OPERATIONAL EVALUATION OF RIGHT TURNS FOLLOWED BY U-TURNS AT SIGNALIZED INTERSECTIONS (6 OR MORE LANES) AS AN ALTERNATIVE TO DIRECT LEFT TURNS

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16. Abstract

Florida uses restrictive medians and directional median openings to manage left turns and crossing movements on multilane highways. Left turn egress movements from driveways or side streets would be replaced by turning right onto the major road and then making U-turns at downstream median openings or signalized intersections. One of the public concerns regarding this treatment is that drivers making a right-turn followed by a U-turn (RTUT) could experience much longer delay and travel time than those who would otherwise make a direct left turn (DLT). This study analyzed the operational effects of a widely used access management treatment: U-turns at downstream signalized intersections as an alternative to direct left turns. Data were collected on eight selected street segments in the Tampa Bay area in Florida. A total of more than 300 hours of traffic data were collected. Empirical models were developed based on the collected field data to quantitatively evaluate the operational impacts. Delay and travel time models provided a tool to help address public concerns related to the operational impacts of U-turns and would be particularly helpful in identifying the circumstances in which the RTUT takes less time than the DLT. Additionally, a ratio model was developed to estimate how many drivers would like to make a RTUT instead of a DLT under specific traffic volume and roadway geometric conditions. The findings of this study are helpful in providing local and state transportation agencies with recommendations for the design and selection of median treatments in six to eight lanes urban or suburban arterial roads.

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ABSTRACT

Increasingly, Florida uses restrictive medians and directional median openings on multilane highways to manage left turn egress maneuvers from driveways or side streets. By installing nontraversable medians and replacing full median openings with directional median openings at various locations, Florida is limiting unsignalized median openings to left turns from the major arterial; hence, direct left turn egress maneuvers (DLT) from driveways or side streets would be replaced by making a right-turn followed by a U-turn (RTUT) at downstream median openings or signalized intersections.

This report is one of the reports that evaluate the safety and operational effects of a widely used access management treatment: U-turns at downstream signalized intersections as alternatives to direct left turns. The focus of this report is on the operational effects of U-turns at signalized intersection, on six to eight lanes urban or suburban arterials. The primary objectives of this study were to explore methodologies for evaluating the operational effects of U-turns at signalized intersections as alternatives to direct left turns; and to provide information on the potential operational impacts of these alternatives under various conditions.

To achieve these objectives, field studies were conducted at eight roadway segments in the Tampa Bay area in Florida. Over 300 hours of traffic data were collected using video cameras. While reviewing videotapes, each vehicle coming from the driveway making DLT or RTUT was tracked. Delay and travel time for each vehicle making DLT or RTUT were recorded. Other information gathered in the field study included traffic volumes, signal parameters, and roadway geometrics.

Delay and travel time models were developed based on collected field data. The delay and travel time of DLT and RTUT were determined as a function of conflicting volumes, signalization conditions, and roadway geometrics. Curves

were developed based on regression results depicting operational differences between vehicles making a DLT versus those making a RTUT. The curves demonstrated the break points at which drivers making a right turn followed by a U-turn at downstream signalized intersection experienced less delay and travel time than those attempting to make a direct left turn through a median opening onto a major road.

In this project, the operations of two widely used U-turn approaches: U-turn at median openings in advance of signalized intersections and U-turn at signalized intersection, were also evaluated by comparing the delay and travel time models developed in this study with those from the USF 2001 study (Methodology to Quantify the Effects of Access Management Treatments on Roadway Operations and Safety). Based on the comparison, it is clear that providing RTUT at median openings in advance of signalized intersections will experience less delay and travel time as compared with the condition where U-turns are accommodated at signalized intersections.

Drivers' selection of RTUT or DLT may be affected by traffic characteristics such as through-traffic volume, left-turn-in volume, and others. In addition, field measurement found that drivers' choice of RTUT is also affected by the offset distance from driveway to downstream signalized intersections. A binary logistic regression model was developed to estimate the percentage of the drivers would like to make a RTUT rather than a DLT under some particular traffic and roadway geometric conditions. The findings indicate that the left-turn volume off major road onto minor road had significant impacts towards increasing the amount of RTUT. Additionally, the regression model developed in this study also indicated that fewer drivers would select RTUT when the offset distance from driveway to downstream signalized intersection is relatively long.

