OPERATIONAL EVALUATION OF RIGHT TURNS FOLLOWED BY U-TURNS AS AN ALTERNATIVE TO DIRECT LEFT TURNS

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OPERATIONAL EVALUATION OF RIGHT TURNS FOLLOWED BY U-TURNS AS AN ALTERNATIVE TO DIRECT LEFT TURNS

(VOLUME III OF THREE REPORTS BASED ON THE PROJECT "METHODOLOGY TO QUANTIFY THE EFFECTS OF ACCESS MANAGEMENT ON ROADWAY OPERATIONS AND SAFETY")

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ABSTRACT

This project evaluated the safety and operational impacts of two alternative left-turn treatments from driveways/side streets. The two treatments were: (1) Direct left turns (DLT) and, (2) Right turns followed by U-turns (RTUT). Ten sites were selected for field data collection where each site experienced one or both of the left turn alternatives from the driveway or side street. Video cameras were set up on scaffoldings to achieve enough viewing height and all the traffic movements at the selected sites were recorded. These videotapes were later reviewed and data related to direct left turns or right turns followed by U-turn movements were gathered. Using the collected data, operational analysis was conducted using two methods, empirical model development and simulation.

Delay and travel time models were developed using the collected data, which indicated that under high major road and driveway volume conditions, vehicles making a direct left turn experienced longer delay and travel times than those that made a right turn followed by U-turn. The break-even points were also obtained for sample situations by using the models. Computer software was developed to represent the developed delay and travel time models so that the corresponding values could be obtained under any given situation. Speed reduction on major road traffic due to RTUT was much lower than that of DLT. Another model was developed to estimate the Ratio, which is the percentage of RTUT vehicles when both choices are available. More drivers were found to be making RTUT when left-turn-in volume (>200vph) and through volume (>4000vph) are high.

In all cases, field data confirmed the simulation models developed using CORSIM.

In addition, a before and after study was conducted at a site where a full median opening was converted into a directional median opening. The weighted average delay and travel time were much smaller for RTUT as compared to DLT. Reductions in total delays were 15% and 22% respectively, during peak and non-peak periods.

The findings indicated that RTUT has more merits than DLT under high volume conditions from a traffic operations standpoint.

1. INTRODUCTION

1.1 Background

As the nation's roadway system becomes more congested and the number of vehicular crashes increases, the importance of access management is increasing. The management of access has been identified as one of the most critical elements in roadway planning and design *(1).* Access management has been defined as the process of managing access to land development while simultaneously preserving the safety and efficiency of the surrounding roadway system *(2)*. It helps achieve the necessary balance between traffic movement and property access by careful control of the location, type, and design of driveways and street intersections. This is accomplished by classifying highways with respect to the level of access and mobility they are expected to provide, and then, identifying and applying the most effective techniques to preserve that function. The impacts of potential techniques on traffic performance and safety are important considerations when deciding which technique to implement.

Access management deals with the control and regulation of the spacing and design of medians, median openings, driveways, freeway interchanges, and traffic signals. Typical access management measures cover the type and design of medians and median openings; the location and spacing of intersections; the spacing and design of interchanges; and location, spacing, and design of driveways and street connections. The location, design, and operation of driveways play a significant role in access management. AASHTO Green Book, "A Policy on the Geometric Design of Highways and Streets", indicates that "Driveways are, in fact, at-grade intersection's and should be designed consistent with the intended use. The number of crashes is disproportionately higher at driveways than at other intersections; thus their design and location merit special consideration." *(3)*

In the "Access Management, Location, and Design, Participant Notebook", the potential access management techniques are categorized into six groups *(4)*. These categories are related to traffic operational actions, which serve to minimize the frequency and severity of traffic conflicts. The six categories are:

1) Limit number of conflict points: These techniques directly reduce the frequency of either basic conflicts or encroachment conflicts, or reduce the area of conflict at some or all driveways on the highway by limiting or preventing certain types of maneuvers.
2) Separate conflict areas: These techniques either reduce the number of driveways or directly increase the spacing between driveways and intersections. They indirectly reduce the frequency of conflicts by separating turning vehicles at adjacent access points and by increasing the decision-processing time for the through driver between successive conflicts with driveway vehicles at successive driveways.
3) Remove turning vehicles from through traffic lanes: These techniques directly reduce both the frequency and severity of conflicts by providing separate paths and storage areas for turning vehicles.
4) Reduce the number of turning movements: The provision of cross-circulation between adjacent properties and the provision of service roads allows inter-site movement without reentry to the abutting major roadway. The elimination of short distance slow movements reduces the number of conflicts along the major roadway.
5) Improve driveway operations: These techniques allow drivers to maneuver from and to the major roadway more efficiently and safely.
6) Improve roadway operations: These techniques are primarily of a policy nature, which are intended to preserve the functional integrity of the roadway. Different standards are commonly applied depending on the category of the road

In general, the benefits of access management measurements can be summarized as: improved safety, improved traffic flow and fuel economy, increased capacity, and reduced delay and vehicle emissions. Improved safety is one of the most important benefits of proper access management. The safety benefits of access management techniques have been attributed to reduction in traffic conflict points, improved access design, and larger driver response time to potential conflicts.

Various research efforts have evaluated the impacts of access management on roadway safety. The "Access Management, Location and Design, Participant Notebook" suggests that effective access management can reduce crashes by as much as 50%, increase capacity by 23-45%, and reduce travel time and delay as much as 40-60% *(4)*. In a study of the statistical relationship between vehicular crashes and highway access, conducted for the Minnesota Department of Transportation, the results from two approaches, a comparison of crash rates using a random sample of roadways from the State's highway system and a before-and-after comparison of crashes, suggested a strong and statistically sound relationship between level of access and crash rates *(5)*. It showed that crash rates reduced with improvements to median opening spacing in both rural and urban roadway categories. Bonneson and McCoy concluded that crash rates on facilities with non-traversable medians are lower than that of facilities with continuous two-way left-turn lanes (TWLTL) *(6)*.

These studies provide important information on various access management methods and techniques. However, questions still remain surrounding the effects of specific access management treatments on roadway safety and operations. Some of these concerns relate to the safety impacts of U-turn movements at median openings, the effect of medians on intersection capacity, the safety impacts of continuous right-turn lanes, and the effect of medians on side street operations. Other questions relate to median and driveway design practices such as right-in right-out only designs, and appropriate driveway channelization measures. Some of these questions remain unexplored either because quantification of some treatments is difficult or because not enough data are available for the evaluation of alternative treatments. Therefore, more research is needed for evaluate the traffic operational and safety impacts of these techniques.

1.2 Outline of the Report

This report on the operational evaluation of direct left turns versus U-turns consists of six chapters. Chapter 1 provides an overview of the research project including a brief summary of the past studies in this subject area. Chapter 2 describes the methodology used to develop travel time and delay models and analyze weaving on at-grade roadways.

It also describes the methodology used in developing the simulation models using CORSIM. The procedure used to conduct field experiments and data reduction is given in Chapter 3. Chapter 4 presents the results of the operational effects of U-turns as alternatives to direct left turns using the modeling approach. Chapter 5 presents the results of a before-and-after analysis of replacing a full median opening with a directional median opening. Chapter 6 includes the simulation results conducted using CORSIM. Chapter 7 presents summary, conclusions, and recommendations regarding the evaluation of direct left turns versus U-turns.

1.3 Selection of the Study Subject

With the intention of identifying the technique that most needed evaluation, a number of previous studies regarding access management techniques were reviewed, including but not limited to Transportation Research Board (TRB) publications, proceedings of the TRB National Access Management Conferences, reports from the National Cooperative Highway Research Program (NCHRP), publications by AASHTO, Institute of Transportation Engineers (ITE) recommended practices, and the ASCE Journal of Transportation Engineering. In addition, current rules, regulations, standards, and practices in Florida were reviewed.

Based on the literature review, the project team's experiences, and FDOT review, the subject selected for analysis was the right turn followed by U-turn as an alternative to a direct left turn from a driveway or side street. The main reasons for selecting this subject were:

1) Little documentation of quantified results and conclusions regarding this subject are available although the impact of U-turns on safety and operations has been identified as one of the important issues in access management.
2) It is feasible to quantify the safety and operational impacts of these alternatives. Both crash data and potential sites for case studies are available.
3) The results of the traffic operational and safety analysis can assist agencies like FDOT with decisions relative to installing medians or closing median openings.

1.4 The Selected Research Subject

There has been little documentation of the operational effects of providing right turns followed by U-turns at downstream median openings as an alternative to direct left-turns from driveways. When a full median opening is replaced with a directional median opening that only allows left-turn ingress to abutting developments, the left-turn egress movements would be made by turning right onto the arterial road and then making U-turns downstream. Florida Department of Transportation (FDOT) prohibits left-turn exits onto major arterials in many areas, instead providing mid-block U-turn lanes to accommodate these movements. The prohibition of direct left-turns from existing driveways may transfer the displaced left-turns to the nearest traffic-signal-controlled intersection, unless intermediate U-turn lanes are provided.

Recently, many states and local transportation agencies have considered installing restrictive medians on multilane highways. However, the operational effects of installing restrictive median opening have not been clear. Therefore, it is necessary to study the operational effects associated with diverting left-turns from driveways. For this reason and due to the lack of available information on the operational impacts of restrictive medians, the subject of U-turns as alternatives to direct left-turns was selected for comprehensive study.

1.5 Problem Statement

Although access management is expected to enhance roadway traffic operations and safety, district transportation engineers currently rely on broad or subjective methods to assess the effects of various access management treatments. There is no procedure available to quantify effects and evaluate the use of right turns followed by U-turns as alternatives to direct left turns from driveways. A quantitative methodology is needed to evaluate access management treatments so that design standards and policy requirements can be met and potential changes in traffic operational performance can be assessed. Such a methodology can be used to determine appropriate access management practices and treatments and will also be used in documenting operational benefits for the public.

After considering several widely used access management techniques, the subject of "U-turns as alternatives to direct left-turns" was chosen for the detailed analysis of safety and operational impacts. Florida prohibits direct left-turn exits onto major arterials in many locations through the use of non-traversable medians, and provides mid-block median openings in advance of intersections in some areas to accommodate U-turn movements. A right-turn plus U-turn movement as an alternative to a direct left-turn movement has the potential to reduce traffic conflict points and improve traffic operations at unsignalized intersections. However, few field data are available to substantiate this assumption. In addition, people often oppose being forced to make a right-turn followed by a U-turn due to the perception that it results in a longer travel time than a direct left-turn or a belief that U-turns are unsafe. Hence, it is necessary to further evaluate the operational effects of these two movements, especially to compare delay, travel time, and speed reduction of through-traffic in the weaving area.

This report describes a quantitative methodology for evaluating the operational effects of U-turns as alternatives to direct left-turns from driveways. A field experiment was set up to collect data at 10 sites in the Tampa and Clearwater areas. Delay, travel time, speed reduction of through-traffic and percentage of drivers choosing a right turn followed by a U-turn rather than a direct left-turn were used to quantify the operational effects of U-turns as alternatives to direct left-turns from driveways. The research results can be directly applied to evaluate the operational effects of median treatments such as installing a restrictive median, replacing a full median opening with a directional median opening, and a median closure.

Several documents support the necessity of this study. NCHRP Report 395 mentioned that research is needed to determine the true effects of median closures on traffic flow patterns and road user costs *(7)*. To be useful, this research would need to identify median closure effects on the following: (1) U-turn volume at downstream intersections and median openings; (2) Right-turn volume at the subject access point; and (3) The types and frequency of use of routes taken by displaced left-turn drivers and the travel time associated with using these routes. It was noted that this research also should address the impact of displaced left-turn drivers on the delay of existing drivers at downstream intersections. NCHRP 420 also revealed several research needs, including assessment of the effects of median closures, both signalized and unsignalized, and their upstream and downstream effects *(8).*

1.6 Research Objectives

The primary objective of this research was to conduct a comprehensive evaluation of the operational effects of U-turns as alternatives to direct left-turns from driveways on urban and suburban arterials. The study consisted of both operational and safety analyses. Operational effects relate primarily to the delay and travel time of two movements: direct left-turn (DLT) vs. right-turn plus U-turns (RTUT) and speed reductions of the major road through-traffic stream. Only U-turns at a median opening were considered in this study because U-turns delays at signalized intersections were highly related to signal timing. Operational effects of the selected issue conducted using both empirical modeling and computer simulation are described in this report, whereas two separate reports address safety impacts using crash data analyses and conflict analyses.

More specifically, the objectives of this part of the research were:

(1) To determine volume conditions (major-road, left-turn-in, and driveway) under which DLT would have more delay or travel time as compared to RTUT,
(2) To estimate delays for DLT and RTUT as a function of conflicting major and minor-road flow rates,
(3) To estimate the speed reduction of major road through traffic in the weaving section due to vehicles making RTUT,
(4) To estimate the speed reduction of major road through traffic by left turn egress movements,
(5) To determine under what volume conditions (major road, left-turn-in, and

driveway) drivers would enter the highway from a driveway using RTUT instead of DLT, and

(6) To supplement the conventional modeling approach through simulation modeling conducted using CORSIM and to compare the results.

1.7 Past Studies

1.7.1 Impacts of Access Management Techniques

Access management, as a relatively new approach to solve congestion and safety problems, has been widely used in Florida and nationally. There have been four national access management conferences (USDOT/FHS, 1993, 1996, 1998, and 2000) since 1993. Recently, several NCHRP projects were established to conduct comprehensive research in this specific area *(2, 7, 8, 9)*. Over 100 access management techniques were identified and divided into four broad categories: traffic operations, traffic safety, environment, and economic (including transportation service and land use).

In the past decade, there have been many studies on operational effects of access management techniques. The general methodologies used include: case study in the form of a before-and-after analysis, *(10)*, field experiment *(11)*, and computer simulation *(12)*. The basic Measures of Effectiveness (MOEs) often used to quantify the operational effects of access management techniques consist of travel time, delay, capacity, and speed. Operational effects of several selected access management techniques in NCHRP 420 Report are briefly summarized here as follows *(8)*:

(1) Traffic Signal Spacing: Each traffic signal per mile added to a roadway reduces speed about 2 to 3 mph. Travel time on a segment with four signals per mile would be about 16 percent greater than on a segment with two signals per mile;

(2) Curb-Lane Effects: Detailed analyses were conducted to estimate curb-lane effects on through traffic due to vehicles turning right into driveways in this report. The percentage of though traffic in the right lane that would be

affected by vehicles turning right into driveways was used to quantify the operational effects. It was found that the percentage of through vehicles affected at a single driveway increases as right-turn volumes increase;

(3) Unsignalized Access Spacing: Speeds are estimated to be reduced by 0.25 mph for every access point up to 10-mph for 40 access points per mile;

(4) Right-Turn Lanes: Installing a right-turn deceleration lane is an effective method to reduce the impact on through traffic. The percentage of through vehicles affected was about 1.8 times the right-turn-in volume when it ranges from 250 to 800 vph; and

(5) U-turns as Alternatives to Direct Left-turns: An analytical model was developed and calibrated to estimate the travel time savings (or losses) in the suburban and rural environment where there are no nearby traffic lights. Primary findings indicated that two stage left-turning vehicles would suffer longer delays than right-turning plus U-turning vehicles when the volumes on the major street are relatively high (i.e., more than 2,000 vph) and the left-turns exceed 50 vph. This finding holds true even in cases where the right-turn plus U-turn movement involves one-half mile of travel to the U-turn median opening.

A few studies have analyzed capacity gains and delay reductions associated with providing U-turns at median openings as an alternative to direct left-turns at signalized intersections. Past studies found that the directional U-turn design gained about 14 to 18 percent more capacity than the conventional dual left-turn lane design and capacity gains of 20 to 50 percent as a result of prohibiting left-turns at intersections and providing two-phase signal operations *(2, 13).*

Little documentation is available on operational effects of providing U-turns at median openings as an alternative to direct left-turns from a driveway. Stover analyzed the operational issues associated with these two movements and established a procedure to calculate the delay in relation to upstream and downstream signal impacts using queuing analysis *(14)*. A case study by Long and Helms showed that limiting access at unsignalized intersections can reduce turning volumes, increase arterial operating speeds, and improve safety *(15)*. A study by Al-Masaeid developed an empirical model to estimate the capacity and average total delay of U-turns at median openings in Jordan *(11)*. There are some studies about travel timesavings of non-conventional left-turn alternatives by computer simulation *(12, 16, 17)*.

Before-and-after data relative to traffic operations are unavailable regarding the effect of changes in median type, spacing of median openings, or the design of median openings. There have been two studies relative to median modification conducted for the FDOT in the past. One is a comparison of two arterials in Fort Lauderdale having similar traffic operational characteristics - Sunrise Boulevard without median modifications and Oakland Park Boulevard with median modifications. This study provided some insight into the benefits of increased median opening spacing and design *(15)*. An operational analysis of median openings using TRAF-NETSIM was prepared for the FDOT by Transportation Engineering, Inc. in September 1995. This report analyzes the impact of median treatments on traffic operations and air pollution emissions for several arterial corridors in Florida. These studies revealed that, as medians are made more restrictive (e.g. fewer median openings spaced farther apart) then travel speeds increase and fuel consumption, emissions, and delay decrease. The study also noted that travel time may increase for some drivers who have to make U-turns or take longer routes.

In terms of traffic movements, direct left-turn movements must be substituted by RTUT after replacing a full median opening with a directional median opening. A before-and-after case study by Sebastian concluded that closing median openings to prohibit left turns, separating conflicting turn movements, and providing deceleration areas for turning motorists outside the through lane are effective measures in reducing crashes and improving operation of the state highway and access to business properties along the corridor *(10)*. It was also found that the median change did not adversely affect travel speed in this area *(10)*.

In the past decade, there have been many studies regarding median treatment selection *(6,* *18, 19, 20, 21),* the comparison of different types of medians *(22),* the impacts of median width *(9),* and median handbooks and guidelines *(23, 24).* However, most studies have focused on operational and safety effects of three common median treatments: raised-curb median, the flush median with two-way left-turn lane (TWLTL), and the undivided cross section. Past studies have not addressed the situation at the specific unsignalized intersection or median opening, where U-turns occur.

1.7.2 Delay Model at Unsignalized Intersections

Delay and travel time are very important MOEs to evaluate operational effects of DLT vs. RTUT because many drivers often oppose making a RTUT due to the perception that it results a longer travel time and delay. But the actual delay and travel time of these two alternatives are not clear.

There have been numerous studies on developing capacity and delay models to evaluate traffic operations at unsignalized intersections. One study developed a delay-flow rate relationship for undivided and divided 4-lane highways *(25).* In this study, delay was defined as seconds per vehicle for major and minor roads. The flow rate is the combination of major-minor flow rate. A linear fitting was tried between delay per vehicle in seconds and flow rates on major highways. It was found that the slope of the fitted line for the undivided highway case was much higher than that for the divided highway case. This result was as expected because the highway median permits drivers to perform their crossing maneuver in two steps and consequently, they experience less delay. Moreover, delay for the undivided highway was found to be less than the delay for divided highways as long as the major flow rates were less than 290 and 315 vph for minor rates of 100 and 50 vph, respectively.

The Highway Capacity Manual has set up a procedure to estimate the delay, capacity, and level of service of unsignalized intersections *(26).* A study by Tian, Kyte and Colyar indicated that using the HCM procedure could overestimate delay and underestimate capacity when a minor street left-turn vehicle would cross the nearest approach and stop in the median position while waiting to join the major street traffic, resulting in a two-stage gap acceptance process *(27).* The two-stage priority situation as it exists at many unsignalized intersections within multilane major streets provides larger capacities and smaller delay compared to intersections without central storage areas *(28).* A study by Robinson presented theoretical models to adjust the basic capacity or delay equations to account for some common occurrences at TWSC intersections: two-stage gap acceptance, flared minor-street approaches, effects of upstream signals, and effects of pedestrians *(29).* However, these theoretical models have not been calibrated against empirical data.

The new HCM 2000 provided updated models to calculate the capacity and delay of unsignalized intersections, including two-way stop-controlled (TWSC) and all-way stop-controlled (AWSC) *(30).* The procedures for TWSC intersections also account for certain conditions such as effects of upstream signals and of median storage where minor street vehicles can proceed through the intersection in a two-stop process, namely a two-stage gap acceptance process. However, as stipulated in the HCM 2000 methodology, each major-street approach can have up to two through lanes and one exclusive right and/or left-turn lane. Each minor-street approach can have up to three lanes, a maximum of one lane for each movement. This is a limitation of the research on which the procedures are based.

As discussed in the research scope, only major arterials with 6 to 8 through lanes (3 or 4 each direction) were investigated for delay and travel time comparison in this study. Therefore, the HCM procedure for unsignalized intersections could not be directly applied to estimate the delay or travel time of right-turns and left-turns at driveways.

Additional analysis is needed to estimate the delay and travel time of two movements: a RTUT and a DLT. In this study, travel time of the RTUT maneuver is a function of artery traffic volumes, driveway volume, and separation distances between driveway exits and the U-turn channel. Delay of these two movements is also a function of major-road and driveway volumes. Empirical equations will be developed to estimate the delay and travel time of DLT and RTUT along six or eight-lane highways with raised curb median design.

1.7.3 Weaving Issues on At-grade Arterial Streets

Currently, there is no exact procedure to analyze weaving problems on at-grade arterial roadways. However, the HCM 1996 presents a methodology for prediction of weaving speed and non-weaving speed in freeway weaving sections. The procedure is sometimes applied to at-grade arterials, although it has been recognized that weaving speed and non-weaving speed are not the best measures of traffic operations of at-grade weaving sections.

Alexiadis developed a model to predict weaving speed and non-weaving speed in the weaving sections on airport roadways *(31).* A new independent variable, the cross-weaving ratio, was defined as the ratio of the ramp-to-ramp weaving volume to the total volume in the weaving area.

Another recent research by Texas Transportation Institute investigated weaving on frontage roads *(32).* This study developed the guideline of minimum weaving distance based on results of the safety and operations studies and desirable weaving distance based on a combination of the distance of weaving requirements and the level of service on the frontage road.

Research by Fazio developed a multiple linear regression model to predict the average running speed by lane in the weaving section using the equivalent peak passenger car flow rate, length, and number of lane within the weaving section as independent variables *(33).*

1.7.4 Operational Effects of U-turns

Past studies attributing operations and safety gains to a nontraversable median have not focused on the specific situation at the median opening, either isolated or at an intersection, where U-turns occur. Additional information would be helpful in reviewing these requests, determining if an opening should be allowed, and developing a design that does not unduly impact the safety or operations of the roadway. There are wide varieties of designs for nontraversable medians. A comprehensive study of the safety and operational impacts of the various median treatments would be beneficial, both in setting design policy and in project-level design.

One current National Cooperative Highway Research Program (Active Project 17-21) is relevant to the subject. This is NCHRP project 17-21, "Safety of U-Turns at Unsignalized Median Openings." Objectives of this research are to document the safety impacts of U-turns at unsignalized median openings and to develop a guide for the use, location, and design of unsignalized median openings for U-turns.

There are few studies on the operational effects of U-turns. The HCM 1996, which contains procedures and models for estimating capacity and delay for different movements at unsignalized intersections, does not provide specific guidelines for estimating capacity and delay of U-turn movements at median openings. Traffic operations at directional median openings have not yet been formally addressed in the United States. A study by Al-Masaeid developed regression equations to estimate the delay and capacity of U-turns by field experiment as follows *(11)*:

$$TD = 6.6 \times e^{q_c / 1200} \tag{1-1}$$

Where TD represents the average total delay for the turning vehicles (s/veh), q_c represents the conflicting traffic flow (PCU/h). The conflicting traffic flow rate was converted into passenger car units (PCU) after accounting for heavy vehicles.

$$C = 1545 - 790e^{q_c / 3600} \tag{1-2}$$

Where C represents the capacity of U-turn movement (PCU/h), and q_c represents the conflicting traffic flow (PCU/h).

The above two empirical formulas indicate that there are strong correlations between delay, capacity, and total conflicting traffic flow.

1.7.5 CORSIM-Based Simulation Models

Recently, simulation technology has extensively been tested and used in access management, even though there exists skepticism about its accountability and accuracy. As indicated in the study by Vargas and Reddy, CORSIM was found to be capable of simulating and estimating the impact of access management improvements on traffic flow with reasonable accuracy *(34).* Results of this study suggested that strong consideration be given to providing adequate U-turn opportunities prior to the signalized intersection. Otherwise, signalized intersections would degrade further and may limit the capacity of the arterial.

Regarding the speed reduction on the arterial due to the egress from driveways, McShane et al. used CORSIM to simulate the effects of right-turn-only strategy from driveways on the average travel speed of major road traffic *(12)*. Compared to right turn and direct left turn, right-turn-only had a very mild effect to the near side upstream traffic. The results showed that DLT would not greatly affect the speed of upstream traffic, but would severely influence the speed of downstream traffic.

Wong has indicated in his study that CORSIM simulation output might vary considerably depending on the simulation time and the random number seeds (*35)*. The study showed that the variation of the outputs was higher when the simulation time was short. As the simulation time became longer, the variation among different random number seeds became less and the value within the same random number seed became stable. More specifically, when the simulation time was at or above 3600 seconds, the output values appeared to stabilize.

Benekohal and Abu-Lebdeh have proposed two different methods to analyze the variability of CORSIM output (*36)*. One method is performed by running the simulation model for one long run and then dividing it into smaller time intervals called batches. For each batch, statistics are collected and variability among batches is used to build a confidence interval on the simulation output. If the batches are long enough, the means from the batches may be uncorrelated. Increasing the length of the batches may reduce their autocorrelation. The other method is the replication approach, which is performed by running the simulation for a number of independent runs. The independent simulation runs are made for the same roadway and traffic conditions. Each run will have initialization time until the system reaches equilibrium condition. After the warm-up time, statistics on system performance are collected. As indicated from this study, misleading and erroneous conclusions may be obtained if the variability in CORSIM output is not seriously considered.

