522	Students	1.62	0%	55/45	0.53	52/48	0.15
	High School	530	Students	1.71	0%	69/31	0.41	47/53	0.14
	Private School (K-8)	534	Students	Use Private K-12 rate	0%	55/45	0.90	47/53	0.61[b]
	Private School (K-12)	536	Students	2.48	0%	61/39	0.79	43/57	0.17
	Church/Synagogue	560	1000 S.F.	9.11	0%	54/46	0.72	52/48	0.66[c]
	Day Care	565	1000 S.F.	79.26	30%	53/47	12.79	47/53	13.18
	Library	590	1000 S.F.	54.00	5%	72/28	1.06	48/52	7.09
Med	Hospital	610	1000 S.F.	16.78	5%	67/33	1.20	33/67	1.18
	Nursing Home	620	1000 S.F.	3.72	5%	65/35	0.38	47/53	0.42
Office	General Office	710	1000 S.F.	Ln(T) = 0.77 Ln(X) + 3.65[d]	5%	88/12	Ln(T) = 0.80 Ln(X) + 1.55	17/83	Ln(T) = 0.74 Ln(X) + 1.83[e]
	Medical Office	720	1000 S.F.	36.13	5%	79/21	2.48	27/73	Ln(T) = 0.93 Ln(X) + 1.47
	Medical Office (Reduced)[f]	PBC	1000 S.F.	18.07	5%	79/21	1.24	27/73	T = 0.36(X) + 1.27
	Government Office	730	1000 S.F.	68.93	5%	Use AM/PM Ratio for Gen. Office		31/69	1.21

III. Historical AADT's

	ITR (Central-Center)	ITR (E of TP Entrance)	ITR (W of A1A)	Ok'bee Blvd (E of TP)	Ok'bee Blvd (W of TP)	Ok'bee (E of US 441)	SR7 (US 441, S of Forest Hill)
Year	AADT	AADT	AADT	AADT	AADT	AADT	AADT
1970		2901	2780		5163		5617
1971		2858	2937		6601		5572
1972		3446	2707		7073		5419
1973		2872	3010		5569		5863
1974		2814	2667		6141		6447
1975		3418	2897		8297		6549
1976	5410	4022	3930	22121	8564		8260
1977	5767	5271	6869	14842	6880		3429
1978	5428	3792	9808	14730	7989	5100	4999
1979	4795	3886	9674	16220	10911	7204	4261
1980	6595	5708	8944	20966	13407	9675	6510
1981	10009	6404	11423	24381	15026	10115	6409
1982	7356	6737	14826	25658	16622	11990	6245
1983	9016	6397	14322	25624	16447	11051	6687
1984	10431	5768	14378	23351	15885	11398	7408
1985	8836	6583	13423	21399	12794	11298	12526
1986	16351	12771	19132	32948	19534	12869	14990
1987	21994	17830	15766	35265	25053	14440	15599
1988	23515	23663	18312	36499	29453	20476	16231
1989	22073	19364	18156	33678	27041	16446	13063
1990	28854	20018	18280	39560	26395	19485	11572
1991	32836	24298	22140	37256	32239	24186	18064
1992	34668	25149	26000	39878	33370	27343	21032
1993	36500	26000	33000	42500	34500	30500	24000
1994	42000	28000	29000	39500	40000	27500	22500
1995	37500	27000	27500	50000	46000	31000	24500
1996	37500	29000	34000	55500	49500	36500	24000
1997	44500	26500	31000	59000	52500	36500	28500
1998	46000	19700	34000	55500	56000	39500	29000
1999	47500	24600	35500	61500	59000	39000	27500
2000	48500	29500	34500	65500	58500	41500	27500
2001	44000	23000	33500	64000	62000	44000	27500
2002	49500	23000	38000	64500	66500	44500	36500
2003	52500	36000	39500	66500	66000	48000	44000
2004	54000	35500	41500	69000	65000	47500	49500
2005	53000	39000	39500	63500	63500	48000	49500
2006	55500	50250	39500	67000	60500	51000	46500
2007	53600	44240	37052	61380	57529	47723	47416
2012	55532	41272	40585				
2030	59798	54382	54570	76154	60726	58226	58094

VITA

Deo Chimba started his primary studies at Mbagala Kuu primary school in Dar es Salaam, Tanzania in 1984 and completed standard seven in 1990. He then joined Kibasila Secondary school for ordinary level education from 1991 to 1994. Mr. Chimba attended Tabora Boys High School from 1995 to 1997 where he studied advanced level secondary school majoring in Physics, Chemistry and Mathematics.

He joined the University of Dar es Salaam in September 1998 where he pursued the Bachelor of Science in Civil Engineering degree and graduated in 2002. He then joined Florida State University (FSU) in the spring semester of 2003 for the Masters of Science in Civil Engineering degree, majoring in Transportation and Traffic engineering, and graduated in Fall of 2004. He then continued with Doctoral studies at University of Miami (UM) in Spring 2005 where he pursued his Ph.D in Civil Engineering, majoring in Transportation Engineering and graduated in May 2008. At the time of submitting this dissertation Mr. Chimba was working as Transportation Engineer with Stanley Consultants, Inc. located in West Palm Beach, Florida.