In order to estimate the effects of U-turns on signalized intersection capacity, a second-degree polynomial regression model was developed to determine the

relationship between the average queue discharge time for each vehicle and the varying percentages of U-turning vehicles in the left-turn traffic stream. Adjustment factors for varying percentages of U-turning vehicles on left-turn saturation flow rate were also developed by using this model. The results could be directly used in estimating the capacity reduction due to the presence of U-turning vehicles at a signalized intersection.

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1 INTRODUCTION

1.1 Background

In the past few decades, more and more states and transportation agencies came to realize the importance of managing roadway access in the state roadway system. In 1979, the nation's first systemwide comprehensive access management program was adopted in Colorado. In 1988, the Florida Legislature adopted the State Highway System Access Management Act, Statutes 335.18, which was considered as an important legal foundation of Florida statewide access management program. In 2003, the Transportation Research Board published the first national access management manual. Since 1993, there have been five national access management conferences (USDOT/FHS, 1993, 1996, 1998, 2000 and 2002). Recently, several NCHRP projects were established to conduct comprehensive research in this area. 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).

Access management was defined as the systematic control of the location, spacing, design, and operation of driveways, median openings, interchanges, and street connections to a roadway *(1)*. Access Management 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 benefits of access management include improved safety, improved traffic flow and fuel economy, increased capacity and reduced delay and vehicle emissions *(2)*.

As indicated in the "Access Management Manual", one of the major principles of access management is to use nontraversable medians to manage left-turn maneuvers. Left-turn

movements have been considered as one of the major resources of traffic operations and safety problems in multilane highways. Past studies indicated that left-turn maneuvers increase delays, conflicts, and crashes, and they reduce capacity and mobility in the major traffic. For example, as mentioned in Access Management Manual, a total of about 74% of access-related crashes involve left turning vehicles.

Many states and transportation agencies have started using nontraversable medians in the state highway system. Since 1993, the FDOT mandated that all new or reconstructed multi-lane arterials with design speeds over 40 mph be designed with restrictive medians *(3)*. In addition, Florida uses directional median openings to manage left-turns and crossing maneuvers *(4)*. By closing existing median openings on some major arterials or replacing them with directional median openings, Florida prohibits direct left-turn exits onto major arterials. Left turn egress movements would be replaced by turning right onto the arterial road and then making U-turns at downstream median openings or signalized intersections *(5) (6)*.

Replacing full median openings with directional median openings has been found to substantially reduce crash rate *(7) (8) (9)*. In practice, however, to close an existing median opening could be very sensitive and is difficult to be handled. Some business owners believe that the median closure would have some adverse impacts on their business. In addition, some people often oppose being directed to make a right-turn followed by a U-turn due to the perception that it may result in much longer delay and travel time than a direct left turn or a believe that U-turns are unsafe.

Several studies have been conducted in Florida and nationally concerning the economic impacts of nontraversable medians. The results generally indicated that median projects have little overall adverse impact on business activity *(10)*. In addition, the project performed by the University of South Florida and sponsored by the FDOT in 2001

(Methodology to Quantify the Effects of Access Management Treatments on Roadway Operations and Safety) provided very useful information on the access management treatment – *Right-Turn followed by U-turn at Median Opening as an Alternative to the Direct Left Turn from Driveways and Side Streets (11) (12) (13)*. The study took three basic approaches in evaluating this issue and involved huge amount of field data collection where extensive data were gathered at several appropriate locations using video cameras followed by a lengthy data reduction process in the lab. This project proved that at high major road through-traffic volume levels, direct left-turns resulted in higher traffic conflicts, stop delay and travel time as compared with right turns followed by U-turns.