Even though CORSIM has some implied validity due to the successful applications in traffic operation analysis for many years, its models are still far from satisfactory to reflect the real situation. As pointed out by Prevedouros and Wang, the default parameters embedded in CORSIM offer no satisfactory results *(37)*. All the simulation models can be applied only after completing the process of validation and calibration. In general, there are two approaches to the calibration of the microscopic traffic simulation systems *(38)*. The first one is model calibration, which re-establishes the input-output relation to obtain the desired system accuracy by changing the basic modules that describe the complex relationship between the input and output of the simulation systems. Actually, only model developers are in the position for adopting such methodology because they have control and accessibility over the internal resources. The other approach is parameter calibration, which is regarded as the optimization problem in which sets of values for operating parameters that satisfy the objective function are to be searched. In this study, we conduct parameter calibration to the CORSIM-based models in order to have simulation models well replicate the real traffic situations.

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2. METHODOLOGY

The methodologies used to achieve the objectives of this study are explained in this chapter, which consists of three sections. The first section explains the data sources for the modeling and computer simulation. The second section explains the methodology used in developing delay and travel time models, including the speed analysis method used to analyze impacts on major-road through-traffic speed and weaving issues on at-grade weaving segments. The third section deals with the methodology related to the computer simulation.

2.1 Data Sources

In this study, field experiments were set up to collect data at appropriate sites in the Tampa Bay area. Several data parameters were required to quantify the operational effects of right turns followed by U-turns as alternatives to direct left-turns from driveways, using modeling and simulation approaches. More specifically, the data needed to develop the delay and travel time models for two movements include:

(1) Traffic volume: major-road through-traffic volume, left-turn-in volume from major-road, driveway volume and U-turn volume,

(2) Traffic delay: delay of left turns and right turns at the subject driveway, delay of left turns and U-turns at median openings,

(3) Traffic running time: average running time of RTUT crossing the weaving segment, and average running time of DLT crossing the through lanes,

(4) Geometric data: cross section, lane assignments, weaving distance, and median type, and

(5) Traffic control features: speed limit, traffic control signs and traffic signals.

2.1.1 Site Selection

A study site for this study was defined as an urban or suburban arterial street segment that has only two or more unsignalized access points along its length. The segment had a uniform cross section and raised curb median. Geometric criteria of specific study sites are as follows:

(1) The arterial should have a raised-curb median with either a full median opening or a directional median opening that can safely store waiting vehicles,

(2) The arterial should have 6 or 8 through traffic lanes (3 or 4 lanes each direction). Passenger cars can normally make U-turns along a divided six-lane arterial. However, as requested by FDOT, two 4-lane arterials were chosen to conduct the field study because there have been many restrictive median openings installed along four-lane highways with a raised median,

(3) Speed limit on the arterial should be 40 mph or higher. The FDOT mandates that all new multi-lane projects with design speeds of 40 mph or greater be designed with a restrictive median,

(4) The studied driveway should have either two lanes (one for right-turn and another for the left-turn) or one wide lane with a flared curb so that the two movements do not interfere with each other,

(5) The driveway volumes should be high so that there were a considerable number of RTUT and/or DLT vehicles, and

(6) The median width should be wide enough to store the left-turn vehicles.

The street segments selected for final comprehensive data collection are listed in Table 2.1, together with the information at each site including location, number of through lanes, weaving distance, median type, the distance from driveway to upstream and downstream signals, and upstream and downstream signal timings. At site 3, field data were collected one week before and after the full median opening was replaced with a directional median opening. The signal timing was only recorded at some sites with a pre-timed signal timing plan at upstream and downstream-signalized intersections. Sites 7 and 10 have no signal timing data because the signals had actuated signal timings. The offset was computed as the difference between the starting time of the red light for major-road through-traffic at upstream and downstream signalized intersections. Some sites do not have this value because the upstream and downstream signals are uncoordinated.

Table 2.1 Description of Field Sites

SITE	1	2	3	4	5	6	7	8	9	10
Arterial	Fowler Ave.	Fowler Ave.	US 19	B. B. Downs	Hillsborough	US 19	US 19	Fowler Ave.	Gunn	B. B. Downs
Location	46^{th} St.	19^{th} St.	115^{th} St.	Medical Center	Golden	Enterp-rise Center	Innisb-rook	52^{th} St.	Hang-ert	Pebble Creek
N*	6	8	6	6	6	6	6	6	4	4
MT*	D*	F*	D/F	F	F	F	F	D	F	D
Speed	45	50	55	45	45	55	55	50	45	45
WD*	800	570	420	970	300	550	600	580	590	850
UGT*	108	100	95	70	100	150	NA*	85	48	50
URT*	17	70	25	40	20	30	NA	65	46	65
UCL*	125	170	120	110	120	180	NA	150	94	145
DGT*	105	90	90	113	90	87	NA	80	58	NA
DRT*	20	80	30	55	30	93	NA	70	21	NA
DCL*	125	170	120	168	120	180	NA	150	79	NA
OUDS*	20	20	20	NA	40	NA	NA	NA	NA	NA
DU*	950	700	600	870	850	1700	5280	1200	2120	1000
DD*	700	1350	1620	1160	750	4750	5808	530	2238	850

* Note: N: # of through lane; MT: Median Type; WD: Weaving distance (ft.); UGT: Upstream signal green time (sec.); URT: Upstream signal red time; UCL: Upstream signal cycle length; DGT: Downstream signal green time; DRT: Downstream signal red time; DCL: Downstream signal cycle length; OUDS: Offset of upstream and downstream signal; DU: Distance from driveway to upstream signal; DD: Distance from U-turn median to downstream signal (ft.), D: Directional median opening, F: Full median opening, and NA: Not applicable/available.

In selecting the appropriate sites, the most challenging criterion to satisfy was the high RTUT and DLT volumes. It was difficult to find the sites with a fairly high RTUT when left turn egress from the subject driveway was permitted. Therefore, some sites with directional median openings were also selected for field data collection. Nevertheless, some sites with relatively high RTUT and DLT volumes were found, and several of these had a high percentage of RTUT yielding desired RTUT ratios.

Street segments with four through lanes were not used for operational analysis conducted using modeling and simulation approaches, because the U-turns can only be conveniently made at arterials with six or more lanes. Data from the sites of four lane arterials were only used to perform a comparison of lane utilizations to verify its effect.

2.1.2 Data Collection

Equipment used to collect field data included four video cameras and two automatic traffic recorders. In all cases, video cameras were used to monitor traffic operations in and around the two median openings and also in the weaving section. Because the installation of video cameras at the ground level was incapable of providing sufficient viewing heights to record all the movements, alternative methods were identified. After careful evaluation, scaffoldings were selected as the best alternative in the absence of appropriate buildings for setting up the video cameras. Accordingly, Figures 2.1 and 2.2 show the cameras set up at the top of 15 feet high, two-story scaffolding and another camera was set up at the top of a building.

A typical field setup is shown in Figure 2.3. The field studies were conducted during March 2000 to December 2000. Data were collected for two weeks at each site for at least four hours a day, including both peak and non-peak hours. A total of more than three hundred-hours of traffic data was recorded by video cameras at the ten sites. All data were collected during weekday, daytime periods between 7:00 a.m. and 7:00 p.m. Data were not collected during inclement weather or during unusual traffic conditions such as traffic crashes or construction.

Figure 2.1   Setting-up of Cameras in the Field-1

Figure 2.2   Setting-up of Cameras in the Field-2

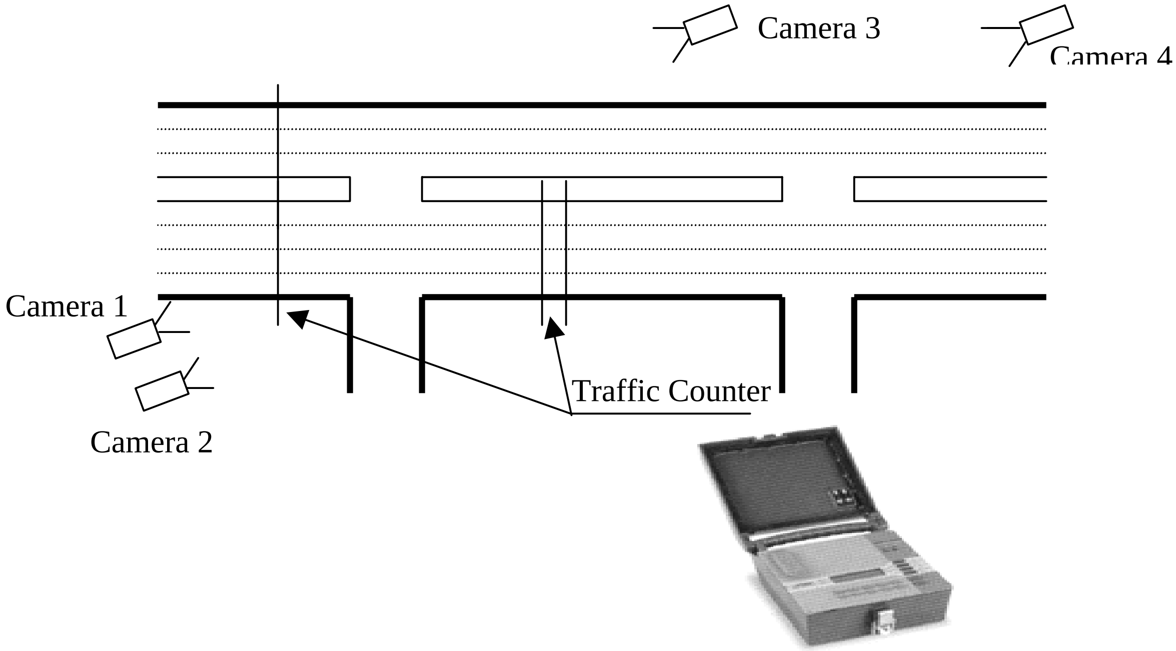


Figure 2.3 Typical Field Data Collection Setup

Delay and travel time data were obtained from the video cameras. Delay of right-turns and left-turns was recorded by camera 2 (Figure 2.3). Waiting delays of left-turns at the median opening and U-turns at the downstream median opening were recorded by cameras 1 and 4 (Figure 4.3). Left-turn-in and driveway volumes were extracted from the videotapes while reviewing the tapes for delay data. Average running time of RTUT in the weaving section and average running time of DLT crossing the through lanes were also recorded by camera one. All the cameras were synchronized so that data extracted from different videotapes could be matched.

Major-road through-traffic volume and speed data were recorded at five-minute intervals by two automatic traffic recorders (Peek ADR-1000). One ADR was installed to record upstream and downstream through-traffic volumes, separately. Another one was used to collect the speed of through-traffic. To measure speed reduction due to RTUT, an ADR was installed in the middle of the weaving section to collect the spot speed of through-traffic at five-minute intervals. The ADR was also installed at one hundred feet upstream of the studied unsignalized intersection to measure the effects of DLT vehicles on through-traffic because it was observed that drivers usually decelerated in this area to avoid collision with left-turn vehicles.

Installing a road tube across the street was a difficult task when through-traffic volume and speed were high. Usually, the road tube was installed during non-peak hour or weekends. A good road tube kit was found to be very important for proper field data collection. After the field test at the first two sites, it was found that road tube grips were not a good tool to fix the tube to the road surface because people usually stay in the through lanes for a long time. In addition, the grips easily loosen and crash the tube, thereby preventing the air from flowing freely. Finally, the mastic tape made by JAMAR was adopted to fix the tube to the road surface. This tape requires less time for data collectors to stay in the traffic, thus reducing the potential safety problems.

2.1.3 Data Reduction

While reducing data, each vehicle coming from the driveway and making RTUT or DLT was tracked. Four cameras and two traffic recorders were synchronized so that time reference data from each of them could be matched. While reviewing the videotapes, the following information was recorded:

(1) Waiting delay of left-turn and right-turn vehicles at the driveway (defined as t_{L1} and t_{RU1}, respectively);
(2) Waiting delay of DLT vehicles at the full median opening and U-turn vehicles at the U-turn median opening (defined as t_{L2} and t_{RU2}, respectively); and
(3) Running time of DLT vehicles crossing through lanes and RTUT vehicles traversing the weaving segment (defined as t_{L3} and t_{RU3}, respectively).

Total delay for each individual vehicle at a driveway was measured as the time elapsed from the time a vehicle joins a queue until the vehicle leaves the stop line. This includes the service time and queuing time. Service time is the time that a right-turn or left-turn vehicle stays at the stop line, which is affected by the through-traffic volume and its distribution. Queuing time is the time that a vehicle moves from a queuing position to the stop line, which is affected by the right-turn or left-turn volume at a driveway.

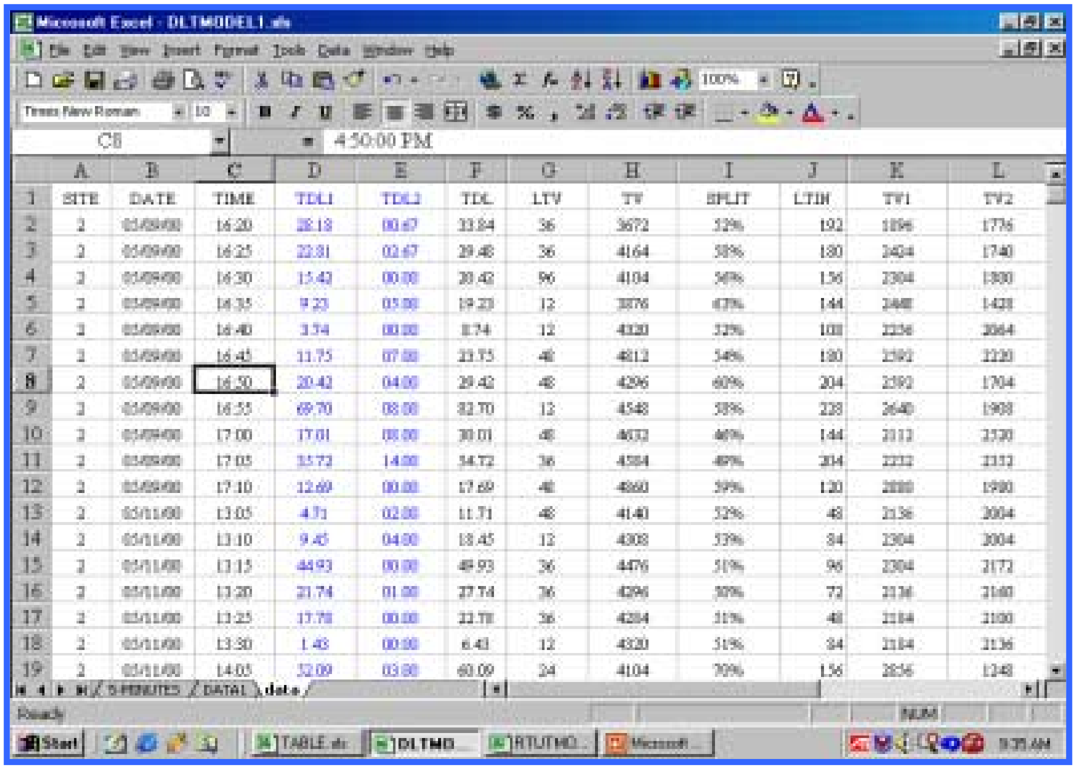
The total delay of right-turn and left-turn at a driveway can be obtained by recording two events: the time a vehicle enters a queue and the time a vehicle exits the stop line. Delay data were extracted by Traffic Data Input Program (TDIP) software, in which the users can identify events on videotapes by pressing computer keys to record event type. For example, “1” is pressed whenever a vehicle joins a queue, “2” is pressed whenever a vehicle exists the driveway and so on.

Waiting delay of left-turns and U-turns at a median opening can be measured by recording the time from when a vehicle stops at the median until it leaves the median. Travel time of RTUT in the weaving segment can be measured through recording the time from when a vehicle leaves the driveway until it stops at the U-turn median opening. The delay data for each vehicle can be summarized to obtain the average delay at five-minute intervals. Traffic volume and speed data were downloaded from the ADR to a computer using the Traffic Data Processing (TDP) software provided by Peek Traffic, Inc. This data can be transferred to a text file and imported to an Excel spread sheet.

2.1.4 Database Summary

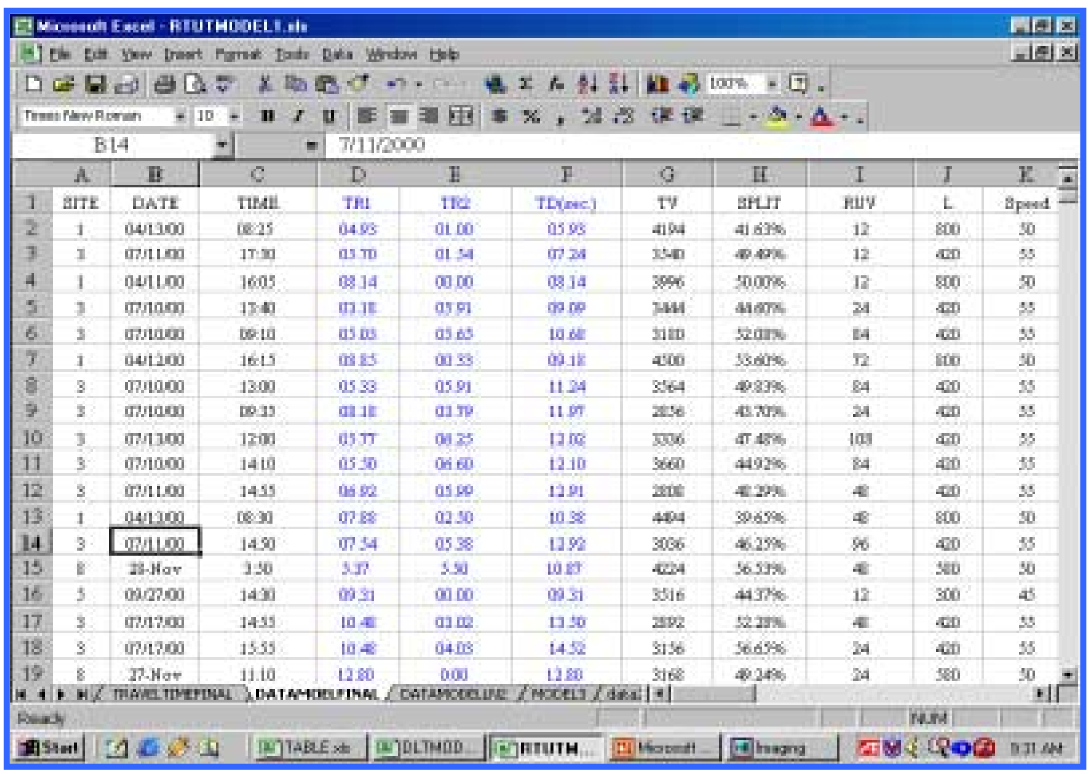
All the delay and travel time data, driveway volumes, through-traffic volumes, and left-turn-in volumes were grouped into five-minute intervals. Finally, a database was set up to perform the statistical analysis for the operational analysis. Figure 2.4 shows the sample database for the delay and travel time of DLT, which includes all the variables for developing the delay and travel time models. It should be noted that only field data collected from site 2 through site 7 are included in the database for the DLT delay model because there is no left-turn egress allowed at sites 1 and 8.

Figure 2.5 shows the sample database for the delay and travel time of RTUT. Field data collected at eight sites were included in the database. However, only the intervals when there is a RTUT were included in the database. It should be noted that there is a very high percentage of RTUT volume at sites 1, 3, and 8 because directional median openings were available at these sites.



	SITE	DATE	TIME	TDL1	TDL2	TDL	LTV	TV	SPLIT	LTIH	TV1	TV2
2	2	05/09/00	16:20	28.13	00.67	33.84	36	3672	32%	192	1896	1776
3	2	05/09/00	16:25	22.81	02.67	29.48	36	4164	58%	180	2424	1740
4	2	05/09/00	16:30	15.42	00.00	20.42	96	4104	56%	156	2304	1800
5	2	05/09/00	16:35	9.23	05.00	19.23	12	3876	63%	144	2448	1428
6	2	05/09/00	16:40	3.74	00.00	8.74	12	4320	52%	108	2256	2064
7	2	05/09/00	16:45	11.75	07.00	23.75	48	4812	54%	180	2592	2220
8	2	05/09/00	16:50	20.42	04.00	29.42	48	4296	60%	204	2592	1704
9	2	05/09/00	16:55	69.70	08.00	82.70	12	4548	58%	228	2640	1908
10	2	05/09/00	17:00	17.01	08.00	30.01	48	4632	46%	144	2112	2520
11	2	05/09/00	17:05	15.72	14.00	34.72	36	4584	49%	204	2232	2352
12	2	05/09/00	17:10	12.69	00.00	17.69	48	4860	59%	120	2880	1980
13	2	05/11/00	13:05	4.71	02.00	11.71	48	4140	52%	48	2136	2004
14	2	05/11/00	13:10	9.45	04.00	18.45	12	4308	53%	84	2304	2004
15	2	05/11/00	13:15	44.93	00.00	49.93	36	4476	51%	96	2304	2172
16	2	05/11/00	13:20	21.74	01.00	27.74	36	4296	50%	72	2136	2160
17	2	05/11/00	13:25	17.78	00.00	22.78	36	4284	51%	48	2184	2100
18	2	05/11/00	13:30	1.43	00.00	6.43	12	4320	51%	84	2184	2136
19	2	05/11/00	14:05	52.09	03.00	60.09	24	4104	70%	156	2856	1248

Figure 2.4 Sample Database for DLT Delay Model



	SITE	DATE	TIME	TR1	TR2	TD(sec.)	TV	SPLIT	RUV	L	Speed
2	1	04/13/00	08:25	04.93	01.00	05.93	4194	41.63%	12	800	50
3	3	07/11/00	17:30	03.70	01.54	07.24	3540	49.49%	12	420	55
4	1	04/11/00	16:05	08.14	00.00	08.14	3996	50.00%	12	800	50
5	3	07/10/00	17:40	03.18	05.91	09.09	3444	44.60%	24	420	55
6	3	07/10/00	09:10	05.03	05.65	10.68	3180	52.08%	84	420	55
7	1	04/12/00	16:15	08.85	00.33	09.18	4500	53.60%	72	800	50
8	3	07/10/00	13:00	05.33	05.91	11.24	3564	49.83%	84	420	55
9	3	07/10/00	09:35	08.18	03.79	11.97	2856	43.70%	24	420	55
10	3	07/13/00	12:00	05.77	06.25	12.02	3336	47.48%	108	420	55
11	3	07/10/00	14:10	05.50	06.60	12.10	3660	44.92%	84	420	55
12	3	07/11/00	14:55	06.92	05.99	12.91	2808	48.29%	48	420	55
13	1	04/13/00	08:30	07.88	02.50	10.38	4404	39.63%	48	800	50
14	3	07/11/00	14:50	07.54	05.38	12.92	3036	46.25%	96	420	55
15	8	28-Nov	3:50	5.37	5.50	10.87	4224	56.53%	48	380	50
16	5	09/27/00	14:30	09.31	00.00	09.31	3516	44.37%	12	300	45
17	3	07/17/00	14:55	10.48	03.02	13.50	2892	52.28%	48	420	55
18	3	07/17/00	15:55	10.48	04.03	14.52	3156	56.63%	24	420	55
19	8	27-Nov	11:10	12.80	0.00	12.80	3168	49.24%	24	380	50

Figure 2.5 Sample Database for RTUT Delay Model

2.2 Delay and Travel Time Models

One of the objectives of this study was to develop empirical equations between the average total delay or travel time and the combination of total conflicting traffic flow and driveway volume. These empirical equations can be used to determine under what volume conditions (major-road, left-turn-in, and driveway volumes) a DLT would experience longer delay or travel time as compared to a RTUT.

2.2.1 Operational Analysis

<u>Direct Left Turns</u>

Based on the definition of the priority of all movements at an unsignalized intersection, DLT egress from a driveway or side street has the lowest priority. Theoretically, DLT egress must therefore yield to all other movements at unsignalized intersections. Thus, it is the most likely movement to be delayed. However, in the real world, when left-turn drivers wait for longer periods, they become more aggressive and enter the median opening without yielding to other maneuvers, such as left-turn-in vehicles from the major road. On the arterials with wide medians, which can allow one or two vehicles to stop, a DLT maneuver may require four steps, as shown in Figure 2.6 and as explained as follows.

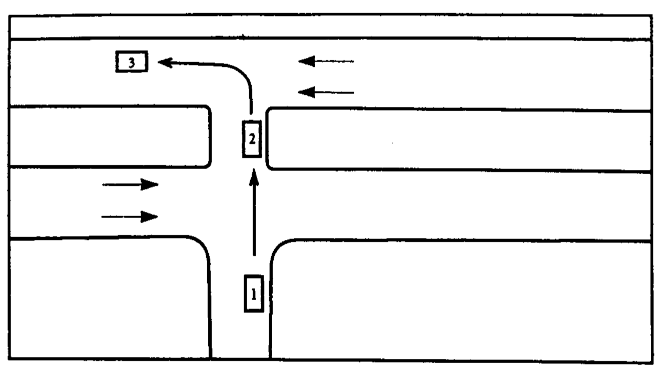


Figure 2.6 DLT Egress Movement (Source: NCHRP 4-20)

Step 1: Stopping and waiting at the driveways,

Step 2: Selecting a suitable gap, accelerating across major-road through-traffic lanes and coming to a stop in the median. Sometimes, drivers can cross the median without stopping at the median openings if there is a suitable gap in both directions,

Step 3: Stopping at the median, and waiting for a suitable gap from right-side through-traffic. Some drivers only need to select a suitable gap for the inside lane, accelerate and merge into through traffic, whereas some others need at least two clear lanes. Sometimes when several left-turn vehicles stop parallel at the median opening, the vehicles stopped at the right side may block visibility for other drivers. This may result in crashes between left-turning vehicles and through traffic; and

Step 4: Accelerating to operating speed on the major roadway. This may force through traffic to decelerate or make a lane change when the left-turning drivers select a small gap.

Based on the operations analysis of a DLT movement, the average delay and total travel time of DLT can be defined by the following equations:

$$TT_L = t_{L1} + t_{L2} + t_{L3} \tag{2-1}$$

$$TD_L = t_{L1} + t_{L2} \tag{2-2}$$

where,

TT_L-average total travel time of DLT movements,

TD_L-average total waiting delay of DLT movements,

t_{L1}- average waiting delay of DLT vehicles at the driveway,

t_{L2}- average waiting delay of DLT vehicles at the median opening, and

t_{L3} - average running time for vehicles leaving the driveway till completing the left turn movement (not including t_{L1} and t_{L2}).

From the above equations, the average total delay of DLT is the sum of average waiting delay of left turns at a driveway and the average waiting delay at a median opening. The average total travel time of DLT is equal to the average total delay plus the average running time for vehicles from the time they leave the driveway to when they stop at the median opening (t_{L3}).

Right Turn Plus U-turns

RTUT can be used as an alternative to DLT in order to eliminate the conflict points associated with the DLT at unsignalized intersections. Under high through-traffic volume conditions, left-turn egress becomes more difficult when there is relatively high left-turn-in volume. In this case, drivers would like to make a right turn followed by a U-turn especially when there is a downstream U-turn median opening within the sight distance. As shown in Figure 2.7, a RTUT maneuver also requires four steps.

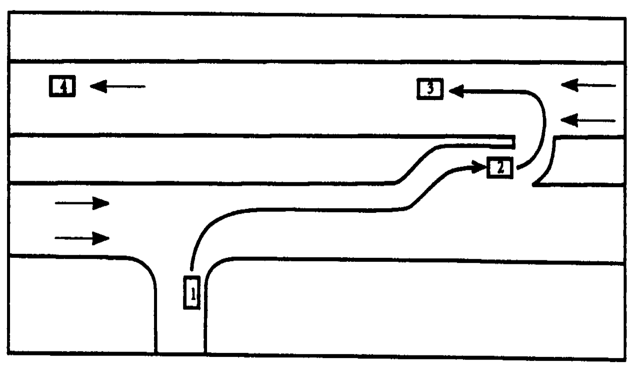


Figure 2.7 A RTUT Movement (Source: NCHRP 4-20)

Step 1: Stopping at the driveway, and making a right turn when there is a suitable gap from left-side through-traffic. This is much easier than left-turn egress because it does not need to yield to other movements at the unsignalized intersection at the same time. So, usually when the upstream signal for the major-road through-traffic turns red, there is a large gap created for right turns. There is a potential conflict between a right turn from a driveway and a U-turn at the median opening. Drivers can easily overlook this conflict, which can result in an accident when their attention is focused on the major-road through traffic;

Step 2: Accelerate, weave to the inside lane, and decelerate to a stop at the U-turn median opening. This movement will cause conflicts such as deceleration and lane change of through traffic. There may also be speed reduction of through traffic in the weaving section;

Step 3: Waiting a suitable gap to make a U-turn. Because U-turns must wait for a gap on the all through-traffic lanes, these may take longer delays than left turn egress vehicles waiting at the median. U-turns at an exclusive U-turn median opening are much easier and safer than at a full median opening. Sometimes drivers are confused about which maneuver should have higher priority because there is no regulation on the priority of U-turns; and

Step 4: Accelerate to the operating speed of through-traffic. This step is similar to a DLT movement.

Accordingly, to estimate total travel time for vehicles making RTUT movements, the following equations can be used:

$$TT_{RU} = t_{RU1} + t_{RU2} + t_{RU3} + t_{RU4} \tag{2-3}$$

$$TD_{RU} = t_{RU1} + t_{RU2} \tag{2-4}$$

$$t_{RU3} = 0.68 \times (l/v_w) \tag{2-5}$$

$$t_{RU4} = 0.68 \times (l/v_T) \tag{2-6}$$

where,

TT_{RU}- average total travel time of RTUT movements (seconds),

TD_{RU}-average total waiting delay of RTUT movements (seconds),

t_{RU1}- average waiting delay of right-turn vehicles at the driveway (seconds),

t_{RU2}- average waiting delay of U-turn vehicles at the U-turn median opening (seconds),

t_{RU3}- average running time from leaving the driveway to stopping at the U-turn median opening(not including t_{R1} and t_{R2}) (seconds),

t_{RU4}- average running time of vehicles crossing the weaving distance at the posted speed of through-traffic (seconds),

l - weaving distance from the studied driveway to the median U-turn opening(ft.),

v_w - average weaving speed (mph), and

v_T - speed limit on the major arterials(mph).

The average total waiting delay of RTUT vehicles includes the delay of right turns at the subject driveway (t_{RU1}) and the delay of U-turns at a median opening (t_{RU2}). The average total travel time of a RTUT movement is the sum of average total waiting delay, the average running time in the weaving section, and the average running time needed for a vehicle traversing the length of the weaving segment at the operating speed of through-traffic. The average total delay and travel time were used to quantify operational effects of RTUT vs. DLT.

<u>Weaving Issues Related to RTUT</u>

Patterns and Types

The Highway Capacity Manual (1996) provides a procedure to estimate the average weaving and non-weaving speed in the freeway weaving areas. A total of three types of freeway weaving areas were identified in the HCM 1996. The type C (b) weaving area illustrated in Figure 2.8 is the one that closely compares to the weaving maneuver of a right-turn followed by a U-turn.

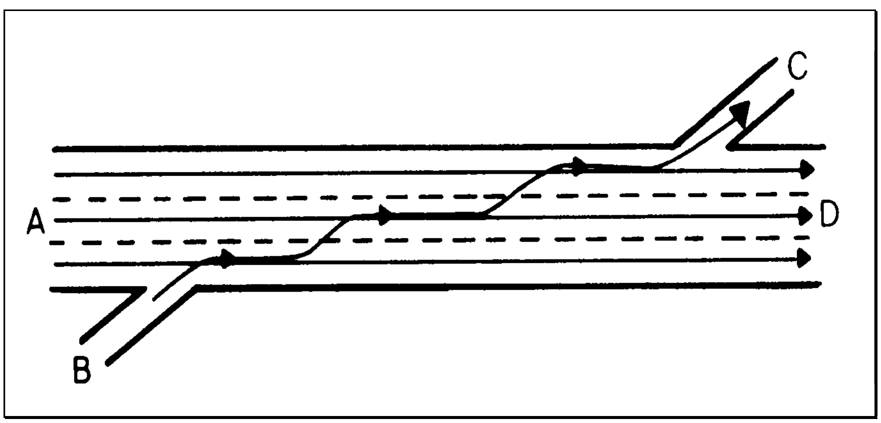


Figure 2.8 Type C (b) Weaving Area in HCM 1994

The major difference is that, in a freeway weaving section, there are acceleration and deceleration ramp lanes so that the weaving vehicles have appropriate entering and exiting speeds, but in the case of at-grade urban and suburban arterial weaving sections, traffic flow is interrupted by upstream signals. Thus, drivers making a RTUT can execute the right-turn in an acceptable gap between the platoons and then decelerate into the median opening. This has no obvious impact on the major-road traffic platoons. Only the random arrivals or stragglers on the major arterial may be impacted by the weaving maneuver of a RTUT in the weaving segment.

Basically, there are three types of weaving patterns of a RTUT as illustrated in Figure 2.9:

(1) When the weaving distance is shorter than the left turn deceleration lane on the major road, many drivers will select a suitable simultaneous gap in all through lanes and then make a direct entry into the left turn deceleration lane,
(2) When the weaving distance is medium, which is not long enough to make a comfortable lane change when executing a RTUT maneuver, many drivers will select a suitable simultaneous gap in all three through lanes and then make a direct entry into the innermost lane, and
(3) When the weaving distance is long, the drivers will select a suitable gap, turn into the right-side lane, accelerate to an appropriate speed, and then make lane changes.

In the field, it was found that many drivers making a RTUT would select the weaving type "B" if they knew the location of the downstream U-turn median opening or when the U-turn opportunity was located within the driver's sight distance. However, some drivers would make a sudden lane change to reach the left turn deceleration lane when they were not familiar with the area and suddenly find the U-turn median opening.

Reduction in Through Traffic Speed (Non-weaving Speed)

For a RTUT maneuver, weaving speed refers to the space mean speed of a RTUT in the weaving section. Non-weaving speed represents the average spot speed of through traffic. The through traffic might experience a reduction of speed due to the increase in number of vehicles making a RTUT.

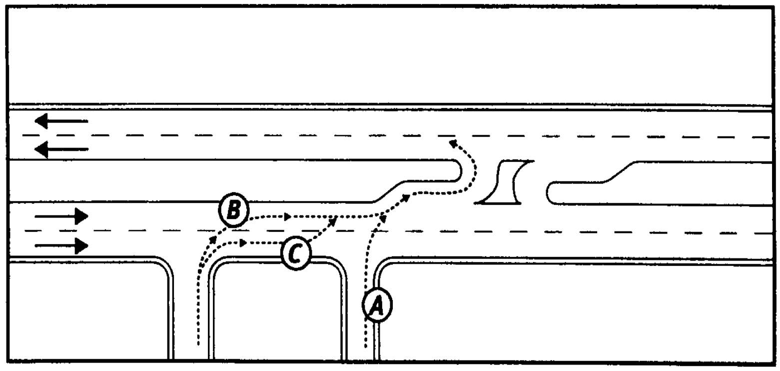


Figure 2.9 Weaving Patterns (Source: NCHRP 4-20)

In this study, an Automatic Traffic Recorder (ADR) was installed in the middle of the weaving section to collect the spot speed of through traffic at five-minute intervals. The ADR was also installed one hundred feet upstream of the studied unsignalized intersection to measure the effects of DLT vehicles on through traffic. The RTUT flow rate, DLT flow rate, and average spot speed of through traffic at every five-minute interval were entered into an Excel worksheet. The relationship between average through speed and the flow rate of DLT and RTUT can be developed to compare the speed effects of these two movements.

Average Weaving Speed of RTUT

The average weaving speed of vehicles making a RTUT will increase as the weaving distance increases. As stipulated in Chapter 24 of HCM 2000: Freeway Weaving, the methodology does not address the problems of weaving segments on collector-distributor roadways and weaving segments on urban streets. Therefore, the algorithm for prediction of average weaving and non-weaving speeds may not be used to estimate average weaving speed of RTUT and the speed reduction of through traffic in the weaving segments.

Additional effort was made to develop an empirical equation to predict weaving speed of a RTUT. Weaving speed is defined as the space mean speed of vehicles making a RTUT. The space mean speed is computed as the length of the weaving segment divided by average running time. A video camera was used to monitor the weaving section to record the space mean speed of vehicles making a RTUT. The travel time of each vehicle making a RTUT was obtained through reviewing the videotapes. In this research, a linear regression analysis was performed to develop the model for prediction of the average weaving speed in the different length weaving segments.

Driver Selection of RTUT

Information on driver selection of roadside business on the basis of accessibility considerations is very useful to help transportation practitioners make decisions on the median treatment. In practice, more drivers may select a RTUT when the average delay of DLT increases. The average delay of DLT is determined by the major-road through-traffic, DLT and left-turn-in flow rates. Therefore, the percentage of RTUT may increase when the left-turn-in and through-traffic flow rates reach a certain value, or when the volume to capacity ratio of DLT increases due to restrained median storage.

2.3 Computer Based Simulation

To supplement the findings of the empirical modeling process, this study utilized computer simulation to compare the operational effects of right turns followed by U-turns with direct left turns. The related methodology is explained here.

2.3.1 Simulation Package

Generally, there are five commonly used simulation packages in the traffic engineering. They are:

(1) CORSIM - a micro-simulation component of the TRAF family of models developed by the Federal Highway Administration (FHWA) for simulation of traffic behavior on integrated urban networks of freeways and surface streets,

(2) INTEGRATION - models aggregate speed-volume interactions of traffic, but can’t model the details of lane-changing and car-following behavior,

(3) WATSim - developed by KLD Associates and based entirely on CORSIM,

(4) PARAMICS - a suite of software tools for microscopic, time-stepping traffic simulation, and

(5) VISSIM - a microscopic, time step and behavior based simulation model analyzing roadways and public transportation operations.

After careful evaluation of these packages in terms of applicability, availability, and usefulness, CORSIM (Corridor Simulation) was selected as the most appropriate simulation tool for the purpose of this study. Reasons for selecting CORSIM include its versatile features, ability to simulate vehicular movements on a street network microscopically, long record as a powerful traffic simulation tool, versatility in choosing parameters for the calibration, attainability, and the availability of animation features that no other package can compete with. It has also been noted by Bloomberg et al. that resources (time, money, or experience) do not always permit the use of multiple models to simulate traffic operation *(39, 40).*

CORSIM uses a fixed-time, discrete-event approach to model the movements of individual vehicles in the network as they travel along the links, crossing the intersections controlled by various devices. It is an interval-scanning model because it computes the state of the system at regular time intervals, specifically every second. Car-following rules, lane changing and overtaking behavior, turning movements, and response to the traffic control system govern the motion of each vehicle. Some of the characteristics of each vehicle are assigned probabilistically using the Monte Carlo approach. The CORSIM-based model can compute a wide range of Measures of Effectiveness (MOEs) as these vehicles interact with one another and respond to the control devices. The users have the option to vary roadway and traffic features including volume, network geometry, turning movements, signal timing, and offsets. The MOE generated by the CORSIM-based model are expected to reflect the effects of the changes in these input variables. Output MOE include travel times, total and stopped delays, running speed, timing data, queue lengths, signal-phase failures, vehicle occupancies, fuel consumption, pollution emissions, and so on. CORSIM also has TRAFVU (TRAF Visualization Utility), which is a state-of-art, object-oriented, user-friendly graphics post-processor for displaying various features.

2.3.2 Direct Left Turn Model

The simulation logic embedded in CORSIM to describe DLT from driveways agrees with the one-step movement, namely vehicles making DLT have to wait until both directions are clear. CORSIM cannot simulate medians and therefore, cannot simulate two-step movements of a DLT. Accordingly, a dummy intersection was used to represent the real situation as shown in Figure 2.10. In this modified model, DLT vehicles search for a gap in upstream traffic only to cross the road and then wait at the stop-controlled intersection (dummy) to seek a gap from downstream traffic. As shown in Figure 1-1, DLT drivers first stop at point ① seeking for gaps of upstream traffic, then cross half the road and stop at position ② looking for gaps of the downstream traffic. They finally reach point ③ to complete their turning movement. The length of the dummy link was determined by the average number of vehicles stored at the median, which can be obtained from field studies.

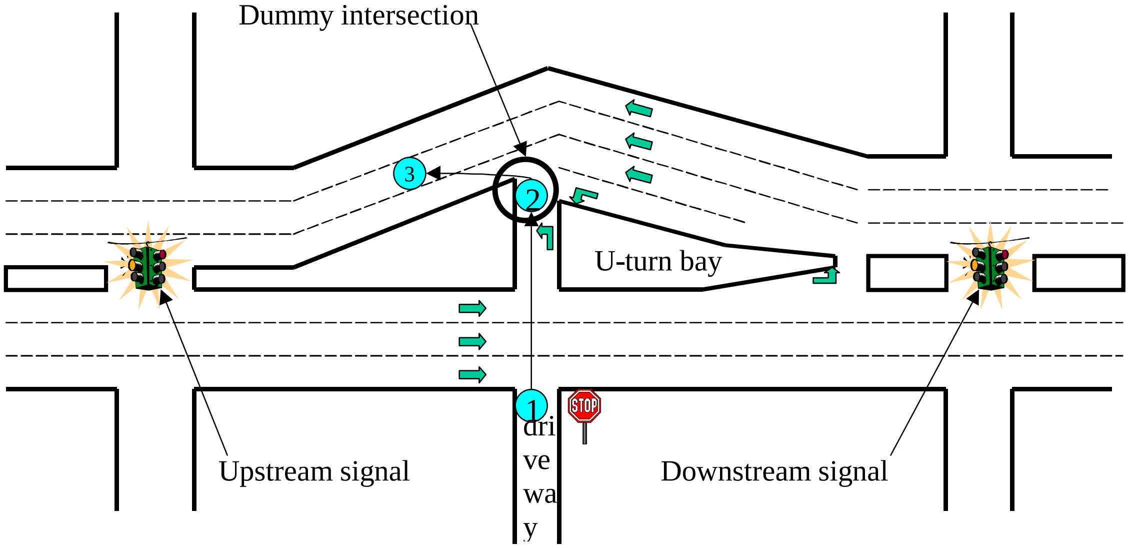


Figure 2.10 Graphic Description of DLT Model

2.3.3 Right Turn Plus U-turn Model

The current version of CORSIM cannot directly simulate U-turn movements. The dummy intersection model, which represents U-turns as two continuing left turns, is shown in Figure 2.11. The U-turn is completed by making the left turn onto the dummy link and stopping at the intersection waiting for the gap in the traffic from the right, and then by making left turns onto the main street. As shown in the Figure, RTUT vehicles first reach point ① seeking gaps of upstream traffic, and then weave to point ④ making a left turn to point ⑤ seeking the gaps of downstream traffic. Finally, they make a left turn again to get to point ⑥ to complete their U-turn movement.

This way of modeling the U-turn is named the dummy intersection approach because it uses an extra intersection to change the U-turn movement into two left turns.   In this study, the dummy intersection approach is used in simulating the right turn plus U-turn because this approach leaves more room to carry out model calibration than the coding approach, based on the case study.

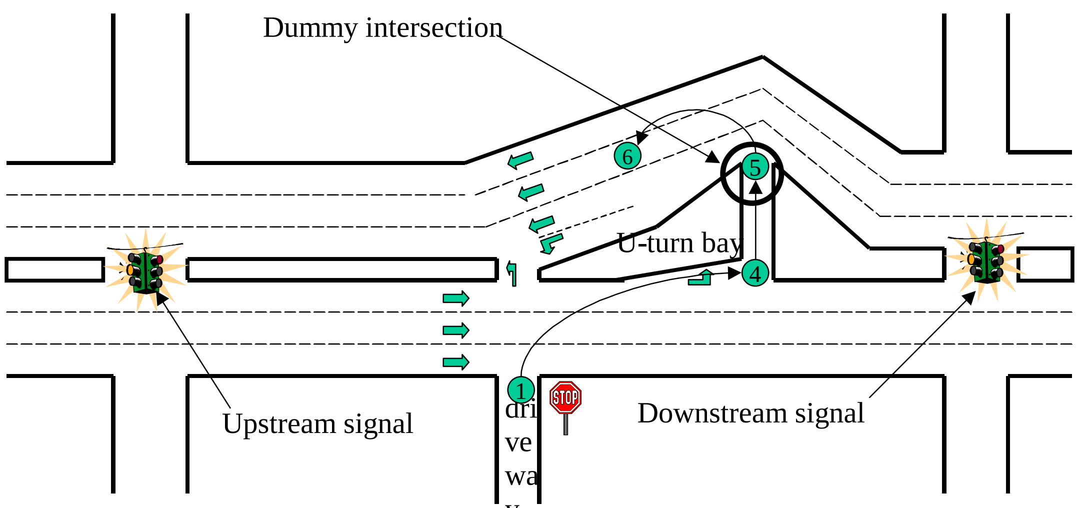


Figure 2.11 Graphic Description of U-Turn Model

2.3.4 Model Calibration

In general, there are two approaches to the calibration of the microscopic traffic simulation systems. The first one is model calibration, which re-establishes the input-output relation to obtain the desired system accuracy by changing the basic modules that describe the complex relationship between the input and output of the simulation systems. The other approach is parameter calibration, which is regarded as the optimization problem in which a set of values for operating parameters that satisfy the objective function is to be searched.

Specifically, parameter calibration of the model consists of systematically varying a number of the model parameters and comparing the MOE (selected) with the field data until there is a reasonable correspondence between two sets of MOE *(41)*. CORSIM provides a lot of parameters for model calibration. In this study, random number seeds, start-up lost time, free flow speed, lane change parameters (RT 81), lane change distribution (RT 152), queue discharge headway, and acceptable gaps (RT 143, 145) for turning movements were used for calibration. Link travel time and delay are selected as the MOE in the calibration process. This is a time consuming process because the values produced from the model should be correspondent to the values from field observation under the given level of confidence, which causes huge trial-and-error runs.

2.3.4 Network Building and Coding

A graphic description of a sample study site, which is a prototype model that needs to be modified for practical use, is shown in Figure 2.12. The CORSIM simulation coding for that model is thereafter described by link/node diagram as shown in Figure 2.13.

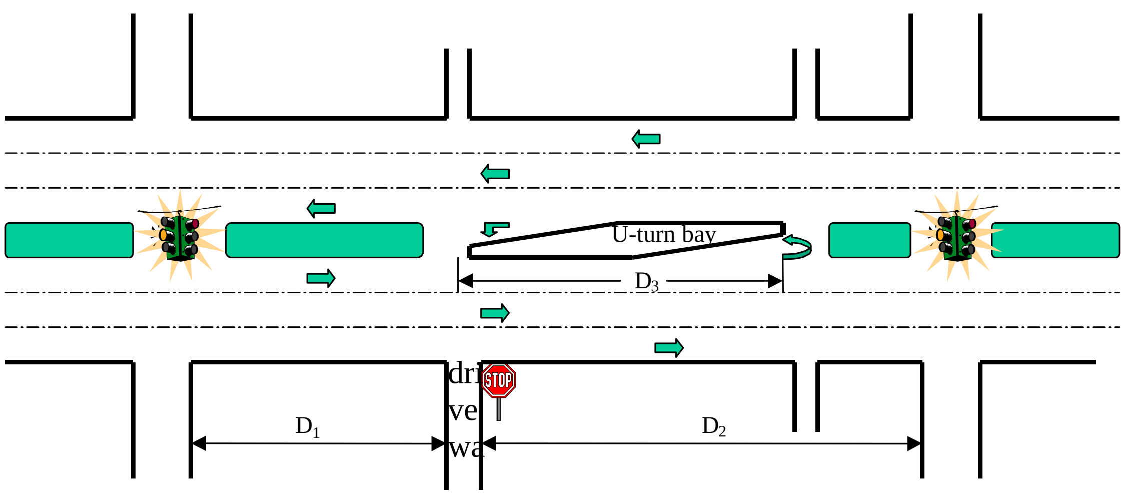


Figure 2.12 Graphic Description of Prototype Model

As shown in Figure 2.13, nodes numbered in 1, to 4 are internal nodes and nodes 5 to 14 are dummy nodes. Nodes numbered in 8000's are entry/exit nodes, which are used for traffic loading. The link (6-2) represents the subject driveway where a stop sign is used as the control device. Node 3 is the place where the vehicles make U-turn movement. Node 1 and 4 represent two signalized intersections nearby.

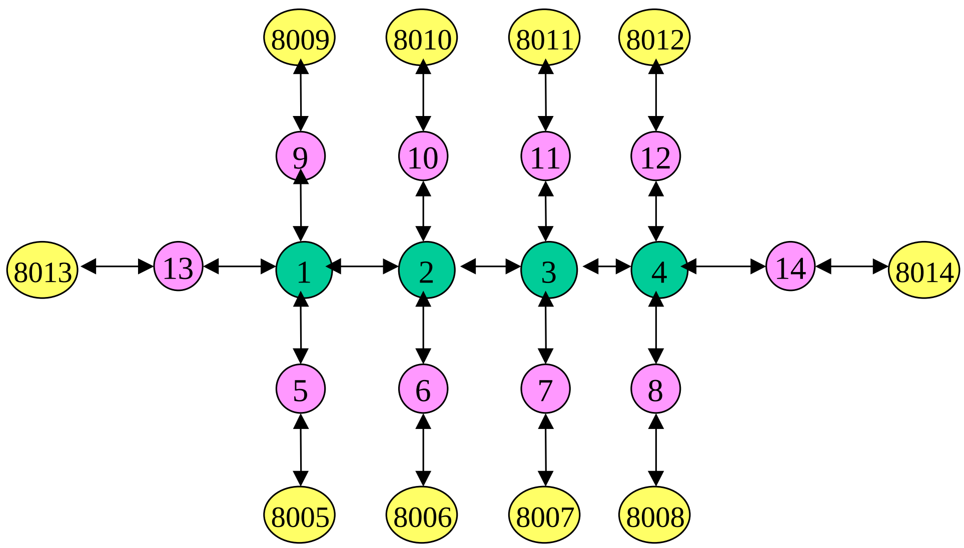


Figure 2.13 Link/Node Diagram of Prototype Model

The two-step DLT from a driveway is modeled by adding one dummy intersection at the median and either special coding tactics or a dummy intersection approach can be used to model RTUT. However, a case study found that the dummy intersection approach is better than the other method, because the dummy intersection approach leaves more room for model calibration. The final working simulation model developed to represent all traffic characteristics related to DLT and RTUT movements is described in Figure 2.14. The CORSIM simulation coding for that model is thereafter described by link/node diagram as shown in Figure 2.15.

The model shown in Figure 2.14 is the concept model, emphasizing the movements of DLT and RTUT. For example, the upstream-signalized intersection could be four-leg or three-leg at the selected sites. In addition, at the U-turn bay there could be a driveway allowing waiting vehicles to make both left turn and U-turn movements or there could be no driveway so that vehicles at the U-turn bay have to make a U-turn only. Each site should have a specific model representing the actual conditions. However, the central part of the model remains unchanged, which means that the codes at nodes 2, 3, 6, 15, 16, and 17 remain unchanged, as shown in Figure 2.15.