The focus of the past FDOT project (*Methodology to Quantify the Effects of Access Management Treatments on Roadway Operations and Safety*), which was conducted by the University of South Florida in 2001 (USF 2001 study), was on safety and operational effects of U-turns at median openings on six to eight lanes urban or suburban arterials. In the real world, however, there are many other conditions where U-turn is accommodated such as at signalized intersections. Little documentation is available concerning this particular situation; and as a consequence, the operational effects of U-turns at signalized intersections are still not clear. In addition, there are many arguments surrounding the selection of specific U-turn locations, including providing U-turns in advance of signalized intersections, U-turns at signalized intersections, or U-turns after signalized intersections. Currently there is no widely accepted procedure to compare the operational performance of these different approaches. With the increasingly installed restrictive medians along multilane arterials in Florida and nationally, it is becoming more and more important to conduct an extensive study to address these issues.

1.2 Research Statement

Practically, drivers usually have two alternatives when making left turns from a driveway or side street: (1) making direct left-turns from the driveway onto the arterial, (2) making right turns followed by U-turns at downstream median opening or signalized intersection. The USF 2001 study has proved that at high through-traffic volume level, direct left-turns resulted in higher traffic conflicts, stop delay and travel time as compared with right turns followed by U-turns at downstream median opening. In practice, however, there are many other conditions where U-turns could be accommodated at downstream signalized intersections. As compared with U-turns at median opening, there are three new issues that need to be considered concerning U-turns at signalized intersections:

(1) In addition to the variables such as major road through-traffic volume, driveway volume, left-turn-in volume from major road, new factors should be considered when estimating the effects of signalized intersection on operations of RTUT movement, such as signal timing, signal control type, "right-on-red" from side street, and the offset distance from the driveway to signalized intersection, etc;

(2) U-turn maneuvers may have some adverse impacts on signalized intersection capacity. In practice, U-turning vehicles generally have longer discharge headway and start up lost time as compared with left-turns. Currently, there is no widely accepted procedure for estimating the effects of U-turning vehicles on signalized intersection capacity. In *Highway Capacity Manual* (HCM 2000), U-turns are treated as left-turns when estimating saturation flow rate. However, the operational effects of these two maneuvers are different; and

(3) Drivers' selection of RTUT is another issue that needs to be considered. The USF 2001 study indicated that more drivers make right turns followed by U-turns at downstream median openings instead of direct left turns from driveway when there was relatively high left-turn-in flow rate (> 200 vph) and major-road

through-traffic flow rate (>4000 vph in both directions). When considering U-turns at signalized intersections, however, drivers' choice behavior may be different from the condition where U-turn is accommodated at median openings.

Little documentation is available concerning the operational effects of providing U-turns at downstream signalized intersections as an alternative to direct left turns. Past studies evaluating operational effects of U-turns have not focused on the particular situation where U-turns are accommodated at downstream signalized intersections. In addition, transportation engineers generally rely on broad or subjective methods to analyze the operational performance of this specific treatment. Very few field data are available to substantiate the potential operational benefits of this technique. This is also one of the reasons why this treatment is still very controversial in practice. In order to address public concerns and to analyze the potential cost and benefits of this treatment, an extensive study based on field observation is needed.

In this study, field experiments were conducted to collect data at eight selected sites in the Tampa Bay area in Florida. Delay, travel time and percentage of drivers choosing right turn followed by U-turn at downstream signalized intersections rather than a direct left turn were used to quantify the operational effects of this specific median treatment. The effects of U-turns on signalized intersection capacity were estimated by applying the U-turn adjustment factors on left-turn saturation flow rate. The analysis results were compared with the results of the USF 2001 study, such that the operations of two U-turn treatments can be compared. The research results can be directly applied to evaluate the operational effects of median treatments such as installing restrictive median, closing existing median openings, and replacing a full median opening with a directional median opening.

1.3 Research Purposes and Objectives

The primary purpose of this project was to conduct a detailed evaluation and investigation on a widely used access management technique: right-turns followed by U-turns at signalized intersections as an alternative to direct left turns from driveways.