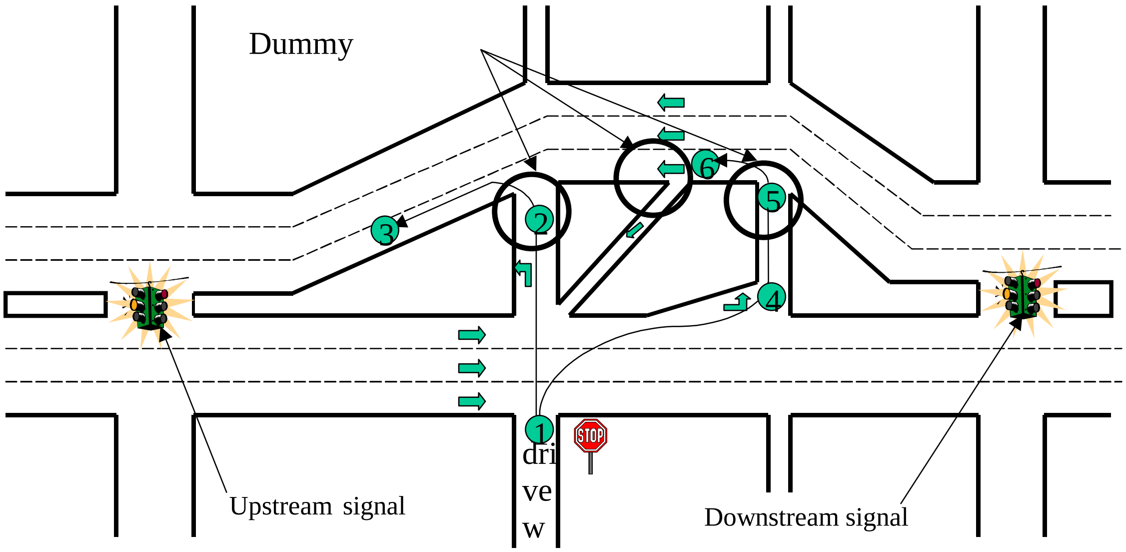


Figure 2.14 Graphic Description of Simulation Model for DLT & RTUT

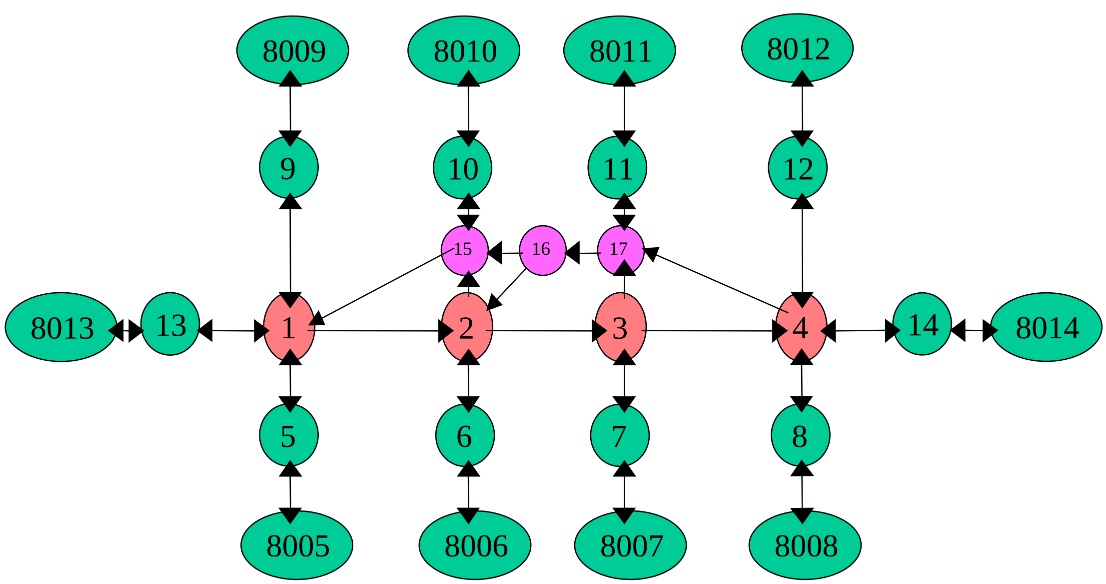


Figure 2.15 Link/Node Diagram of Working Simulation Model

2.3.5 Delay and Travel Time Models

In order to evaluate the traffic operational effects between DLT and RTUT under the same traffic conditions, delay and travel time values were obtained using the CORSIM based simulation. Volume levels of the driveway were divided into five categories, 50, 100, 150, 250, and 350 vph. For main street, five volume levels were 3000, 4000, 5000, 6000, and 7000 vph for both directions. Due to the stochastic feature of CORSIM models, one simulation run is not reliable. Therefore, an average of ten simulation runs of identical traffic environments was obtained as the final outcome, with the simulation time being 7200 seconds. In addition to developing the site-specific simulation models based on the characteristics of each location, a general simulation model that can be used for a wide variety of roadway networks with similar characteristics was also developed in this study. The development of the general model largely depends on the procedure of modeling for the eight locations and experience from that procedure. The control parameter used in the general simulation model was the average of the parameters for the eight site-specific models. The model testing was conducted by comparing the model output with the average values of the eight sites. The calibration of this general simulation model was based on the data collected from the eight sites so that it will adequately reflect the characteristics of the arterial, such as geometric, traffic, land use, and driver behavior.

In order to find the relationship between delay (travel time) and the explanatory variables, regression models were developed for delay and travel time for DLT and RTUT based on the simulation data produced from the general simulation model. The explanatory variables used in the regression procedure included through volume of main street, left-turn out volume from driveway, left in volume from main street, right turn plus U-turn volume, volume split on main street, and weaving distance.

3. OPERATIONAL EFFECTS THROUGH MODELING

3.1 General

In this study, operational effects of U-turns as alternatives to direct left turns were analyzed in four parts: (1) delay models for two movements; (2) travel time effects; (3) speed reduction of major-road through-traffic; and (4) amount of RTUT under both choices. Field data collected from eight sites were used to develop delay models for DLT and RTUT movements. Based on the delay models, the average total travel times of these two movements were estimated by adding the respective average running times. The average speed reduction of major-road through-traffic due to DLT and RTUT in the weaving section were also measured to evaluate the impacts on major-road through-traffic. Drivers' selection of a DLT or a RTUT is influenced by left-turn-in volume and major-road through-volume. A new variable, ratio of RTUT, was defined as the number of RTUT vehicles divided by the sum of the number of DLT and RTUT maneuvering volumes at fifteen-minute intervals. Field data collected from sites two through seven, where there were both options (DLT and RTUT), were used to investigate how these variables affected drivers' selection.

Microsoft Excel was used to develop statistical and engineering analyses explained in this chapter. Once the data and parameters are provided for each analysis, Excel uses the appropriate statistical or engineering macro functions and then displays the results in an output table. This analysis tool performs linear regression analysis by using the "least squares" method to fit a line through a set of observations. It can be used to analyze how a single dependent variable is affected by one or more independent variables; for example, how the average total delay of DLT is affected by such factors as through volume, split, left-turn-in volume, and direct left turn volume.

3.2 Average Total Delay

The following sections describe the procedure used to develop delay models for these two movements.

3.2.1 Delay Model for DLT

Data collected from sites two through seven were used to perform regression analysis for the DLT delay model because left-turn egress was prohibited through installing a restrictive median opening at sites one and eight. The dependent variable was the average total delay of DLT, including the average delay at driveways and the average waiting time at median openings. Independent variables included the flow rate of major-road through-traffic, split, the flow rate of left-turn-in traffic from a major roadway, and the flow rate of DLT as shown in Figure 3.1.

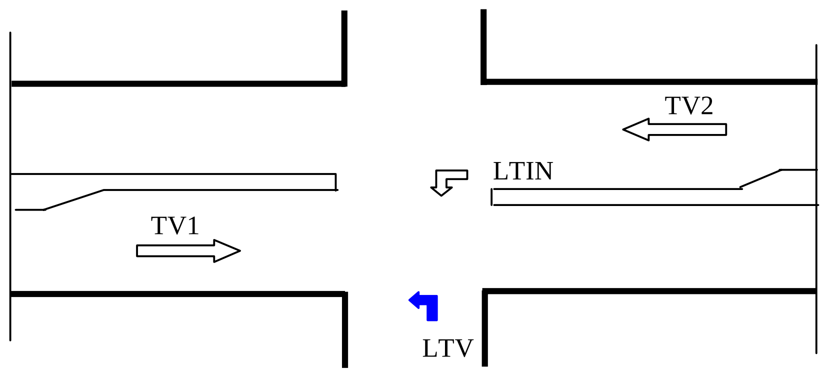


Figure 3.1 Traffic Flow Affecting the Delay of DLT

The original data at five-minute intervals were aggregated to fifteen-minute intervals because the data at fifteen-minute intervals were found to have better statistical characteristics. Analysis showed that linear and exponential forms were appropriate to describe the relationship. However, the exponential form was found to have better theoretical and statistical characteristics. The delay model was described as Equation 3-1.

$$TD_L = e^{a_1 TV + a_2 LTV + a_3 LTIN + a_4 SPLIT + a_0} \qquad (3\text{-}1)$$

where,

TD_L - average total delay of DLT (sec./vehicle),

TV - flow rate of major-road through-traffic (vph), TV=TV1+TV2,

LTV - flow rate of DLT from a driveway (vph),

$LTIN$ -flow rate of left-turn-in from major roads (vph), *SPLIT* - percentage of upstream through traffic flow rate,

SPLIT=TV1/(TV1+TV2), and

a_0, a_1, a_2, a_3, a_4 - parameters.

Total of 451 observations at fifteen-minute intervals, whose statistical characteristics are given in Table 3.1, were used to estimate the delay model for DLT. The dependent variable (average total delay of DLT) refers to average total waiting delay per vehicle making a left turn during a fifteen-minute period. The independent variables, including left-turn-in flow rate, through traffic flow rate, and DLT flow rate, are equal to four times traffic volume at fifteen-minute intervals. Multiple regression analysis was carried out to determine the best model by testing different independent variables. The final regression results are listed in Table 3.2.

Table 3.1 Descriptive Statistics of DLT Delay Data

Parameter	Average Total Delay	Flow Rate of DLT	Flow Rate of TV	Flow Rate of LTIN	Split
Mean	50.08	46	4910	80	0.48
Standard Error	1.39	1	30	1	0.00
Median	43.96	44	4864	76	0.48
Mode	#N/A	32	4384	68	#N/A
Standard Deviation	29.51	20	636	31	0.04
Sample Variance	870.66	381	404785	981	0.00
Range	149.83	132	3204	172	0.22
Minimum	6.68	12	3532	8	0.38
Maximum	156.51	144	6736	180	0.61
Count	451	451	451	451	451

Table 3.2 Regression Results for Delay Models of DLT

N	R-Square		Intercept	TV	LTV	LTIN	SPLIT
451	0.39	Coefficients	0.47	0.0006	0.011	0.004	-1.18
		t- statistics	1.14	14.89	8.51	5.04	-1.86

As shown in Table 3.2, the delay model of DLT included four independent variables. The variable TV, LTV and LTIN were significant at a 95 percent confidence level. The flow rate of through-traffic was the most significant variable with a fairly high t-statistic of 14.89. However, the t-stat suggested that independent variable SPLIT (t=-1.86) and Intercept (t=1.14) were significant at a 90 and 75 percent confidence level, respectively. According to these parameter estimates, the final developed regression equation was:

$$TD_L = 1.6e^{0.0006TV + 0.011LTV + 0.004LTIN - 1.18SPLIT} \tag{3-2}$$

where,

TD_L - average total delay of DLT (sec./vehicle),

TV - flow rate of major-road through-traffic(vph),

LTV - flow rate of DLT from a driveway (vph),

$LTIN$ - flow rate of left-turn-in from the major road (vph), and

$SPLIT$ -percentage of upstream through traffic flow rate,

SPLIT=TV1/(TV1+TV2).

In Equation 3-2, the coefficient of LTV (0.011) is much greater than the coefficient of TV (0.0006). This implies that DLT flow rate has greater impact on the delay of DLT than that of major-road through-traffic. The independent variable SPLIT has a negative coefficient, indicating that the downstream through-traffic flow rate (TV2) has a greater impact on the delay than corresponding upstream flow rate (TV1). This is because when the median space is occupied by other maneuvers, left-turn vehicles must wait at the driveway even if suitable gaps are available at the upstream through-traffic stream. The intercept refers to the minimum delay of a DLT when the volumes approach zero, where the model provided a reasonable value of 0.47 sec. The residual plot for each independent variable was obtained from the results of regression analysis. It was found that the residual for each independent variable was randomly scattered about the x-axis line, which indicated that the model was correctly specified.

Based on Equation 3-2, curves for the average delay of DLT can be developed. Figure 3.2 shows a group of curves for average delay of DLT assuming the left-turn-in flow rate from the major road was 100 vph, split was 0.5, and the flow rate of DLT was made equal to 50, 100, and 150 vph, respectively. The x-axis represents the flow rate of two-directional through-traffic on the major road. The y-axis represents the average total delay of DLT.

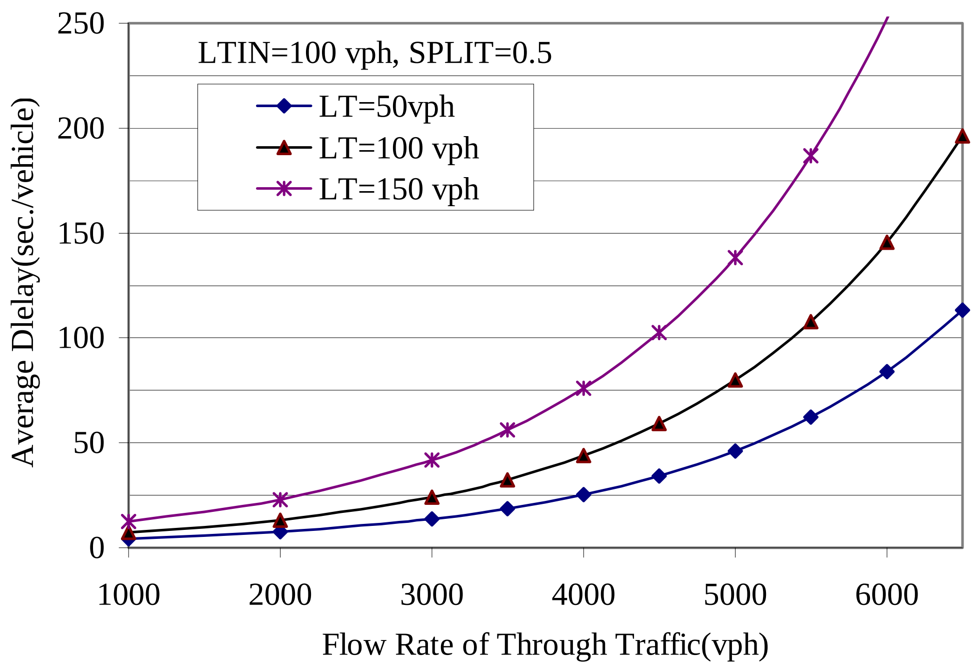


Figure 3.2 Curves for the Average Total Delay of DLT (LTIN=100 vph, Split=0.5)

3.2.2 Delay Model for RTUT

Field data collected from eight sites were used to develop the delay model for RTUT. Sites one and eight had directional median openings and therefore, only right turn followed by U-turn was allowed. Other sites had both DLT and RTUT options where the intervals with only RTUT movements were included in the analysis data set.

The average total delay model for RTUT can be described as follows:

$$TD_{RU} = e^{a_1 TV + a_2 RUV + a_3 SPLIT + a_0} \tag{3-3}$$

where,

TD_{RU} - average total delay of RTUT (sec./vehicle),

TV - flow rate of major-road through-traffic (vph),

RUV - flow rate of RTUT (vph),

$SPLIT$ - percentage of upstream through-traffic flow rate,

SPLIT=TV1/(TV1+TV2), and

a_0, a_1, a_2, a_3 - parameters.

The dependent variable was the average total delay of RTUT, including the average delay of right turns at the subject driveway and average delay of U-turns at the median opening at fifteen-minute intervals. As shown in Figure 3.3, variables expected to affect the average delay of RTUT included two-directional through-traffic flow rate (TV), split, and RTUT flow rate (RUV). RUV refers to the number of vehicles making a right turn at the driveway followed by a U-turn at the downstream median opening in one hour.

Total of 614 observations at fifteen-minute intervals were used to perform the regression analysis. Table 3.3 illustrates the descriptive statistics of the collected data. The mean of average total delay of RTUT (37 sec./vehicle) was less than the mean of average total delay of DLT (50 sec./vehicle). Sample variance and standard deviation for average delay of RTUT were much less than those for DLT. The split of through-traffic flow-rate has the range from 0.41 to 0.58. The maximum and minimum through-traffic flow rate is 2600 vph and 6400 vph, respectively.

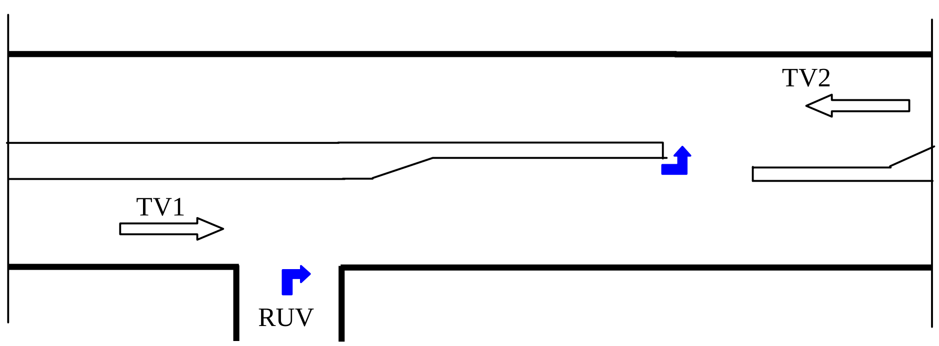


Figure 3.3 Traffic Flow Affecting the Delay of RTUT

Table 3.3 Descriptive Statistics of RTUT Delay Data

Parameter	RUD	RUV	TV	SPLIT
Mean	36.79	95	4271	0.50
Median	35.60	92	4238	0.50
Mode	#N/A	72	4776	0.48
Standard Deviation	13.76	41	645	0.03
Sample Variance	189.35	1665	415683	0.00
Range	66.86	220	3850	0.18
Minimum	7.10	12	2562	0.41
Maximum	73.96	232	6412	0.58
Count	614	614	614	614

Results of regression analysis for RTUT delay are given in Table 3.4. The model includes three independent variables, major-road through-traffic flow rate, RTUT flow rate, and split. The regression analysis suggested that major-road through-traffic flow rate (t=20.1), RTUT flow rate (t=7.7)) were significant at a 95% confidence level. The independent variable (SPLIT, t=0.85) was not significant at a 95 percent confidence level. The positive sign for SPLIT implies that the upstream through-traffic flow rate has a greater impact on the delay of RTUT movements. The intercept represents the minimum delay of a RTUT. The R-square of the model is about 0.44.

Table 3.4 Regression Results for Delay Models of RTUT

N	R-Square		Intercept	TV	RUV	SPLIT
614	0.44	Coefficients	1.42	0.0004	0.0023	0.38
		t- statistics	5.7	20.1	7.7	0.85

Based on regression results, the developed regression equation for average delay of RTUT movements was as follows:

$$TD_{RU} = 4.1e^{0.0004TV + 0.0023RUV + 0.38SPLIT} \qquad (3\text{-}4)$$

where,

TD_{RU} - average total delay of RTUT (sec./vehicle),

TV - flow rate of major-road through-traffic (vph),

RUV - flow rate of RTUT (vph), and

$SPLIT$ - percentage of upstream through-traffic flow rate,

SPLIT=TV1/(TV1+TV2).

A group of curves for the average total delay of RTUT can be developed based on Equation 3-4. Figure 3.4 shows a group of curves for average total delay of RTUT assuming the SPLIT is equal to 0.5 and the RTUT flow rates are equal to 50, 100, and 150 vph, respectively. The x-axis represents the major-road through-traffic flow rate; the y-axis represents the average total delay of RTUT. The three curves are very close because the average delay is not very sensitive to the flow rate of RTUT.

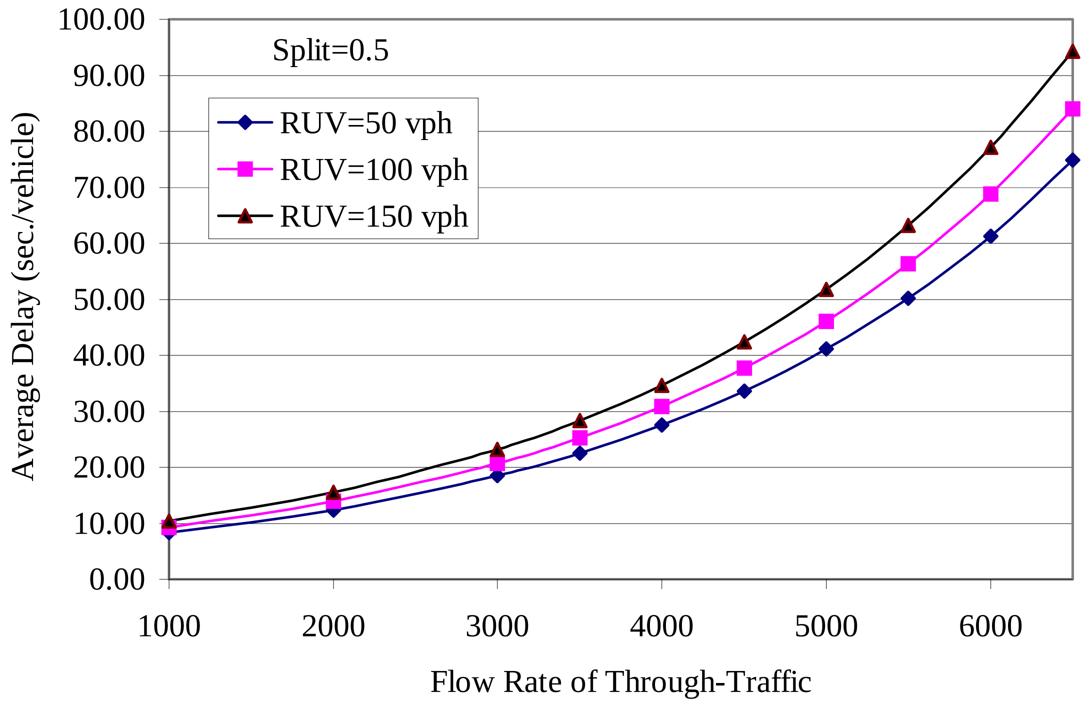


Figure 3.4 Curves for Average Total Delay of RTUT (Split=0.5)

3.2.3 Delay Comparison

When a full median opening is replaced by a directional median opening, the direct left turns must be diverted to make a right turn followed by a U-turn at the downstream median opening. Obviously, DLT will have less delay than RTUT when the flow rates are low. But when the flow rates increase, the delay of DLT increases rapidly because of the restrained median storage. To compare the average total delay of two movements, Figures 3.2 and 3.4 were combined together to obtain Figure 3.5, assuming the left-turn-in flow rate is 100 vph and the SPLIT is 0.5.

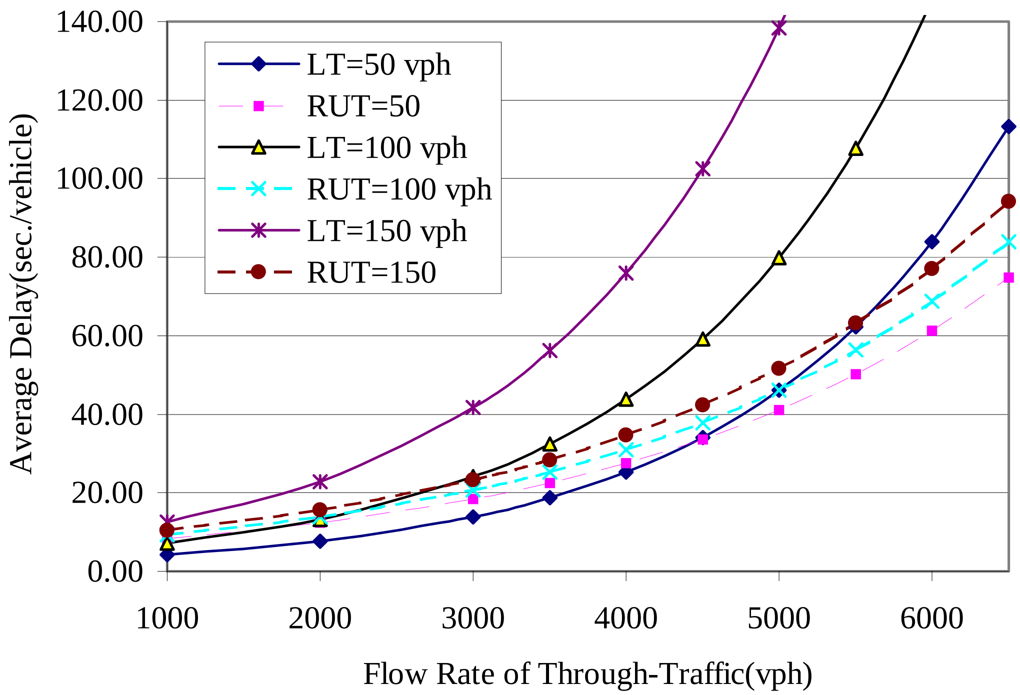


Figure 3.5 Comparison of Average Delay of Two Movements

As shown in Figure 3.5, the breakpoints for delay of these two movements can be found as follows:

(1) When both DLT and RTUT flow rates are equal to 50 vph, the average total delay of DLT will be less than RTUT until the through-traffic flow rate is greater than 4500 vph;

(2) When both DLT and RTUT flow rate are equal to 100 vph, the average total delay of DLT will be less than RTUT until the through-traffic flow rate is greater than 2200 vph; and

(3) When the DLT flow rate reaches 150 vph, the average total delay of DLT will be always greater than RTUT.

The average total delay is much more sensitive to the flow rate of DLT than RTUT because the median can only store one or two left-turn vehicles at each time. However, several U-turn vehicles can easily store at a left turn deceleration lane at the same time.

3.3 Travel Time Effects

As defined earlier, the average total travel time of RTUT includes the average total delay, the average running time in the weaving section, and the average running time of vehicles traversing the weaving segment at the posted speed along the major arterial. To estimate the average total travel time of RTUT movements, an empirical equation was developed to calculate the average running time in different weaving distances. The average running time of vehicles traversing the weaving segment in the operating speed on the major-stress was explained earlier in Chapter 2**.** The average total travel time of DLT is equal to the average total delay plus the average running time for vehicles from leaving the driveway to stopping at the median opening. Based on field observations, about 4.0-5.0 seconds were required for vehicles to cross 3 or 4 through lanes.

3.3.1 Average Running Time in Different Weaving Distances

Average running time of RTUT in the weaving section is the time used by a vehicle traversing the weaving segment. This time was recorded for each vehicle at eight sites with different weaving distances. The frequency histogram and cumulative curve of the average running time for each site are given in Figures 3.6 through Figure 3.12. As shown in the Figures, average running time at site 1, 2, 3, 4, 7, and 8 indicate a normal distribution. There were no sufficient RTUT vehicles at sites five and six.

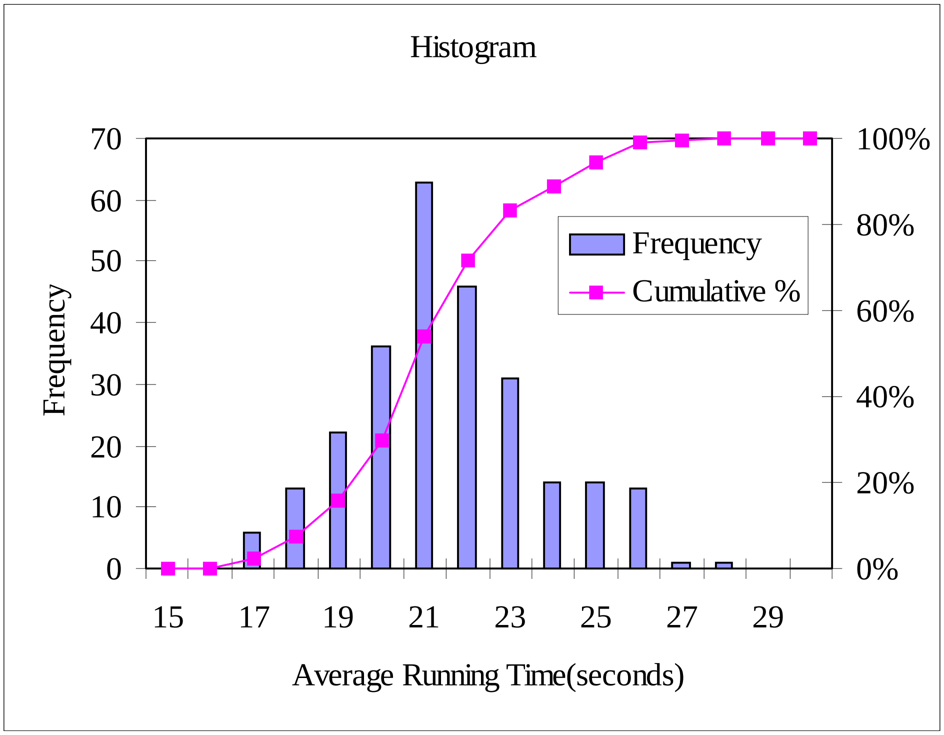


Figure 3.6 Average Running Time in the Weaving Section (800 ft.) of Site 1

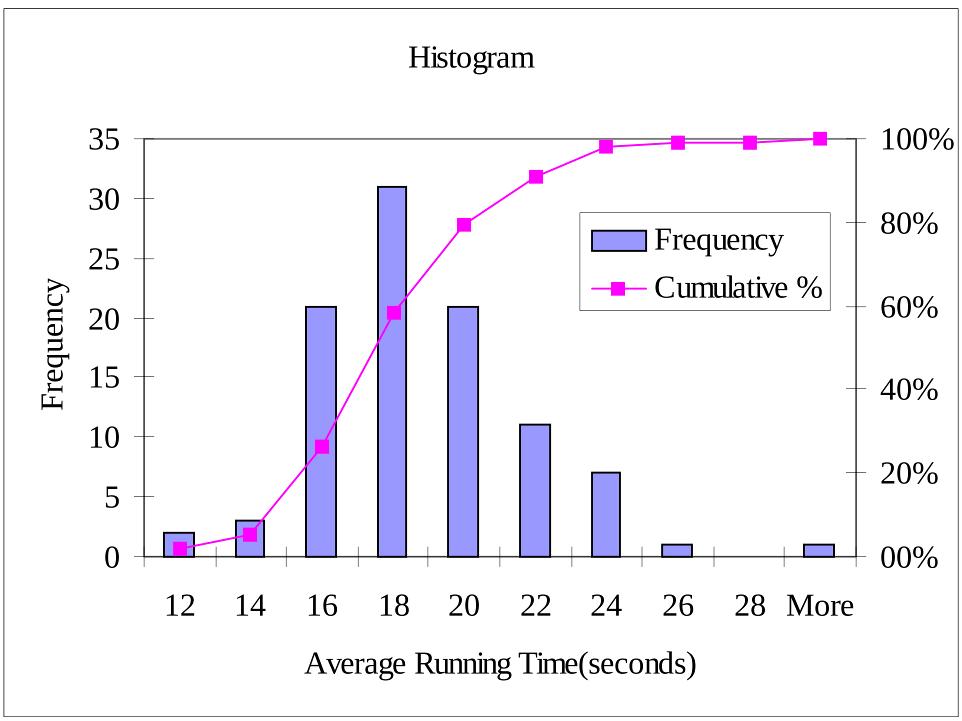


Figure 3.7 Average Running Time in the Weaving Section (570 ft.) of Site 2

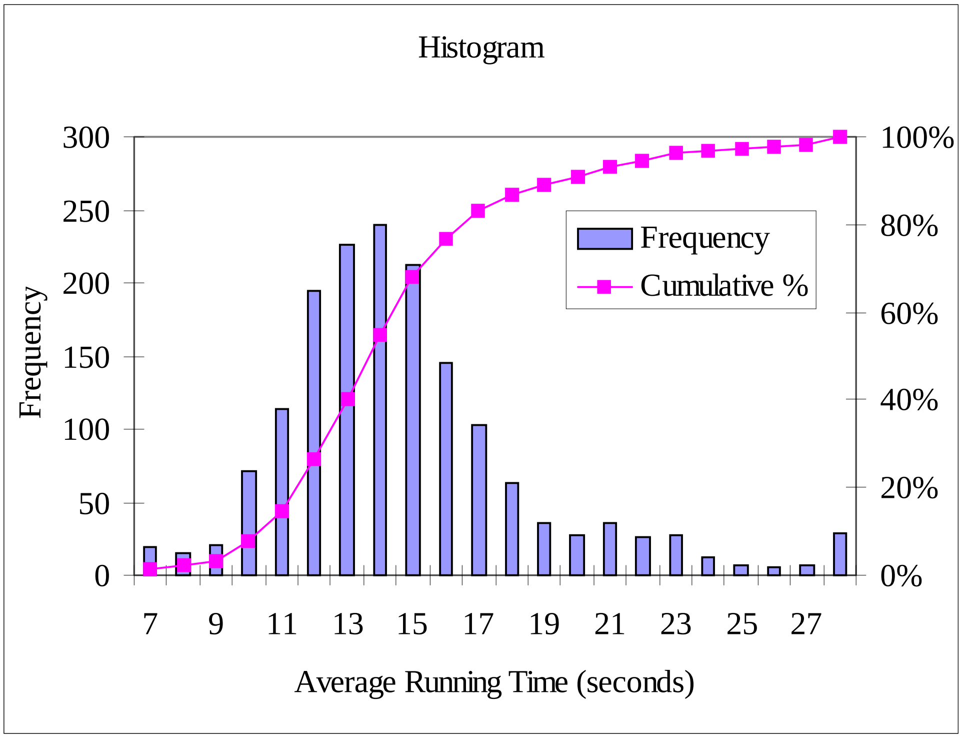


Figure 3.8 Average Running Time in the Weaving Section (420 ft.) of Site 3

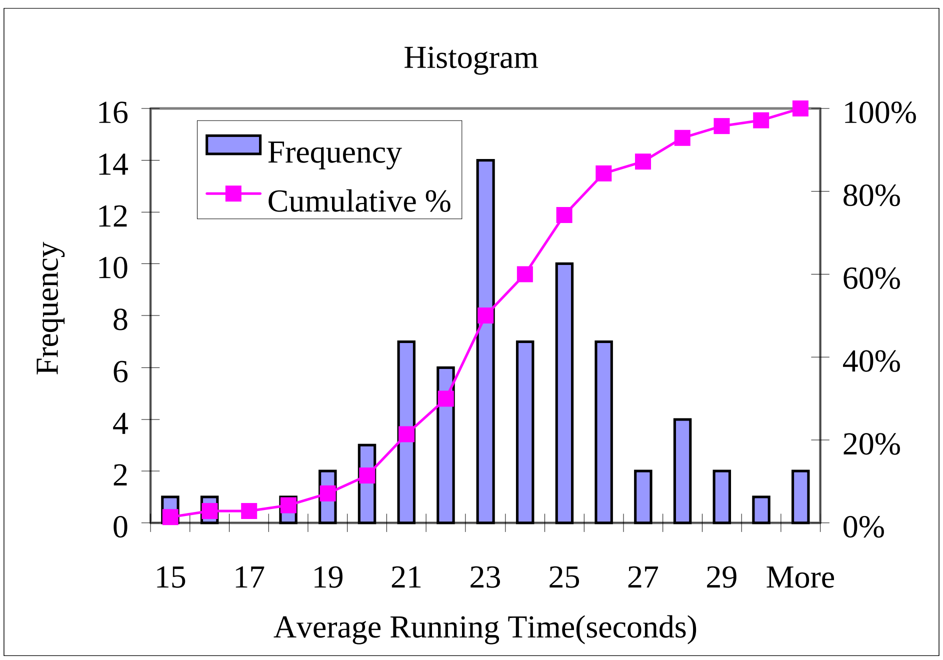


Figure 3.9 Average Running Time in the Weaving Section (970 ft.) of Site 4

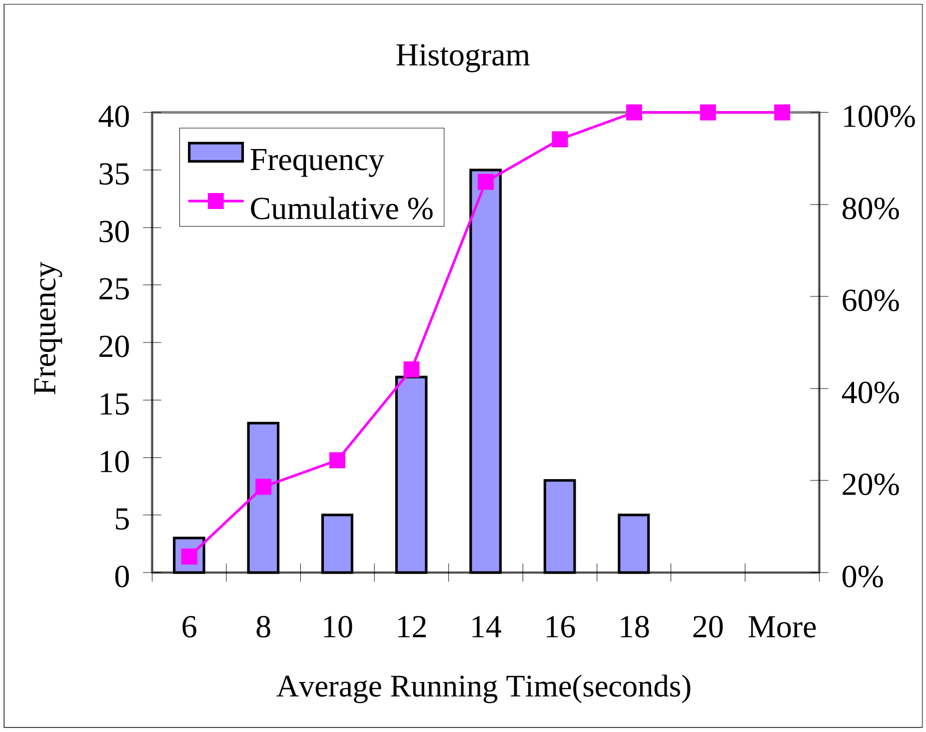


Figure 3.10 Average Running Time in the Weaving Section (300 ft.) of Site 5

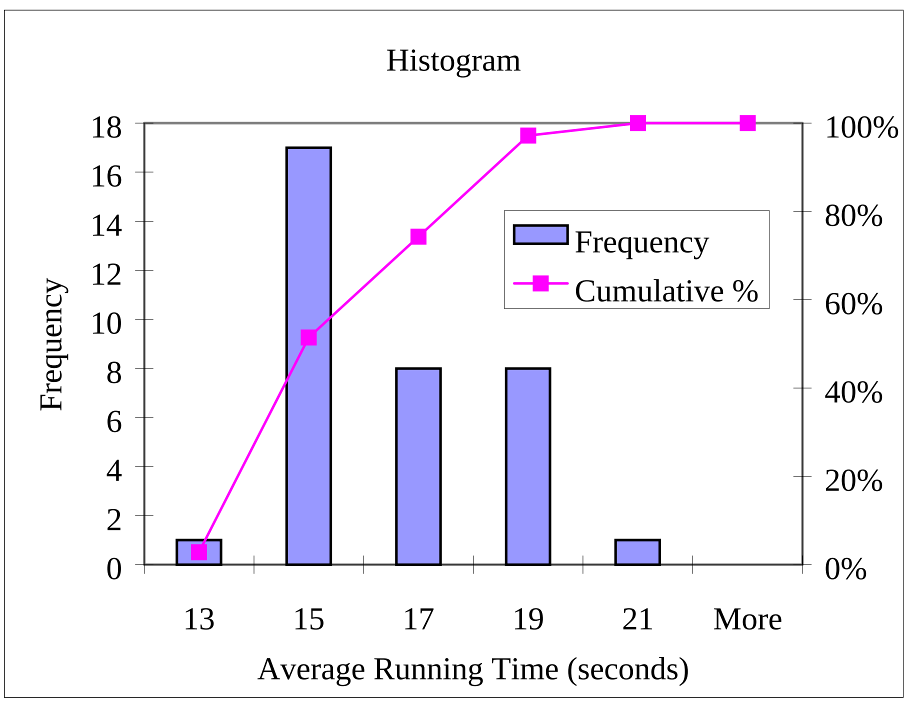


Figure 3.11 Average Running Time in the Weaving Section (550 ft.) of Site 6

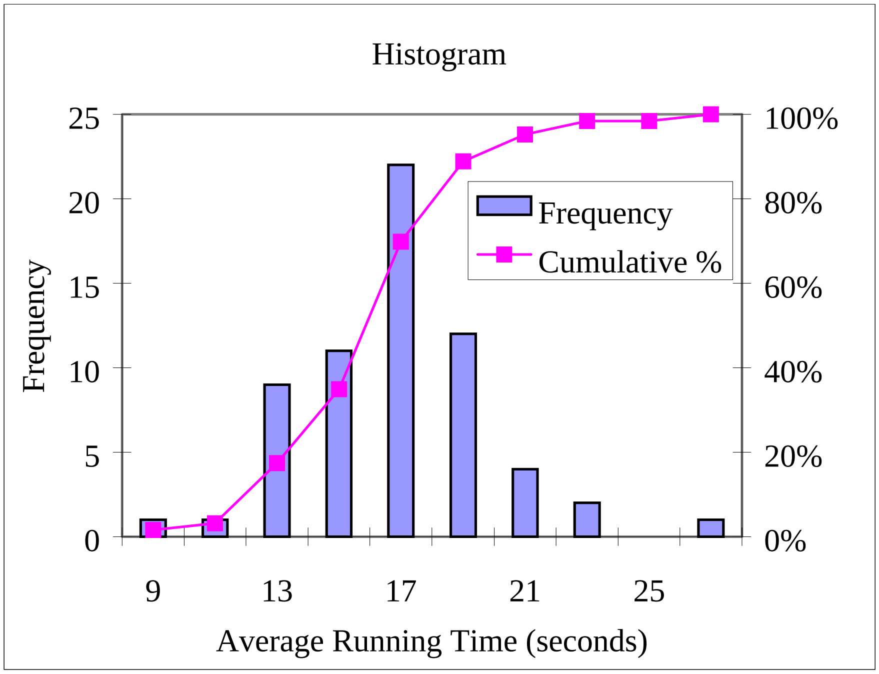


Figure 3.12 Average Running Time in the Weaving Section (600 ft.) of Site 7

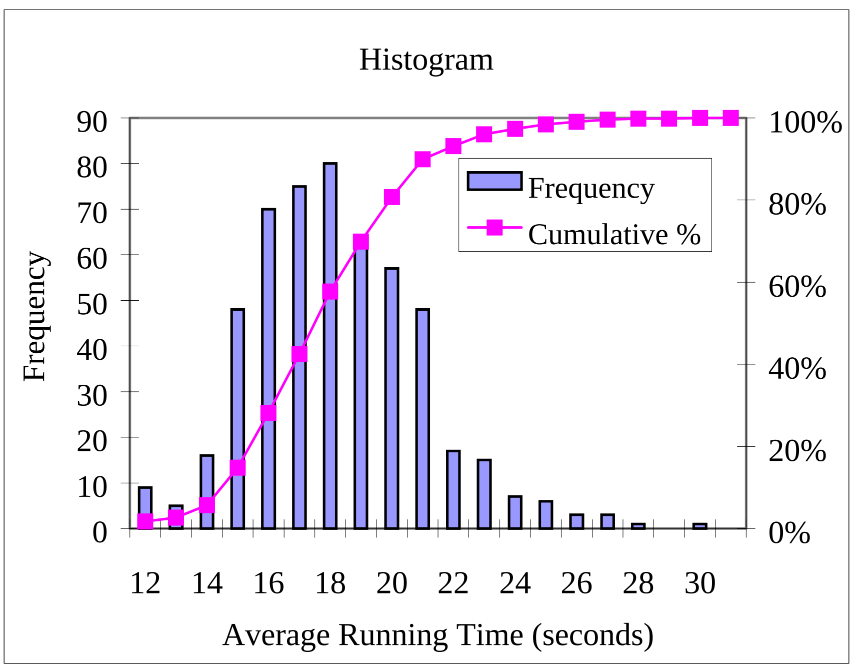


Figure 3.13 Average Running Time in the Weaving Section (580 ft.) of Site 8

Table 3.5 lists the statistical characteristics of the collected data in the eight sites.

Table 3.5 Descriptive Statistics of the Average Running Times

Site	1	2	3	4	5	6	7	8
Weaving Distance (ft.)	800	570	420	970	300	550	600	580
Mean (sec.)	21.5	18.0	14.8	23.9	12.0	15.9	16.5	18.2
Median	21.0	18.0	14.0	23.5	13.0	15.0	16.0	18.0
Mode	21.0	18.0	14.0	23.0	13.0	14.0	17.0	18.0
Standard Deviation	2.1	2.9	4.0	3.5	2.9	2.0	3.0	3.0
Range	11.0	18.0	26.0	22.0	12.0	8.0	18.0	24.0
Minimum	17.0	12.0	7.0	15.0	6.0	13.0	11.0	6.0
Maximum	28.0	30.0	33.0	37.0	18.0	21.0	29.0	30.0
Count	260	96	1637	70	86	35	62	524

A linear relationship was set up between the weaving distance and the average running time of RTUT. Figure 3.14 illustrates the fitting line and original data points at eight sites representing weaving distance and average running time. The empirical equation of the fitting line is as follows:

$$t_{RU2} = 5.1 + 0.021l \tag{3-5}$$

where,

t_{RU2} - average running time of RTUT in the weaving section (seconds),

l - weaving distance (ft.).

The fitted line had a fairly high R^2 value of 0.97. The range of weaving distance at eight sites was from 300 ft. to 1,000 ft. Therefore caution should be employed in the application of this equation. When weaving distance is more than 1000 ft., the average running time of RTUT may be shorter than the result calculated because the average running speed increases when the weaving distance increases.

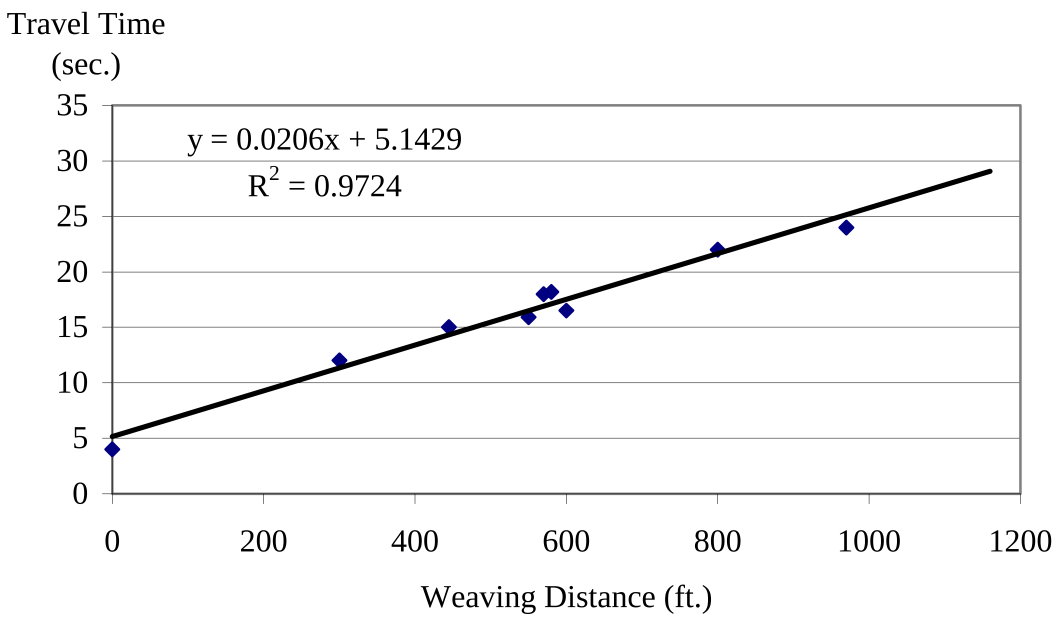


Figure 3.14 Average Running Time vs. Weaving Distance

3.3.2 Travel Time Model of DLT

Average total travel time of DLT is the sum of average total delay and average running time of vehicles crossing the through lanes. The same dataset for the delay model was used to develop the travel time model for DLT. The dependent variable was the average total travel time of DLT at fifteen-minute intervals. The independent variables were as same as those for the delay model of DLT. Regression results are listed in Table 3.6. The variables (through-traffic flow rate, DLT flow rate, and left-turn-in flow rate) have a positive sign and high t- values.

Table 3.6 Regression Results for Travel Time Model of DLT

N	R-Square		Intercept	TV	LTV	LTIN	SPLIT
451	0.39	Coefficients	0.87	0.00055	0.0092	0.004	-0.89
		t- statistics	2.38	15.12	8.15	5.0	-1.60

Based on regression results, the average travel time of DLT can be calculated by following empirical equation:

$$TT_L = 2.4e^{0.00055TV + 0.0092LTV + 0.004LTIN - 0.89SPLIT} \quad (3\text{-}6)$$

where,

TV - flow rate of major-road through-traffic (vph),

LTV - flow rate of DLT from driveways (vph),

$LTIN$ - flow rate of left-turn-in from major roads (vph), and

$SPLIT$ -percentage of upstream through traffic flow rate .

3.3.3 Travel Time Model of RTUT

Average total travel time for RTUT is more complicated than that for DLT. As defined earlier, the average total travel time of RTUT includes average total delay, average running time in the weaving section, and running time for a vehicle traversing the weaving distance at posted speed.

The travel time model for RTUT was developed using regression by considering average total travel time at fifteen-minute intervals as the dependent variable. In addition to the independent variables considered for the delay model, weaving distance and speed were also considered as potential independent variables and the results are listed in Table 3.7. The intercept of the travel time model of RTUT is much greater than that for DLT because the minimum travel time required for RTUT traversing the weaving segment is more than that of DLT crossing the through lanes. The variable, weaving distance, has a positive sign and high *t* value. The SPEED variable is negative, which implies that RTUT will take less travel time when there is a relatively high speed limit on the major arterial.

Table 3.7 Regression Results of Travel Time Model for RTUT

N	R-Square		Intercept	TV	RUV	L	SPEED	SPLIT
614	0.46	Coefficients	2.62	0.00023	0.00079	0.00065	-0.0026	0.39
		t- statistics	13.31	19.98	4.14	10.10	1.29	1.47

The empirical equation for average total travel time of RTUT is as follows:

$$TT_{RU} = 13.9e^{0.00023TV + 0.00079RUV + 0.00065l + 0.39SPLIT - 0.0026SPEED} \qquad (3\text{-}7)$$

where,

TT_{RU} - average total travel time of RTUT (sec./vehicle),

TV - flow rate of major-road through-traffic (vph),

RUV - flow rate of RTUT (vph),

l - weaving distance (ft.),

SPEED- speed limit along the arterial (mph), and

SPLIT- percentage of upstream through-traffic flow rate.

3.3.4 Travel Time Comparison

Based on Equations 3-6 and 3-7, the average total travel time of two movements can be calculated under different traffic flow conditions. Figure 3.15 is an example of travel time comparison assuming that the left-turn-in flow rate was 100 vph, weaving distance was 600 ft., split is 0.5, and speed limit was 50 mph. For this case, the breakpoint for travel time comparison of DLT and RTUT can be found as follows:

(1) When both DLT and RTUT flow rates are equal to 50 vph, the average total travel time of RTUT is greater than that of DLT until the major-road through-traffic flow rate is greater than 5600 vph;

(2) When both flow rates are equal to 100 vph, RTUT has less travel time than DLT when the through-traffic flow rate is more than 4500 vph; and

(3) When both flow rates are equal to 150 vph, RTUT will suffer less travel time when the through-traffic flow rate is about 3100 vph.

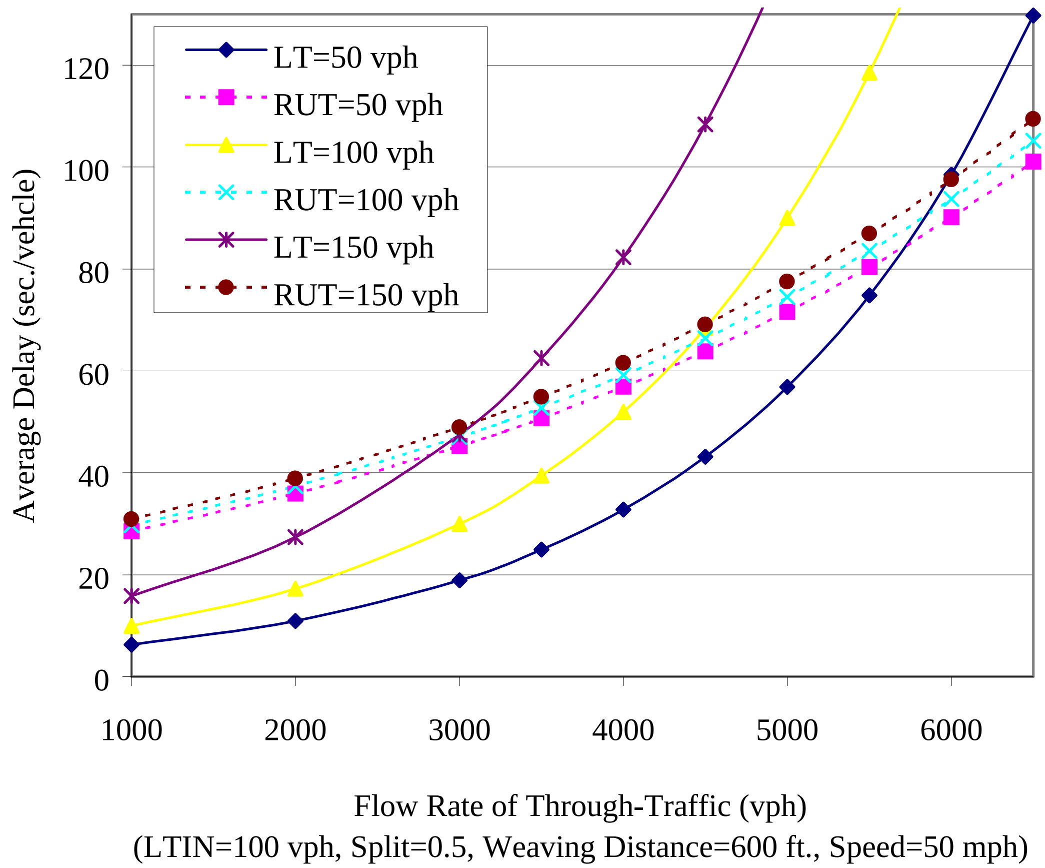


Figure 3.15 Travel Time Comparison (Left-turn-in=100 vph, Weaving distance =600 ft., Split=0.5, Speed = 50 mph)

A sample table as shown in Table 3.8 was developed to compare the delay and travel time of the two movements under the assumed conditions. Flow rates of DLT and RTUT were also assumed equal. The flow rates were classified into three groups: low (50 vph), median (100 vph), and high (150 vph). The through-traffic flow rate is from 1000 vph to 6500 vph. Tables could be developed for different combinations of left-turn-in flow rates, weaving distances, and splits These tables can be used as a reference to compare the delay and travel time of DLT at a full median opening and RTUT after installing a directional median opening. Comparison of delay and travel time of these two movements may help determine if a full median opening should be replaced with a directional median opening.