This research took two main approaches to evaluate this specific access management technique including operational analysis and safety analysis. Operational effects of right turn followed by U-turn at signalized intersection were quantified in this report through field studies and data collection. Empirical models concerning delay, travel time and driver's selection of RTUT was developed using collected field data. More specifically, the objective consists of the following parts:

(1) To determine under what volume conditions (major-road, left-turn-in, and driveway) would DLT have more delay or travel time as compared to RTUT;

(2) To estimate delay for RTUT at signalized intersections as a function of conflicting major and minor-road flow rates and signalization conditions;

(3) To estimate delay for DLT as a function of conflicting major and minor-road flow rates;

(4) To determine under what roadway traffic and geometric conditions would drivers using RTUT instead of DLT;

(5) To evaluate the effects of U-turns on signalized intersection capacity; and

(6) To compare the operational performance of two widely used U-turn approaches: U-turns at a median opening in advance of signalized intersection and U-turns at signalized intersection.

1.4 Outline of the Report

This report consists of six chapters. Chapter 1 provides a brief introduction of the research. Chapter 2 describes a summary of past studies in this area. Chapter 3 explains the methodology employed in achieving the previously mentioned objectives, which consists of three subsections: delay and travel time models, driver's selection of RTUT or DLT and U-turn effects on signalized intersection capacity. Chapter 4 focuses on the data collection and the data reduction procedure. Analysis results and research findings are presented in Chapter 5. Finally, Chapter 6 provides summary, conclusions and recommendations of this research.

2 LITERATURE REVIEW

2.1 General

In order to determine the operational effects of a widely used median treatment: providing U-turns at downstream signalized intersections as an alternative to direct left turns, extensive work was conducted to search current rules and regulations, design standards, policies and state of practice in Florida and nationally. In addition, past studies and reports related to this topic were also searched and reviewed. Generally, the references can be categorized into three parts: current rules and regulations, delay and travel time models, and operational effects of U-turns.

2.2 Current Rules and Regulations in Florida

Florida is heavily encouraging restrictive medians on its higher designed at-grade arterial roadways *(14)*. The 1993 Multi-lane Facilities Median Policy required that all new or reconstructed multilane highways with a design speed over 40 mph be designed with a restrictive median *(15)*. It also directs designers to find ways to use restrictive medians in all multi-lane projects, even those below the 40 mph design speed. One of the major purposes of installing restrictive medians is to eliminate left turn movements. By closing existing full median openings on some major arterials or replacing them with directional median openings that allow only U-turn movements or left-turn ingress movements, the left-turn exits onto major arterials would be prohibited and these left-turn movements would be made by turning right onto the arterial road and then making U-turns at downstream median opening or signalized intersections.

2.3 Delay and Travel Time Models

Delay and travel time are important measures of effectiveness (MOEs) of traffic operations. In the real world, people often opposed making a right turn followed by a U-turn because many of them generally believe that vehicles making a RTUT will experience much longer delay and travel time than those would otherwise make a DLT. Past studies documenting the operational effects of U-turns have not focused on the delay and travel time of right turns followed by U-turns as a complete procedure, especially for the specific condition where U-turns are accommodated at downstream signalized intersections.

2.3.1 Delay Models at Signalized Intersection

Delay is an important parameter that is used in estimating the level of service of signalized intersections. In addition, delay is a measure that most directly relates the driver's experience, in that it describes the amount of time consumed in traversing the intersection. There are many different ways to define delay. As presented in *Traffic Engineering (Second Edition) (16)*, the most frequently used forms of delay are defined below:

a. *Stopped Time Delay*: Stopped time delay is defined as the time a vehicle is stopped while waiting to pass through the intersection.

b. *Approach Delay*: Approach delay includes stopped time, but also includes the time lost when a vehicle decelerates from its ambient speed to a stop, as well as while accelerating from the stop back to its ambient speed. Sometimes it is very difficult to measure decelerate delay in the field without sophisticated tracking equipment.

c. *Travel Time Delay*: Travel time delay is defined as the difference between the driver's desired total time to traverse the intersection and the actual time required to traverse it.

d. *Time-in-Queue Delay*: Time-in-Queue delay is the total time from a vehicle joining an intersection queue to its discharge across the stop-line or curb-line.