Table 3.8 Comparison of Delay and Travel Time of Two Movements

Left turn in volume=100 vph, Split=0.5, Weaving distance=600ft., Speed limit=50 mph					
Volume		Delay		Travel Time	
LT/RU	T-Volume	LT	RU	LT	RU
50	1000	4.18	8.30	6.30	28.54
50	2000	7.61	12.38	10.92	35.92
50	3000	13.87	18.47	18.92	45.21
50	3500	18.73	22.56	24.91	50.72
50	4000	25.28	27.55	32.80	56.91
50	4500	34.12	33.65	43.18	63.84
50	5000	46.06	41.10	56.85	71.62
50	5500	62.18	50.20	74.85	80.35
50	6000	83.93	61.31	98.54	90.14
50	6500	113.30	74.89	129.73	101.13
100	1000	7.24	9.31	9.98	29.69
100	2000	13.20	13.89	17.30	37.37
100	3000	24.05	20.72	29.98	47.03
100	3500	32.46	25.30	39.47	52.77
100	4000	43.82	30.91	51.96	59.20
100	4500	59.15	37.75	68.41	66.41
100	5000	79.84	46.11	90.06	74.51
100	5500	107.77	56.32	118.57	83.59
100	6000	145.47	68.79	156.10	93.77
100	6500	196.37	84.01	205.50	105.20
150	1000	12.55	10.44	15.81	30.89
150	2000	22.87	15.58	27.40	38.88
150	3000	41.68	23.24	47.49	48.93
150	3500	56.26	28.39	62.52	54.89
150	4000	75.94	34.67	82.31	61.58
150	4500	102.51	42.35	108.36	69.09
150	5000	138.38	51.73	142.66	77.51
150	5500	186.79	63.18	187.82	86.95
150	6000	252.14	77.17	247.27	97.55
150	6500	340.36	94.25	325.53	109.44

3.4 Speed Reduction

3.4.1 Speed Reduction of Through-Traffic in the Weaving Sections

RTUT movements may have some impacts on major-road through-traffic in the weaving segment. One impact could be the speed reduction of the major-road through-traffic. In order to verify that the relationship between the average through traffic speed and RTUT flow rate during peak and non-peak hours was obtained based on the field data for sites with sufficiently high RTUT volumes. Figures 3.16 –3.18 show that the average speed of major-road through-traffic decreased slightly with the increase of RTUT flow rate for peak hour and non-peak hour conditions in the daytime at sites 1, 3, and 8, respectively. Table 3.7 lists the slope of all the trend lines at the three sites. The range of the slope was from -0.005 to –0.010. This implies that there was about a 0.5-1.0 mph speed reduction of major-road through-traffic caused by every 100 RTUT movements per hour. The average value shows that there was a 0.7 mph speed reduction of major-road through-traffic during non-peak hours, and 1.0 mph speed reduction during the peak-hours when the flow rate of RTUT was about 100 vph.

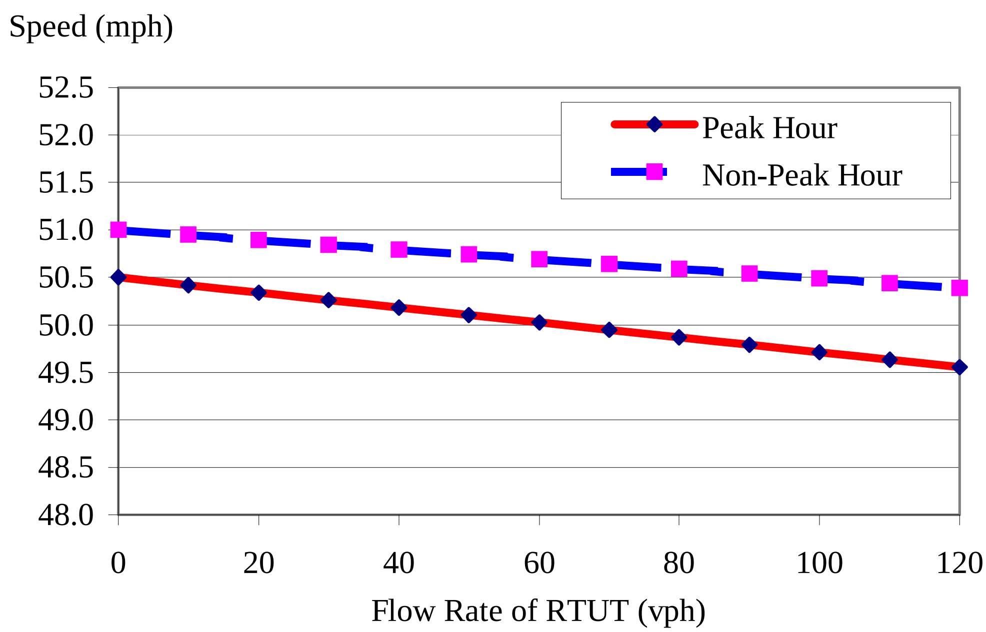


Figure 3.16 Major-Road Traffic Speed Reduction Due to RTUT Movements in the Weaving Section of Site One

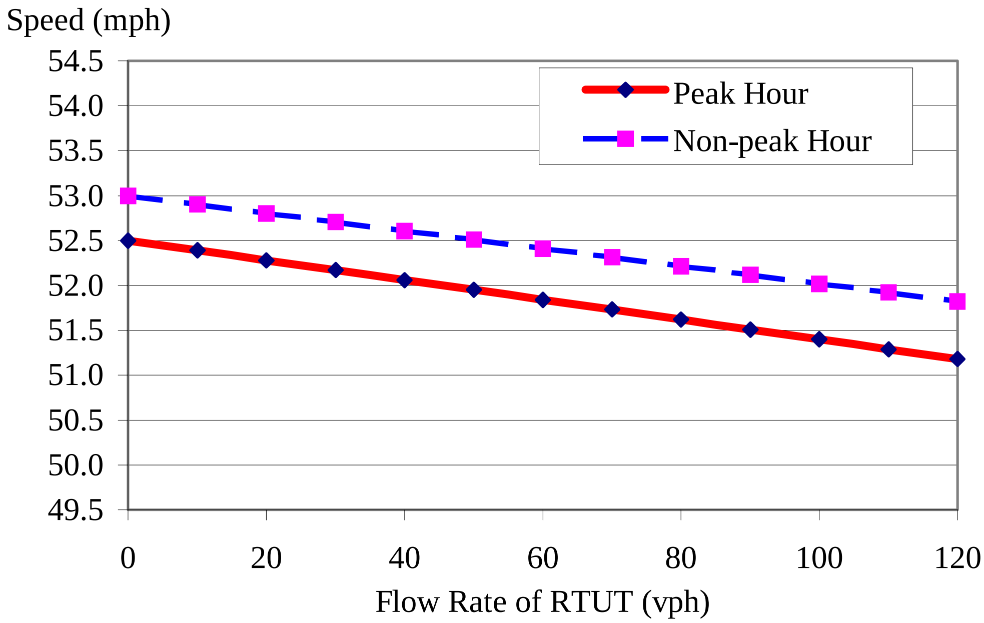


Figure 3.17 Major Road Traffic Speed Reduction Due to RTUT Movements in the Weaving Section of Site Three

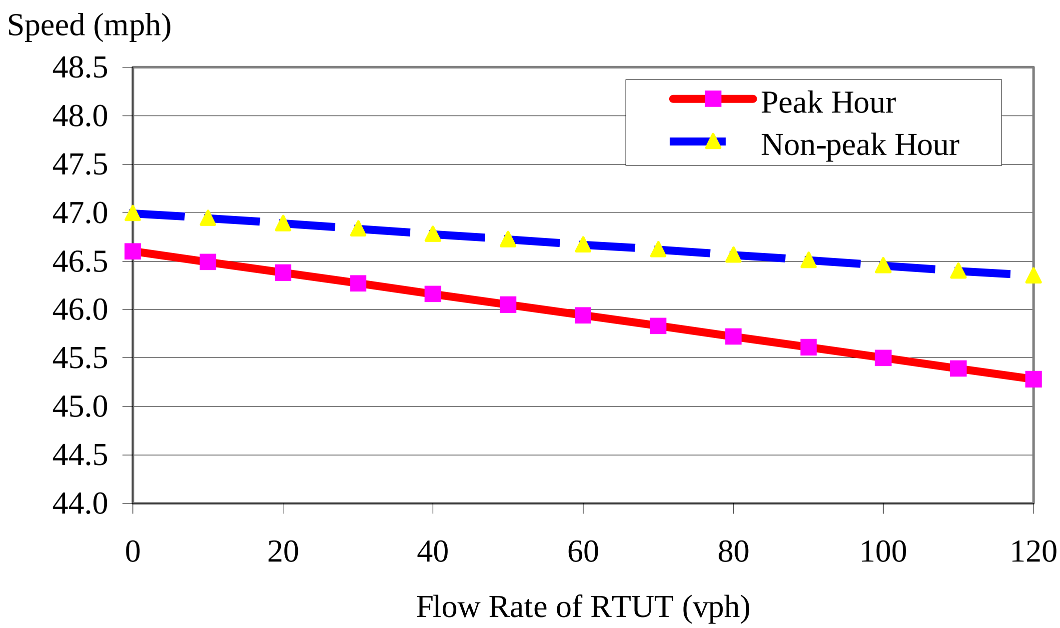


Figure 3.18 Major Road Traffic Speed Reduction Due to RTUT Movements in the Weaving Section of Site Eight

Table 5.9 Summary of the Impacts of RTUT on Through-Traffic

Site	Period	Slope of the Fitting Line	RUV (vph)	Speed Reduction (mph)
1	Peak Hour	-0.008	100	0.8
1	Non-peak Hour	-0.005	100	0.5
3	Peak Hour	-0.011	100	1.1
3	Non-peak Hour	-0.01	100	1.0
8	Peak Hour	-0.011	100	1.1
8	Non-peak Hour	-0.006	100	0.6
Average	Peak Hour	-0.010	100	1.0
Average	Non-peak Hour	-0.007	100	0.7

3.4.2 Weaving Speed

Weaving speed defined as the space mean speed of vehicles making a RTUT in the weaving section was computed as the length of the segment divided by the average running time. The running time is the average time taken by a RTUT to traverse the weaving segment. A video camera was set up to monitor the weaving section and record the space mean speed of vehicles making a RTUT. The average weaving speed was computed as the length of weaving segment divided by the average running time at each site. Figure 3.19 shows the fitting line and original data points at eight sites. With the increase of the weaving distance, the average weaving speed increases. This implies that it may help the weaving maneuvers and reduce the speed difference in the weaving section if there is a longer weaving distance.

A regression model for prediction of the average weaving speed in the different length weaving segments was developed as follows:

$$S_w = 13.8 + 0.015l \tag{3-8}$$

where,

S_w - average weaving speed of RTUT movements (mph), and

l - weaving distance (ft.).

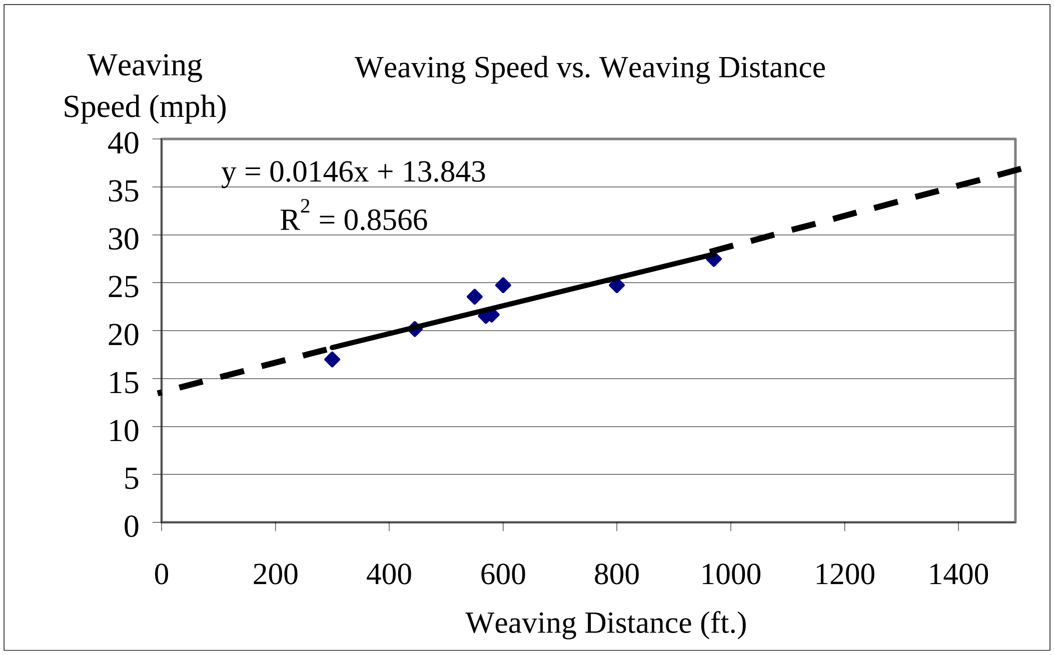


Figure 3.19 Average Weaving Speed in Different Weaving Distances

3.4.3 Speed Reduction Due to DLT

Speed of major-road through-traffic may also be affected by DLT from a driveway. To measure the impacts on the through-traffic speed due to DLT movements, an automatic traffic recorder was installed at 100 ft. upstream of the driveway because conflicts between major-road through-traffic and a DLT vehicle often happened in this area based on traffic conflicts analysis. Speed data were averaged to a five-minute interval. Figures 5.20 through 5.24 show that the average speed of major-road through-traffic decreases slightly with the increase of DLT flow rate during peak hour and non-peak hour conditions in the daytime at sites 3, 4, 6, 7, and 8, respectively. Table 3.10 lists the slope of all the fitting lines at the five sites. The range of the slope was from -0.004 to –0.020, implying that there was about a 0.4-2.0 mph speed reduction of through-traffic caused by 100 DLT vehicles. The average value shows that there was a 0.9 mph speed reduction during non-peak hours, and 1.7 mph speed reduction during peak hours due to 100 DLT vehicles per hour.

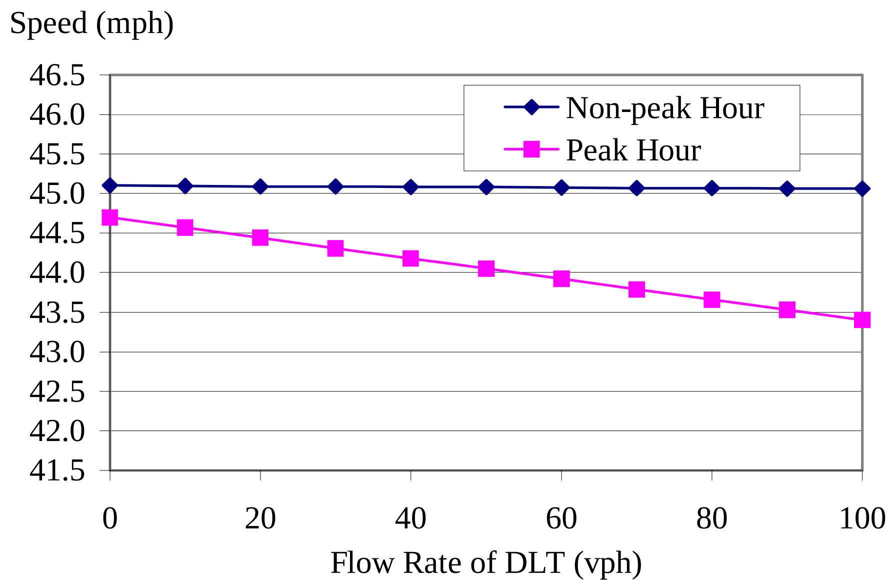


Figure 3.20 Speed Reduction of Through-Traffic Due to DLT at Site 2

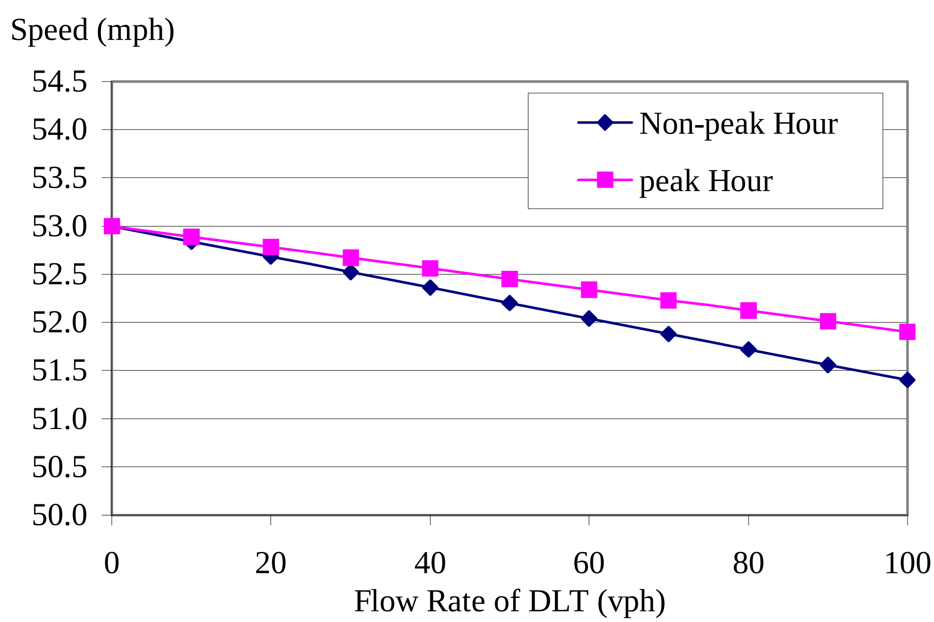


Figure 3.21 Speed Reduction of Through-Traffic Due to DLT at Site 3

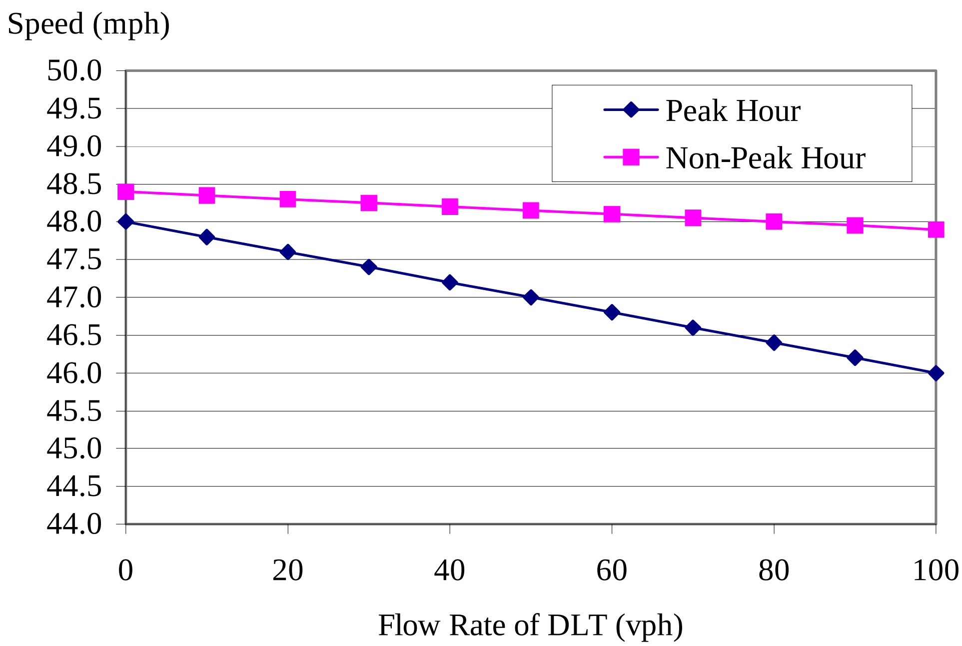


Figure 3.22 Speed Reduction of Through-Traffic Due to DLT at Site 4

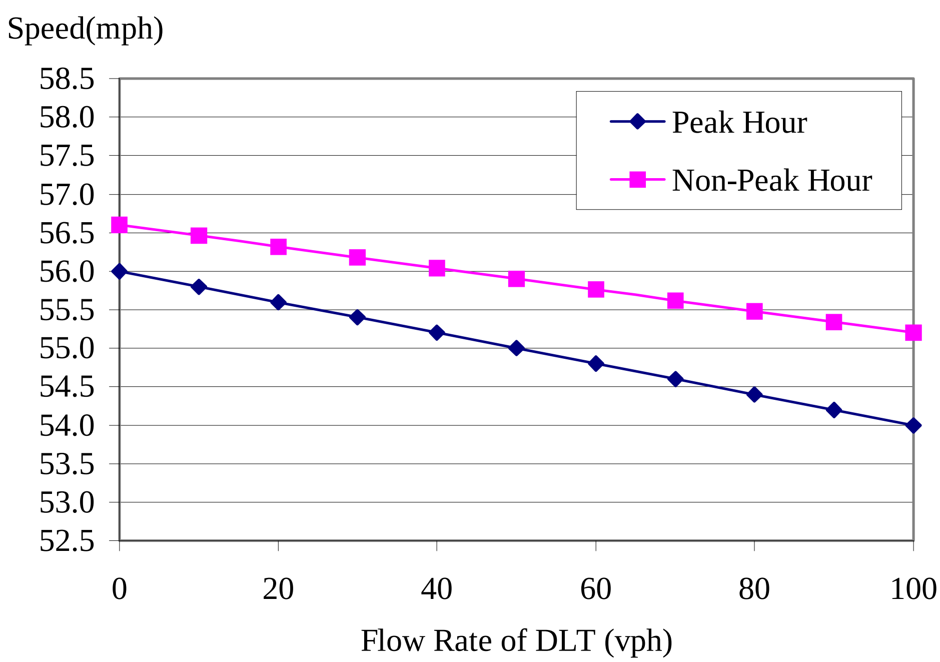


Figure 3.23 Speed Reduction of Through-Traffic Due to DLT at Site 6

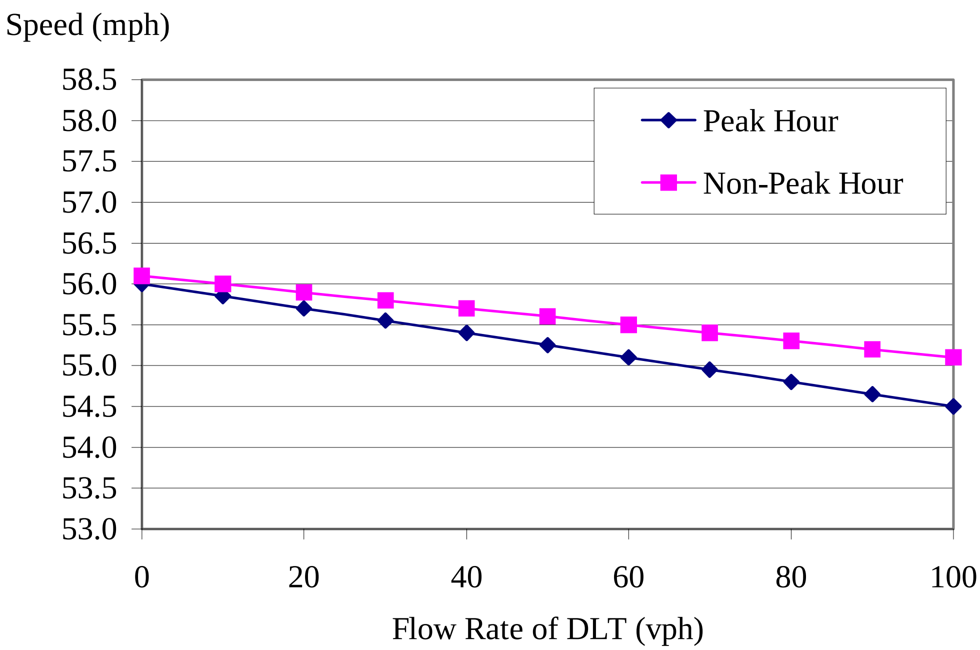


Figure 3.24 Speed Reduction of Through-Traffic Due to DLT at Site 7

Table 3.10 Summary of Impacts of DLT on Through-Traffic

Site	Period	Slope of the Fitting Line	RUV (vph)	Speed Reduction (mph)
2	Peak Hour	-0.013	100	-1.3
2	Non-peak Hour	-0.004	100	-0.4
3	Peak Hour	-0.016	100	-1.6
3	Non-peak Hour	-0.011	100	-1.1
4	Peak Hour	-0.020	100	-2.0
4	Non-peak Hour	-0.005	100	-0.5
6	Peak Hour	-0.020	100	-2.0
6	Non-peak Hour	-0.014	100	-1.4
7	Peak Hour	-0.015	100	-1.5
7	Non-peak Hour	-0.010	100	-1.0
Average	Peak Hour	-0.017	100	-1.7
Average	Non-peak Hour	-0.009	100	-0.9

3.4.4 Summary of Impacts on Through Traffic

Based on the above analysis, it was found that DLT has greater impacts on average speed of major-road through-traffic (Figure 3.25). During peak hours, there was about 1.7 mph speed reduction of through-traffic when the flow rate of DLT was 100 vph and about 0.9 mph speed reduction of through-traffic when the flow rate of RTUT was 100 vph in the weaving section. During non-peak hours, there was about 1.0 mph speed reduction of through-traffic when the flow rate of DLT was equal to 100 vph and there was about 0.7 mph reduction when the flow rate of RTUT is 100 vph in the weaving section.

In practice, the speed of major-road through-traffic, however, was affected by other factors such as through-traffic volume, right-turn volume, and left-turn volume on the major road as well. It is very difficult to control these factors and measure only the impacts by DLT from a driveway or RTUT in the weaving section. Therefore, these findings may imply the combined effect.

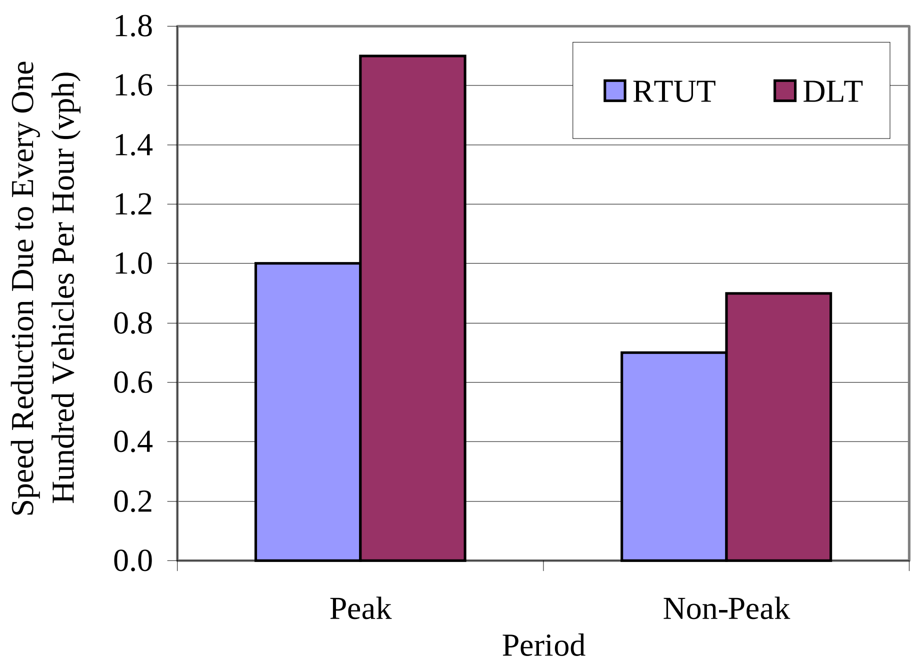


Figure 3.25 Speed Reduction Due to One Hundred Turning Vehicles per Hour

3.5 Amount of RTUT under Both choices

In practice, when there is a suitable U-turn median opening downstream, some drivers prefer to make a RTUT rather than a DLT to avoid conflict with all other movements at the median opening. This decision is encouraged when the median storage space is occupied by other maneuvers or when there is a large left-turn-in volume from the major-road. The drivers' selection of a RTUT or a DLT will be affected by traffic volume conditions.

The ratio of RTUT was defined as the number of RTUT divided by the sum of DLT and RTUT at fifteen-minute intervals as shown in Equation 3-8.

$$\text{Ratio} = (\text{\# of RTUT})/(\text{\# of RTUT} + \text{\# of DLT}) \tag{3-8}$$

Field data collected from sites 2 through 7 were used to develop the relationship between the ratio of RTUT and the combination of left-turn-in flow rate and major-road through-traffic flow rate. The regression results are given in Table 3.11.

Table 3.11 Regression Results for Ratio of RTUT

N	R-Square		Intercept	TV	LTIN	SPLIT
105	0.36	Coefficients	-1.48	0.0002	0.004	-2.19
		t- statistics	-2.95	3.89	4.83	-2.94

$$Ratio = 0.23e^{0.004LTIN + 0.0002TV - 2.1SPLIT} \tag{3-9}$$

where,

Ratio - percentage of RTUT at fifteen-minute intervals,

LTIN - left-turn-in flow rate from the major-road (vph),

TV - flow rate of major-road through-traffic (vph), and

SPLIT- percentage of upstream through-traffic flow rate, SPLIT=TV1/TV.

Equation 3-9 was developed based on 105 observations, which yielded a R^2 of 0.36. Only intervals when there are both DLT and RTUT were chosen to perform the regression analysis. All the independent variables have a relatively high *t-stat* value and are significant at 95 percent confidence level. Split carries a negative sign implying that downstream through-traffic flow rate has a greater impact on the ratio than the upstream though-traffic flow rate.

Figure 3.26 shows the relationship between the ratio and major-road through-traffic flow rate assuming that the SPLIT was equal to 0.5 and left-turn-in flow rate were 50, 150, and 250 vph, respectively. According to the figure,

(1) When the left-turn-in flow rate is equal to 50 vph, the ratio is always below 50 percent;

(2) When the left-turn-in flow rate is about 150 vph, the ratio is equal to 50 percent when the major-road through-traffic flow rate is about 6100 vph; and

(1) When the left-turn-in flow rate is about 250 vph, the ratio is equal to 50 percent if the major-road through-traffic flow rate is about 4100 vph.

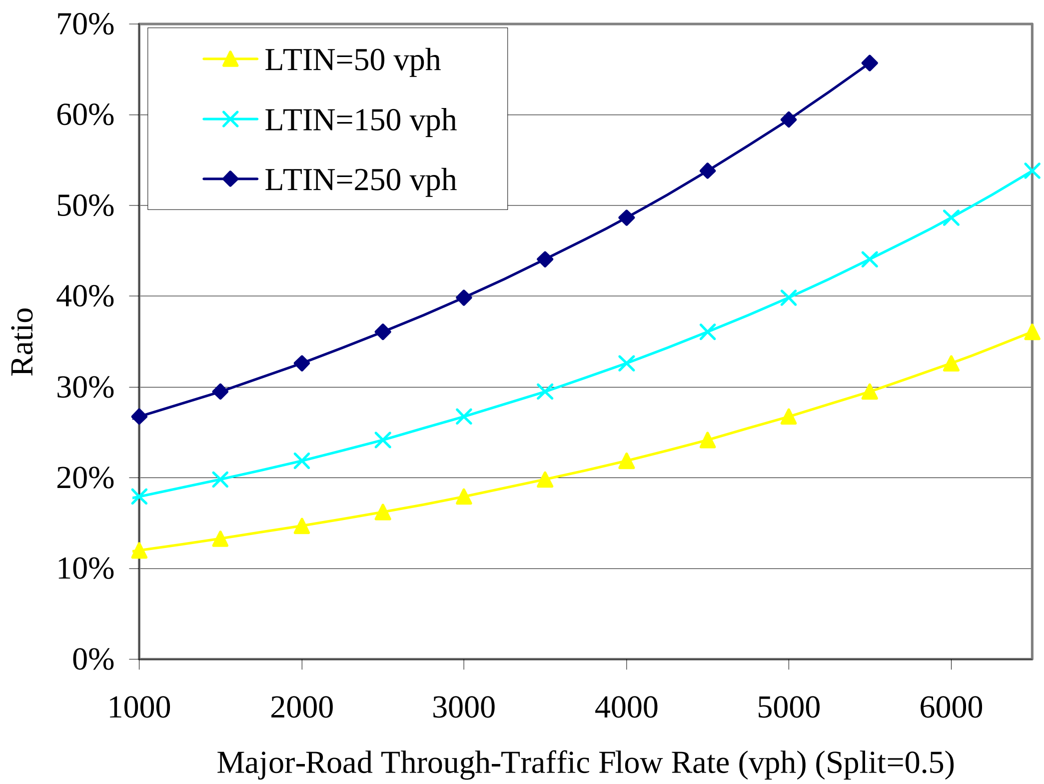


Figure 3.26 Ratio of RTUT vs. Major-Road Through-Traffic Flow Rate

Based on field observations, it was found that more drivers make RTUT when there was relatively high left-turn-in flow rate (more than 200 vph) and major-road through-traffic flow rate (more than 4000 vph). If the ratio of RTUT was over 50 percent, a full median opening might be replaced with a directional median opening while providing a downstream U-turn median opening to accommodate the diverted left turns.

3.6 Summary

Four major conclusions emanate from this part of the study, where empirical models were developed to evaluate the operational effects of DLT and RTUT, as follows:

(1) Delay and travel time models for DLT and RTUT can be used to determine under what traffic flow rate conditions (major road, left-turn-in, and driveway) DLT would experience more delays or travel time as compared to RTUT;

(2) Speed reduction of major-road through-traffic was about 0.9 mph during the non-peak periods and about 1.7 mph during the peak periods, if the DLT flow rate was 100 vph;

(3) Speed reduction of major-road through-traffic is about 0.7 mph during non-peak periods and 1.0 mph during the peak periods, if the RTUT flow rate was 100 vph.

(4) The driver selection of a RTUT or a DLT on the basis of accessibility considerations is affected by traffic flow conditions. An empirical formula was developed to estimate percentage of RTUT under different traffic flow rate conditions; and

(5) Average running time of RTUT in the weaving segment had a linear relationship with the weaving distance.

4. CASE STUDY: BEFORE-AND-AFTER ANALYSIS

4.1 Introduction

This chapter describes a case study where comparisons of the delay and travel times of DLT and RTUR movements were conducted using a 'before and after' study. The before-and-after analysis may provide additional information to help determine operational effects of replacing a full median opening with a directional median opening in terms of weighted average total delay and weighted average total travel time.

Field data were collected at the US 19 and 115th St. intersection in Pinellas County, Florida for about one week before and after the full median opening was replaced with a directional median opening. Field data collection was conducted in the daytime during the weekdays under good weather conditions. A total of 37 hours of before and after data were collected in the field.

4.1 Existing Conditions

The study area included two median openings and a 420 feet long weaving section between theses two median openings as shown in Figure 4.1. This segment of US19 is a six-lane divided highway with a speed limit of 55 mph. The 26 ft. wide median has a raised curb. The studied site is located in a suburban area. The subject driveway is connected to a large residential community generating a high driveway volume. During the peak-hour, there was a fairly high percentage of RTUT maneuvers. The distances from the subject driveway to the upstream signal and downstream signal are about 600 ft. and 1,620 ft., respectively.

Figure 4.2 shows a full median opening across from the driveway during the before period, which allows left turn egress from the driveway. The median was channelized into a directional median opening by installing physical barriers, as shown in Figure 4.3. Left turn egress from the driveway was replaced with a right turn followed by a U-turn at the downstream median opening, which is 420 ft. downstream from the subject driveway.

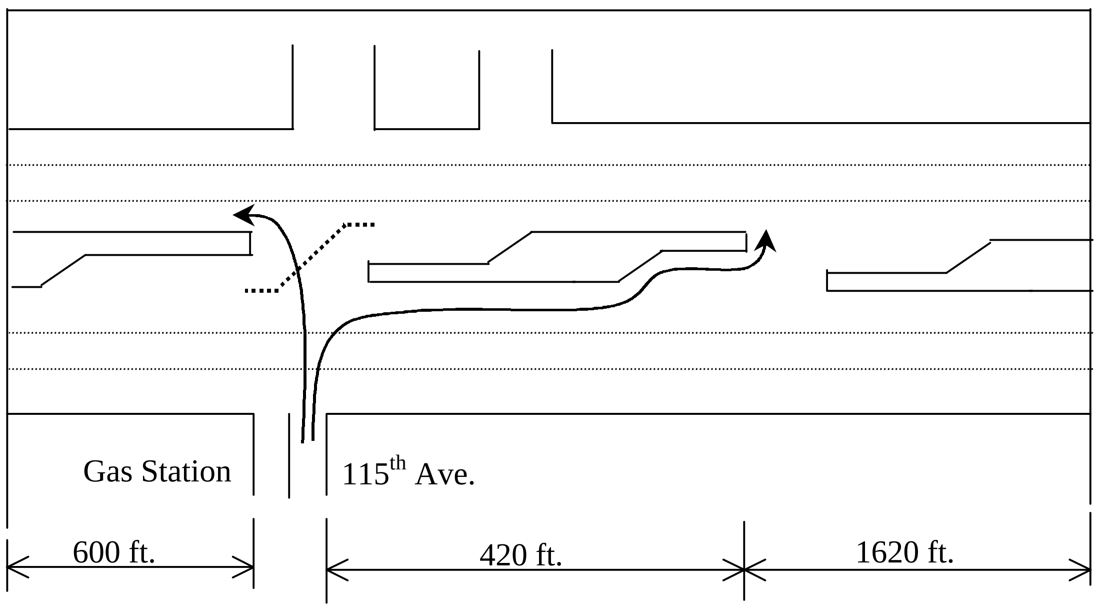


Figure 4.1 Geometric Layout of Study Site: US 19 @ 115th St.

Figure 4.2 A Photograph of the Study Site During Before Period

Figure 4.3 A Photograph of the Study Site During After Period

4.3 Data Collection and Reduction

Field data were collected for one week before and one week after the full median opening was changed to a directional median opening, by using the same methodology applied to the other study sites. Data collected with video cameras were reviewed and the following information was recorded:

(1) Waiting delay of DLT vehicles and right turn vehicles at the driveway (defined as t_{L1} and t_{R1}, respectively),

(2) Waiting delay of DLT vehicles at the full median opening and U-turn vehicles at the U-turn median opening (defined as t_{L2} and t_{R2}, respectively), and

(3) Running time of DLT vehicles crossing the major-road through lanes and RTUT vehicles crossing the weaving section (defined as t_{L3} and t_{R3}, respectively).

All traffic data were averaged based on five-minute intervals. A total of 18 hours of “before” data and 19 hours of “after” data were obtained and input into an Excel spreadsheet, including time, upstream through volume, downstream through volume, left turn egress volume and delay, RTUT volume and delay, and average running time of the RTUT movement in the weaving section.

4.4 Comparison of Weighted Average Delay and Weighted Average Travel Time

The primary operational difference between a full median opening and a directional median opening is that left turn egress from a driveway is replaced by a right turn followed by a U-turn. This results in additional travel distance for drivers who want to make a direct left turn out of a site onto the main roadway. Based on the analysis results of field data from eight sites described earlier, it was found that RTUT results in less delay and travel time than DLT under certain major road through volume conditions and left turn in volume conditions from the major roadway.

In this before-and-after analysis, data collected at the same site with the exact same geometric conditions except median type were used to compare operational performance of a full median opening vs. a directional median opening. For a full median opening, a driver who wants to make a left turn from the driveway has two options: either a DLT or a RTUT. Each DLT and RUTU movement was tracked to obtain the delay, travel time and volume information during the “before” period. To evaluate operational performance of a full median opening, Weighted Average Total Delay (WATD) and Weighted Average Total Travel time (WATT) were defined to combine the average delay and travel time of DLT and RTUT during the “before” period.

For a directional median opening, a driver who wants to make a left turn from the driveway has no choice but to make a RTUT. Each RTUT vehicle was tracked to obtain the average total delay and average total travel time. The WATD and WATT during the “after” period are equal to the average total delay and average total travel time of RTUT. The two Measures of Effectiveness (WATD and WATT) reflect the system performance of a full median opening and a directional median opening.

The weighted average total delay and travel time for a full median opening can be calculated in Eqs.4-1 and 4-2, respectively:

$$WATD_B = \frac{TD_L \times LTV + TD_{RU} \times RUV}{LTV + RUV} \tag{4-1}$$

$$WATT_B = \frac{TT_L \times LTV + TT_{RU} \times RUV}{LTV + RUV} \tag{4-2}$$

where,

$WATD_B$ - weighted average total delay during the “before” period,

$WATT_B$ - weighted average total travel time during the “before” period,

TD_L - average total delay for DLT,

TT_L - average total travel time for DLT,

TD_{RU} - average total delay for RTUT,

TT_{RU} - average total travel time for RTUT,

LTV - flow rate of DLT(vph), and

RUV - flow rate of RTUT(vph).

Weighted average total delay and travel time for a directional median opening can be calculated in Eqs.4-3 and 4-4, respectively:

$$WATD_A = TD_{RU} \tag{4-3}$$

$$WATT_A = TT_{RU} \tag{4-4}$$

where,

$WATD_A$ - weighted average total delay during the “after” period (sec./vehicle),

$WATT_A$ - weighted average travel time during the “after” period (sec/vehicle).

Field data were used to calculate the WATD and WATT for a full median opening and a directional median opening during the peak and non-peak hours. During the “before” period, nine hours of peak hour data and nine hours of non-peak hour data were collected in the field. Average traffic volumes are listed in Table 4.1. Upstream through traffic volume (TV1) and downstream through traffic volume (TV2) are 2,441 vph and 2,558 vph during peak hours, respectively, and 1,793 vph and 1,914 vph during non-peak hours. Eighteen percent of the vehicles were making a RTUT during the peak-hour and 13 percent of the vehicles were making a RTUT during non-peak hour. Left turn in volume from the major road (LTIN) was about 86 vph.

Table 4.1 Traffic Volumes for the Full Median Opening

Traffic Volume During the Before Period (vph)						
Period	Time	TV1	TV2	LTV	RUV	LTIN
Peak	9 hrs	2441	2558	83	18	85
Non-peak	9 hrs	1793	1914	72	11	86
Total	18 hrs	2151	2272	78	15	86

Note: TV1, TV2: Upstream and downstream through traffic volume,
LTV: Left turn volume from the driveway,
RUV: Right turn plus U-turn volume, and
LTIN: Left turn in volume from major road.

Table 4.2 lists the average total delay and average total travel time of DLT and RTUT during peak hours and non-peak hours. Based on volume, average total delay and travel time of DLT and RTUT, the WATD and WATT for the "before" period can be obtained by Eqs.4-1 and 4-2. WATD for the full median opening is 60.18 sec./veh and 39.92 sec./veh during the peak and non-peak hours, respectively. WATT for the full median opening is about 67.04 sec./veh and 46.12 sec./veh during the peak and non-peak hours, respectively.

Table 4.3 lists the traffic volume, average total delay and travel time of RTUT for a directional median opening during the "after" period. A total of 10 peak hours and 9 non-peak hours of data were collected in the field. The major road through traffic volume was very similar to the before period. There was no direct left turn egress in the "after" period. The WATD and WATT are equal to the average total delay (51.20 sec./veh during peak hours and 31.33 sec./veh during non-peak hours) and average total travel time (66.00 sec./veh during peak hours and 45.83 sec./veh during non-peak hours) of RTUT for a directional median opening.

Table 4.2 Delay and Travel Time for the Full Median Opening

Delay and Travel Time During Before Period (sec./vehicle)							
Period	Time	TD_L	TD_{RU}	TT_L	TT_{RU}	WATD	WATT
Peak	9 hrs	63.51	44.89	67.51	64.88	60.18	67.04
Non-peak	9 hrs	41.61	29.31	45.61	49.30	39.92	46.12
Total	18 hrs	53.78	33.10	57.78	53.09	50.49	57.04

Figures 4.5 and 4.6 illustrate the before-and-after comparison of weighted average total delay and weighted average total travel time, respectively. It was found that there was about a 15 percent reduction of WATD during peak hours and about 22 percent reduction of WATD during the non-peak hours. The WATT had no significant change during the before and after periods.

Table 4.3 Volume, Delay and Travel Time for the Directional Median Opening

Traffic Volume, Delay and Travel Time During After Period								
Period	Time	TV1	TV2	RUV	TD_{RU}	TT_{RU}	WATD	WATT
Peak	10 hrs	2423	2472	144	51.20	66.00	51.20	66.00
Non-Peak	9 hrs	1776	1944	96	31.33	45.83	31.33	45.83
Total	19 hrs	2100	2208	120	41.06	55.86	41.06	55.86

These findings of the case study indicated that replacing direct left turns with right turns followed by U-turns could significantly reduce the average total delay experienced by the left turning vehicles. The reduction in delay was more evident during non-peak periods than that of peak periods. However, there was no significant impact on the average total travel time depending on the left turn movement type.

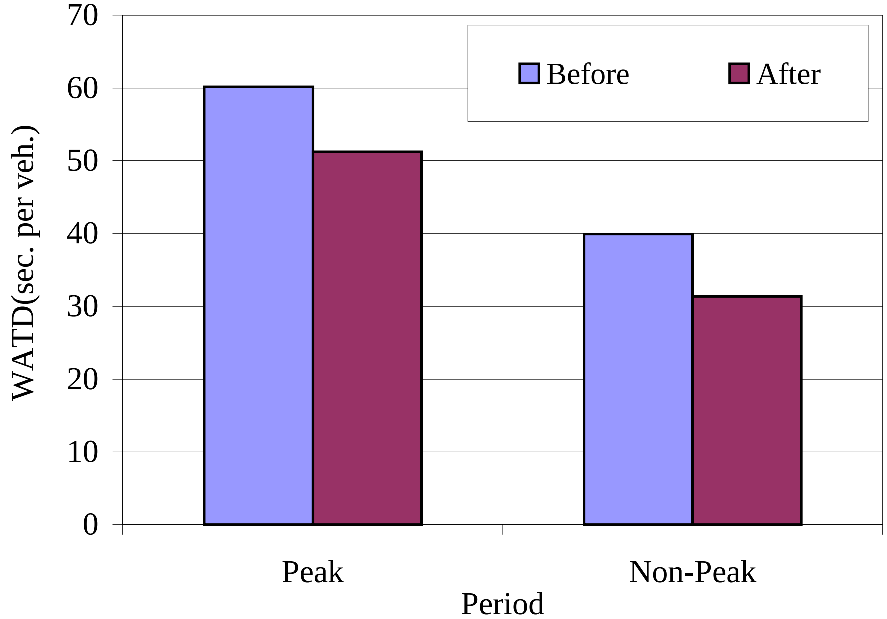


Figure 4.4 Before and After Comparison of WATD

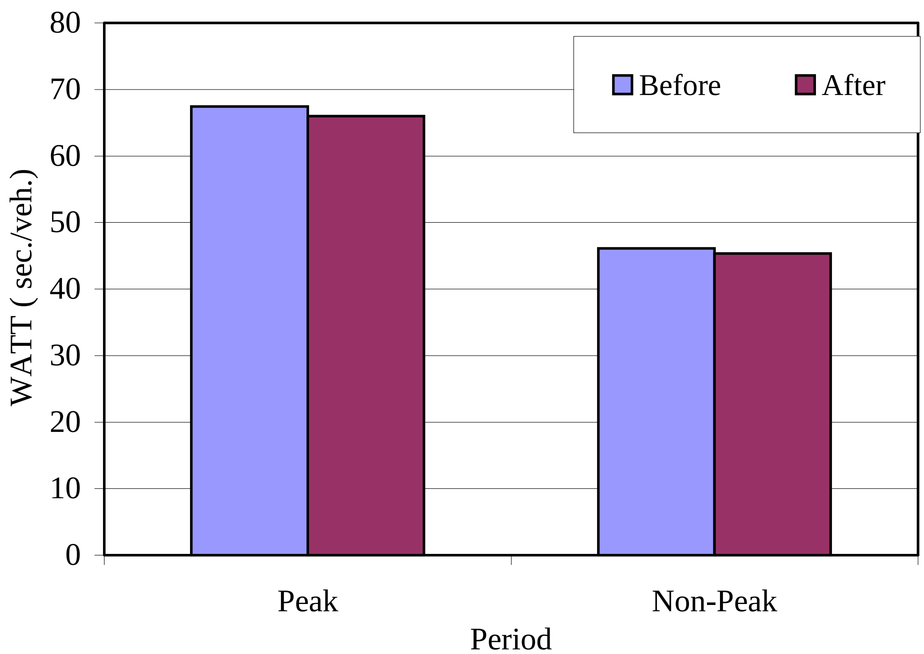


Figure 4.5 Before and After Comparison of WATT

4.5 Application and Calibration of Models

Empirical delay and travel time models developed in this study are based on field data collected at the eight sites on urban and suburban arterials, with traffic signal spacing of 2 miles or less. Through traffic on the urban or suburban road is interrupted by traffic signals. Distribution of available gaps in the major-street stream depends on total volume on the street, its directional distribution, and the degree and type of platoons in the traffic stream. However, gap sizes required by a DLT or a RTUT depend on driver characteristics such as eyesight, reaction time, age, and so on. These human factor variables cannot be incorporated into delay and travel time models. This is the main reason that the R^2 of delay and travel time model is so low.

Frequently, after a model is developed, it is validated by comparing estimates from the model with values measured in the field from an independent set of sites. A regression line fitted to the plot of points from field-measured versus model-estimated values will result in a line with a slope different from 45 degrees. The difference can be considered the relative accuracy of the model. Then dispersion of the points around the regression line can be considered the precision of the model. The measure of dispersion with which many analysts are familiar is the R^2 value. These statistics, based on field and predicted data, indicate the limitations of the models in predicting with great precision and accuracy. To calibrate the results predicted by the models, the delay and travel time models were used to estimate the average total delay and travel time of DLT and RUTU during the peak and non-peak hours using the average traffic volume data at site three. Based on the predicted average delay and travel time of DLT and RTUT, WATD and WATT can be calculated based on the definition.

As shown in Tables 4.4 and 4.5, WATD and WATT computed based on field data and predicted data are listed.

Table 4.4 Calibration of Delay Models by the Field Data

Period	Before (A Full Medina Opening)				After (A Directional Median Opening)			
	WATD (sec./vehicle)				WATD (sec./vehicle)			
	Field	Model	Difference	Error	Field	Model	Difference	Error
Peak	60.18	55.19	4.99	8.3%	51.20	48.83	2.37	4.6%
Non-Peak	39.92	27.74	12.19	30.5%	31.33	30.31	1.02	3.3%

Table 4.5 Calibration of Travel Time Models by the Field Data

Period	Before (A Full Median Opening)				After (A Directional Median Opening)			
	WATT				WATT			
	Field	Model	Difference	Error	Field	Model	Difference	Error
Peak	67.04	68.86	1.82	2.7%	66.00	64.86	1.14	1.7%
Non-Peak	46.12	41.73	4.38	9.5%	45.83	50.55	4.72	10. %

After computing the difference, it was found that most errors between the field data and model prediction were less than 10 percent. Only the WATD for a full median opening during non-peak hours has a relatively high error rate (30%). Results of model prediction suggest that there is a 12% reduction in WATD during the peak hours.

Comparison of field data and predicted results demonstrated that the delay and travel time model could predict reasonable results and assist transportation professionals in evaluating the relative impacts of median changes.

4.6 Summary

This before-and-after case study addressed operational effects of replacing a full median opening with a directional median opening in terms of average total delay and average travel time. In the case of the study site at US 19 and 115th St. intersection, which experienced more than 2400 vph traffic volume on the major street during peak periods, weighted average total delay was reduced by 15% after installing a directional median opening to prohibit direct left turns from the driveway. During the non-peak periods where the major street traffic volume was around 1700 – 1900 vph, the reduction in the average travel delay was 22%. No significant change was observed in the weighted average travel time during the before and after periods.

The delay and travel time models were calibrated by comparing the weighted average total delay and travel time computed based on field data and model prediction, which provided acceptable results. This demonstrated that the delay and travel time models could produce reasonably accurate results to compare operational effects of a directional median opening vs. a full median opening for a specific site. Engineering judgment is however required in applying the results of analysis to the actual situations, because individual sites may have different characteristics than those studied in this project.

5. OPERATIONAL EFFECTS THROUGH SIMULATION

Operational effects of DLT and RTUT obtained using computer simulation are discussed here under three categories: calibration results for DLT and RTUT, simulation results for each site and comparisons between the two movements, and development of the general simulation model and the results from the model.

5.1 Calibration Results of Site-Specific Models

The objective of model calibration is to make the outputs from the models as close as possible to the field values. Based on the literature review the state of the art in traffic simulation includes very few references on calibration methodologies. As indicated in "Computer Simulation & Modeling", calibration is difficult and remains one of the least developed areas in system simulation *(42)*. However, the quality of modeling is highly dependent on such quantitative details. Indeed, few references have talked about the failures and difficulties, which are normal in a subject like traffic operations where human factors are involved. Even though CORSIM has some implied calibrations due to the successful applications in traffic operation analysis for many years, it is not expected to reflect the real situation perfectly *(37)*.

The parameter calibration process of the model consists of systematically varying a number of the model parameters and comparing the selected MOEs with the field data until there is a reasonable correspondence between two sets of MOEs *(41)*. In this study, four parameters were used in the calibration of the site-specific models. They were delay of left-turn out vehicles at the driveway, delay of right-turn out vehicles at the driveway, delay of U-turn, and delay of left out at the median opening, which can be obtained from field observation. If the median opening is directional, not allowing left turns out, only two parameters, delay of right turn at driveway and delay of U-turn were used in the calibration. According to the literature review, when the difference between simulation data and field data is within 10%, it is considered acceptable.

The calibration results for the eight sites are shown in Table 5.1. These are the average of 12 simulation runs. Since the simulation values are very close to the field data, models can be expected to reasonably replicate the real situation at these eight sites.

Table 5.1 Comparison of Simulation Results and Field Data at each Site

Sites		1	2	3	4	5	6	7	8
Delay of DLT at driveway	Field	---	25	35	37	25	19	34	---
	Simulation	---	26.3	33.6	39.5	26.4	18.53	34.2	---
Delay of RT at driveway	Field	18	20	20	19	20	15	19	30
	Simulation	20.2	21.5	21.1	17.8	18.7	14.23	18.0	29.8
Delay of U-turn	Field	13	17	25	48	36	25	14	22
	Simulation	14.1	18.3	24.7	50.7	37.2	23.47	17.7	20.7
Delay of DLT at median	Field	---	15	23	19	16	17	11	---
	Simulation	---	16.2	22.3	20.6	18.3	18.02	11.2	---

5.2 Simulation Results of Eight Sites

Since the objective was to compare the performances of DLT and RTUT, simulation results were used to compare the delays and travel times of the two movements. The through traffic volumes on the major street were considered in 5 levels, ranging from 3000 vph to 7000 vph in increments of 1000 vph. Similarly, the volume from driveways was also considered in 5 levels: 50, 100, 150, 250, and 350 vph. The percentages of left-turn in volume and U-turn volume were assumed as unchanged irrespective of volumes.

5.2.1 Delay Comparisons

Simulation results for the DLT and RTUT delay at each of the eight sites were first estimated. In order to compare the delays of the two movements, the difference in delay times were obtained and presented in Table 5.2 through 5.7. Negative values show that delay of RTUT is less than DLT, demonstrating that, RTUT has an advantage over DLT at this volume combination. Accordingly, if the difference carries a positive sign, RTUT delay is greater than DLT and therefore RTUT has no advantage over DLT, under the given volume combination.

Table 5.2 Differences in Delays Between RTUT and DLT at Fowler/19th Street

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	0.4	-0.9	2.1	2.5	5
4000	-0.5	0.3	1.9	0.8	-3
5000	0.1	-1.8	2.3	0.1	3.1
6000	2.7	-2.8	-1.4	-4.3	-7.2
7000	-15.5	-24.8	-28	-50.5	-114.4

Table 5.3 Differences in Delay Between RTUT and DLT at US 19/116th Street

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	3.7	5.1	8.4	9.3	10.5
4000	8.7	6	7	1.3	-2.2
5000	-5.6	-7.9	-10.7	-16.2	-19.4
6000	-19.7	-23.9	-25.3	-25.3	-45.4
7000	-24.3	-29.3	-37	-66.1	-108.1

Table 5.4 Differences in Delay Between RTUT and DLT at US 19/Enterprise St.

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	9.3	8.9	8	7.9	6.5
4000	9.7	9.9	4.8	6.9	6.8
5000	7.2	15.5	17.8	19.3	-3.1
6000	-5.4	-8.7	-10.7	-16.4	-22.7
7000	-4.4	-11.9	-15.3	-18.3	-33.2

Table 5.5 Differences in Delay Between RTUT and DLT at US 19/Sunset St.

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	5.9	6.3	9.9	12.6	17
4000	2.8	4.1	6.3	9.2	14.3
5000	0.7	1.9	7.6	8.6	11.8
6000	1.7	7.9	0.3	-3.3	-9.5
7000	-5.2	-5.9	-6.5	-9.3	-15.9

Table 5.6 Differences in Delay Between RT+UT and DLT at Bruce B. Downs/V.A. Medical Center

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	5.9	6.3	9.9	12.6	17
4000	2.8	4.1	6.3	9.2	14.3
5000	0.7	1.9	7.6	8.6	11.8
6000	1.7	7.9	0.3	-3.3	-9.5
7000	-5.2	-5.9	-6.5	-9.3	-15.9

Table 5.7 Differences in Delay Between RTUT and DLT at Hillsborough/Golden

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	1.6	1.4	1.3	1.3	0.8
4000	0.3	2	0.4	1.5	-9.1
5000	-7.8	-8.9	-7.5	-12.3	-20
6000	-5.8	-6.5	-6.1	-9.6	-19
7000	-5.7	-9.2	-17	-20	-71.2

5.2.2 Comparisons of Total Travel Time

The total travel time for DLT is defined as the delay time at the driveway, plus the crossing time from driveway to the median, plus the delay time at the median. The total travel time for RTUT is defined as the delay time at the driveway, plus the weaving time from the driveway to the U-turn bay, plus the delay time at the U-turn bay, and finally the travel time from the U-turn bay back to the median opening at the driveway.

In order to compare the travel times of the two movements, the differences in travel times for DLT and RTUT are obtained and the results are given in Tables 5.8 through 5.13. When the values are negative, this shows that total travel time of RTUT is less than DLT, demonstrating that at this volume combination of volume from the major road and driveway, RTUT has an advantage over DLT. Alternatively, RTUT has no advantage over DLT, if the travel time difference takes a positive sign.

Table 5.8 Differences in Total Travel Times Between RTUT and DLT at Fowler/19th Street

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	19.9	18.6	21.6	22	24.5
4000	19	19.8	21.4	20.3	16.5
5000	19.6	17.7	21.8	19.6	22.6
6000	22.2	16.7	18.1	15.2	12.3
7000	4	-5.3	-8.5	-31	-94.9

Table 5.9 Differences in Total Travel Times Between RTUT and DLT at US 19/116th Street

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	18.2	19.6	22.9	23.8	25
4000	23.2	20.5	21.5	15.8	12.3
5000	8.9	6.6	3.8	-1.7	-4.9
6000	-5.2	-9.4	-10.8	-10.8	-30.9
7000	-9.8	-14.8	-22.5	-51.6	-93.6

Table 5.10 Differences in Total Travel Times Between RTUT and DLT at US 19/ Enterprise St.

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	23.8	23.4	22.5	22.4	21
4000	24.2	24.4	19.3	21.4	21.3
5000	21.7	30	32.3	33.8	11.4
6000	9.1	5.8	3.8	-1.9	-8.2
7000	10.1	2.6	-0.8	-3.8	-18.7

Table 5.11 Differences in Total Travel Times Between RTUT and DLT at US 19/Sunset St.

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	22.4	22.8	26.4	29.1	33.5
4000	19.3	20.6	22.8	25.7	30.8
5000	17.2	18.4	24.1	25.1	28.3
6000	18.2	24.4	16.8	13.2	7
7000	11.3	10.6	10	7.2	0.6

Table 5.12 Differences in Total Travel Times Between RTUT and DLT at Bruce B. Downs/V.A. Medical Center

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	30.5	30.6	31.3	30	28.5
4000	34.8	35.1	35.4	33.5	35.9
5000	35.1	33.9	36.6	35.3	35.4
6000	33.2	33.9	30.1	30.2	22.3
7000	24	23.2	19.6	13.1	3

Table 5.13 Total Travel Times of RT+UT at Fowler Avenue/52nd Street (sec/veh)

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	48.5	49.6	52.3	58.3	65.5
4000	64.5	68.5	72.8	78.8	90.7
5000	70.3	72.3	78.8	94.6	102.5
6000	73.5	79.2	94.3	102.9	114.7
7000	79.3	82.8	101.3	112.5	123.4

5.3 General Simulation Model

In addition to the site-specific models, a General Simulation Model (GSM), which can be used to simulate any arterial with 6 or 8 lanes, was also developed. Unlike the site-specific models, the GSM is adaptable to the changes in geometric conditions such as weaving distance, length of turning bay, number of lanes, and the distances to the signalized intersections from the subject driveway. The GSM is developed based on the results of the site-specific simulation models of the eight sites.

The geometric conditions of GSM were set forth to be the average characteristic of the sites where the data were collected. Accordingly, the major roadway had six lanes with a full median opening and the driveway had two lanes exclusively used by left out traffic and right turn movement. The weaving distance from the driveway to the U-turn bay was set to be 600 feet, the average of the weaving distances for the eight sites. The distance from the upstream-signalized intersection to the driveway was 1000 feet and the distance from the downstream-signalized intersection to the driveway was 2000 feet. The signal timing plans of the intersections at both ends were kept in the optimization status. For each scenario of through volume (from 3000 to 7000), TRANSYT-7F was used to optimize the timing plans for the two intersections and then optimized timing plans were used in the network.

Similar to the site-specific models, the parameter calibration process was conducted for the GSM. The average parameter values for the eight sites were used to compare the simulation outputs. The calibration results for the general simulation model are shown in Table 5.14, which shows that the simulation data after calibration were closer to the field data than the data before calibration. The before calibration denotes the conditions under which all parameters used are the default values embedded in CORSIM. The differences of all parameters between field observations and calibrated simulation models meet the requirement of being within 10%. Therefore they can be expected to simulate the DLT and RTUT movements with reasonable accuracy.

Table 5.14 Comparison of Simulation Data & Field Data For GSM

Parameters	Field data	Simulation data	
		Before calibration	After calibration
Delay of DLT at driveway	29.2	33.19	30.26
Delay of RT at driveway	20.1	17.69	20.96
Delay of DLT at median	15.2	20.12	16.72
Delay of U-turn (sec/veh)	22.5	24.87	20.42

The delays and total travel time of DLT and RTUT were obtained using the calibrated GSM. Ten simulation runs were made for each scenario and each run needs 7200 seconds of simulation time. There were five levels of through traffic volume and five levels of driveway volume, resulting in a total of 250 simulation runs.

Delay

The delays defined in CORSIM are the stop delay plus travel delay. The travel delay is the difference between the real travel time on the link and calculated travel time based on free flow speed. The delays of DLT and RTUT produced from simulation are shown in Tables of 5.15 and 5.16. The differences in delays between DLT and RTUT for general simulation model are shown in Table 5.17.

Table 5.15 Delays of DLT Produced From GSM (sec/veh)

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	17.8	18.1	21.4	24.3	26.8
4000	28.5	29.6	34.2	38.5	42.3
5000	35.5	37.4	44.8	57.8	65.3
6000	51	56.8	67.1	71.9	75.4
7000	64.4	72.8	89.4	93.6	115.6

Table 5.16 Delays of RTUT Produced From GSM (sec/veh)

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	21	24.6	25.3	28.4	32.3
4000	30.4	31.7	36.3	40	43.2
5000	37.6	38.9	43.2	48.9	53.8
6000	44.4	46.3	50.5	52.7	62.6
7000	58.3	62.7	63.6	68.8	85.4

Table 5.17 Differences in Delays Between RTUT and DLT Produced From GSM (sec/veh)

Main street volume (vph)	Driveway volume (vph)				
	50	100	150	250	350
3000	3.2	6.5	3.9	4.1	5.5
4000	1.9	2.1	2.1	1.5	0.9
5000	2.1	1.5	-1.6	-8.9	-11.5
6000	-6.6	-10.5	-16.6	-19.2	-12.8
7000	-6.1	-10.1	-25.8	-24.8	-30.2

From Table 5.17, it can be observed that when through volume on main road reaches 5000 vph and driveway volume reaches 150 vph, the delay of RTUT is less than DLT. In order to clearly illustrate the relationship, two groups of curves were developed as shown in Figure 5.1. One group of curves represents the delay of DLT and the other represents the delays of RTUT. Within a group, each curve represents different level of driveway volume from 50 to 350 vph. According to the figure, when traffic volume on the major road is very low, the delay of DLT is lower than that of RTUT. With the increase of the volume, these two curves gradually come across and finally the curves of DLT reaches above the curves of RTUT. The volumes corresponding to the break-even points of the delays could also be obtained using the figure. It can be seen that with increase of the driveway volume, the break points move toward the lower level of through traffic volume.

Total Travel Time and Comparison

The total travel time defined for DLT in CORSIM is the delay time at the driveway, plus the crossing time from the driveway to the median, plus the delay time at the median. The total travel time for RTUT is defined as the delay time at the driveway, plus the weaving time from the driveway to the U-turn bay, plus the delay time at the U-turn bay, and finally the travel time from the U-turn bay back to the median opening at the driveway. Based on the simulation runs, the difference between total travel time and the delay for DLT and RTUT was found to range from 4.5 to 5.5 seconds and from 22.8 to 25.6 seconds, depending on the traffic demand. The total travel times for DLT and RTUT using GSM are shown in Tables 5.18 and Table 5.19, respectively.

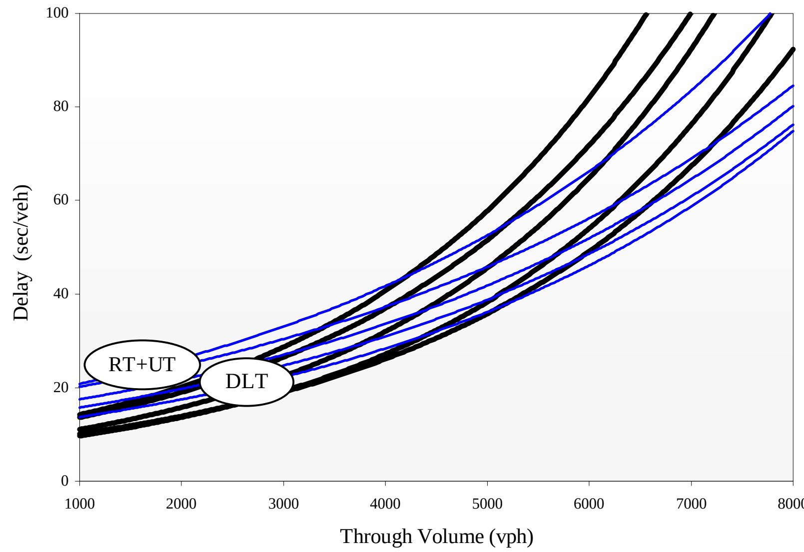


Figure 5.1 Comparison of Delay Between RTUT and DLT Based on GSM

Table 5.18 Total Travel Times of DLT Produced From GSM (sec/veh)

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	22.8	23.1	26.4	29.3	31.8
4000	33.5	34.6	39.2	43.5	47.3
5000	40.5	42.4	49.8	62.8	70.3
6000	56	61.8	72.1	76.9	80.4
7000	69.4	77.8	94.4	98.6	120.6

Table 5.19 Total Travel Times of RTUT Produced From GSM (sec/veh)

Main street volume (vph) (both directions)	Driveway volume (vph)				
	50	100	150	250	350
3000	45.6	49.2	49.9	53	56.9
4000	55	56.3	60.9	64.6	67.8
5000	62.2	63.5	67.8	73.5	78.4
6000	69	70.9	75.1	77.3	87.2
7000	82.9	87.3	88.2	93.4	110

The differences in total travel times between RTUT and DLT based on the results from general simulation model is given in Table 5.20, which shows that when through volume reaches 7000 vph and driveway volume is 150 vph, the total travel times of RTUT is less than that of DLT.

Table 5.20 Differences in Total Travel Times Between RTUT and DLT Produced From GSM (sec/veh)

Main street volume (vph) (both	Driveway volume (vph)				
	50	100	150	250	350
3000	22.8	26.1	23.5	23.7	25.1
4000	21.5	21.7	21.7	21.1	20.5
5000	21.7	21.1	18	10.7	8.1
6000	13	9.1	3	0.4	6.8
7000	13.5	9.5	-6.2	-5.2	-10.6

Groups of curves representing the DLT and RTUT from which the break points can be found are graphically illustrated in Figure 5.2. When the volume of the driveway is low, the break point occurs at high major road volumes. With the increase of the driveway volume, the break point moves towards lower major road volumes. It can also be seen that the increase of travel time for RTUT is not as sharp as DLT.

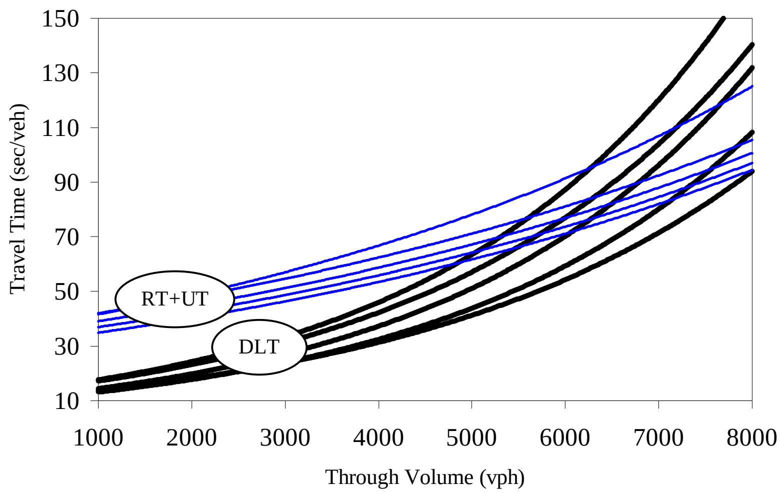


Figure 5.2 Comparison of Total Travels Time Between RTUT and DLT using GST

6. SUMMARY AND CONCLUSIONS

6.1 Summary

Transportation engineers and planners have used access management to improve operational and safety conditions of the road system. One of the objectives of access management is to reduce the number of conflict points. In particular, access management actions often seek to minimize direct left turn (DLT) movements from driveways as they generate many conflict points and can increase the incidence and severity of traffic crashes. Medians are used to replace DLT movements in some areas with right turn movements followed by U-turns (RTUT).

This report is one of three reports that evaluated the safety and traffic operational effects of direct left turns versus right turns followed by U-turns from driveways or side streets. This research focused on evaluating the traffic operational impacts of RTUT and DLT using both empirical modeling and computer-based simulation. The primary objectives of this part of the study were to explore methodologies for evaluating the operational effects of U-turns as alternatives to direct left turns and to provide information on the potential impacts of these alternatives under various conditions.

To achieve this, a full-scale data collection was conducted in Tampa Bay area, which involved 10 sites collecting a total of more than 300 hours of traffic data video taped to evaluate delay, travel time, and other issues. Delay models were developed based on data sets on DLT and RTUT delays as a function of major-road through-traffic flow rate, left-turn-in flow rate from the major road, flow rate of DLT/RTUT, and split. Curves were developed based on regression results depicting operational differences between making a DLT versus making a RTUT. The curves demonstrated the point at which a driver making a right turn and U-turn from a driveway experiences less delay than a driver attempting to make a direct left turn through a median opening onto a major road. The average running time of a RTUT vehicle was also recorded at each site with a different weaving distance. The analysis indicated that there was a linear relationship between the length of weaving segment and average running time. In other words, the longer the weaving distance was, longer the average running time was for weaving distances from 300 to 1000 ft. This was incorporated into a travel time model that was used to compare the average travel time for the two movements studied in this project to assure that the right turn followed by U-turn would not experience longer travel times.

Potential effects of both DLT and RTUT movements on the speed of through traffic were also investigated. The data indicated that the average reduction in traffic speed for the RTUT was very small. The average speed reduction of through traffic was found to be higher for DLT than the reduction resulting from RTUT. This may be due to the fact that the impact from crossing vehicles is more significant than that from merging and weaving vehicles.

Drivers' selection of RTUT or DLT may be affected by some traffic characteristics such as through-traffic volume, left-turn-in volume, and so on. A ratio model was developed to estimate how many drivers would like to make a RTUT rather than a DLT when a suitable downstream U-turn median opening is provided. The findings indicated that the left turn in volume and major road through traffic volume had significant impacts towards increasing the amount of RTUT. Additionally, down stream through traffic flow rate was more influential than the upstream flow rate in determining the amount of RTUT.

In addition, a before-and-after analysis was completed as a case study where a full median opening on US 19 and 115th St. intersection in Pinellas County in Florida was replaced with a directional median opening during data collection for this project. The delay for the driveway left turning vehicles were found to be reduced by 15% -22% when direct left turns were forced to make right turns followed by U-turns.

Simulation models using CORSIM were established to describe the two-step left turn out movements and U-turns. Moreover, the site-specific models were well calibrated to simulate DLT and RTUT and produced satisfactory results. The general simulation model was based on the results obtained from the eight site-specific models. This model can be useful for simulating the operational performance of DLT and RTUT movements for arterials with six or eight lanes. In addition, regression models for delay and travel time for DLT and RTUT were developed and these models can be used to evaluate the relative operational impacts of replacing a DLT with a RTUT under various traffic conditions.

6.2 Conclusions

The methodology used to quantify the operational effects of DLT and RTUT demonstrated that U-turns could have better operational performance than DLT under higher traffic volume conditions. Break-even point for the determination of the higher volume could be estimated by using the models developed in this study. The outcome implies that restrictive median designs would provide more efficient traffic flow than full median openings under certain traffic conditions. The following conclusions were reached as a result of this study:

(1) The curves based on the delay models properly represent the operational impacts of direct left turns and right turns followed by U-turns and demonstrate at what point a RTUT experience less delay than a DLT from an operational perspective; There are no significant impacts on through traffic speed by either movement because these two movements have no impact on the platoon speed, they only affect the speed of random arrivals between platoons;Directional median openings may provide more efficient traffic flow than full median openings when the major-road through-traffic flow rate is more than 4,000 vph in both directions and the left-turn-in flow rate from the major-road is over 150 vph;The percentage of RTUT movements increases with major-road through-traffic flow rate and left-turn-in flow rate from major-road;

(5) The average running time of a vehicle making a RTUT from a driveway has a linear relationship with the length of weaving segment or the running time increases as the weaving distance gets longer;

(6) The average weaving speed of RTUT linearly increases with the increase of weaving distance; and

(7) The before and after study indicated that there was about 15-22% less delay for the drivers turning left from a driveway after the median opening was replaced with a directional median opening, forcing them to make a RTUT at a median opening 420 feet downstream, in place of a DLT.

Based on the simulation results from the eight sites, it was confirmed that RTUT might not pose as much delay or total travel time as DLT from a driveway under higher traffic volumes.

Results, information, and analysis provided by this study could be helpful for retrofit decisions. Delay and travel time results provide a tool to help address public concerns related to the operational impacts of U-turns and would be particularly helpful in identifying the circumstances where the right turn followed by U-turn takes less time than the direct left turn.

Left-turn-in volume was found to have a dramatic impact on the delay of left turn out from driveways. This, in turn, indicates that left-turn-out vehicles also have an impact on the left-turn-in vehicles. In practice, the left-turn-out drivers may not always yield to the left-turn-in vehicles resulting in increased delay for left-turn-in drivers. Usually, business owners care more about the ability of motorists to make left-turns into their business, than left-turns out.

The delay of vehicles at a side street or driveway is not what should drive the design of the median. To guide the decision as to what type of median opening should be allowed, safety has the first priority, followed by the operational efficiency of the highway, and then the driveway delay.

Some issues were not addressed in this study including operational effects of U-turns at signalized intersections, the location of U-turn median openings, impacts of coordination of upstream and downstream signal time, optimum weaving distance in terms of safety and operations, and operational effects of truck U-turns. Further research is needed in such areas.

In general, appropriate weaving distances should not be too long (approximately 1000 ft) or too short (approximately 500 ft) based on the field observation of operational performance of RTUT maneuvers. The U-turn median opening must not be located in the functional area of the downstream-signalized intersection. In practice, it was found that RTUT maneuvers would be blocked by major-road through traffic during the peak hours if through traffic queuing length reaches the U-turn median opening.

Figure 6.1 A Suggested New Sign for RTUT

In high volume driveways or street intersections, it may be helpful to inform drivers of the down stream U-turn location. A sample sign is shown in Figure 6.1. A sign similar to this may help RTUT drivers make an early lane change and a desirable weaving maneuver. However, more detailed study will be required before implementing such a sign.

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