Several studies have been conducted to estimate delay at signalized intersections. Among them, the most often quoted model is perhaps the Webster model. In this model *(17)*, Webster estimated delay at isolated traffic signals as a sum of uniform delay (d_u) and random delay (d_r). Uniform delay is the delay at signalized intersection assuming uniform arrival rate. As indicated in HCM 2000, the uniform delay can be expressed as:

$$d_1 = \frac{0.5C\left(1-\frac{g}{C}\right)^2}{1-\left[\min(1,X)\frac{g}{C}\right]} \quad \text{..... (2-1)}$$

where,

d_1 = uniform delay assuming uniform arrivals (s/veh);

C = Cycle length (s); cycle length used in pretimed signal control, or average cycle length for actuated control;

g = effective green time for lane group (s); green time used in pretimed signal control, or average lane group effective green time for actuated control

X = v/c ratio or degree of saturation for lane group.

The random delay can be expressed as:

$$d_r = \frac{X}{2c(1-X)} \quad \text{..... (2-2)}$$

where *c* is the capacity of a lane group. Webster also estimated an adjustment term by simulation and concluded that control delay can be approximated as $d = 0.9\ (d_u + d_r)$.

Webster model is a very classical delay estimation model and it was widely accepted as an accurate depiction of delay for the idealized case of uniform arrivals, stable flow and no initial queue. Following Webster's work, a number of stochastic models have been developed, including those by Newell *(18)*, Miller *(19) (20)*, McNeil *(21)*, and Heidemann *(22)*. These models generally assume that arrivals are Poisson distributed, with an underlying average rate of vehicles/unit time, and the system remains under-saturated over the analysis period. Therefore these models can not be directly used when traffic demand exceeds intersection capacity for a significant period of time.

The HCM 2000 *(23)* uses control delay as the criteria for LOS of both signalized and unsignalized intersections. In this manual, the total delay was defined as "the difference between the travel time actually experienced and the reference travel time that would result during base conditions, in the absence of incident, control, traffic, or geometric delay". Control delay was defined as the proportion of total delay attributed to control measures. Control delay includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. With respect to field measurements, control delay is defined as the total elapsed time from the time a vehicle stops at the end of the queue to the time the vehicle departs from the stop line.

The HCM 2000 developed a procedure to estimate average control delay for a given lane group. The average control delay was divided into three components. The first component represents delay assuming the uniform arrival of vehicles. The second component adds an incremental delay to account for stochastic arrivals and occasional oversaturation. The third component adds delay as the result of an initial queue at the beginning of the analysis period. The average control delay per vehicle for a given lane group is given by the following equation:

$$d = d_1 (PF) + d_2 + d_3 \qquad (2\text{-}3)$$

where,

d = control delay per vehicle (s/veh);

d_1 = uniform control delay assuming uniform delays (s/veh);

PF = uniform delay progression adjustment factor, which accounts for effects of signal progression;

d_2 = incremental delay to account for effect of random arrivals and oversaturation queues, and

d_3 = residual demand delay to account for initial queues.

In this model, d_1 has the same form as the uniform delay in Webster model (2-1). The incremental delay d_2 can be estimated by the following equation:

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8klX}{cT}}\right] \quad \text{(2-4)}$$

Where *T* is the length of the analysis period (hrs), *k* is the incremental delay factor that is dependent on controller settings, and *I* is the upstream filtering/metering adjustment factor. The model is adjusted for traffic-actuated control with factor *k* depending on unit extension and degree of saturation. For isolated pretimed signals *k*= 0.5 and *I*=1.0.

A set of research studies have been conducted to test and compare existing delay models *(24) (25) (26)*. Luttinen *(24)* compared the HCM2000, Danish DanKap, and Swedish Capcal 2 models with simulation data and indicated that HCM 2000 underestimate capacity and overestimate delay at high degrees of saturation (X>0.75). For traffic-actuated control HCM 2000 estimated somewhat too low delays at low degrees of saturation. Another problem with HCM 2000 model is that it does not consider the extra delay due to the blocking effect of short turning lanes. This effect is emphasized especially in the already problematic situation with high degrees of saturation and a large number of left-turning vehicles.

Qureshi (2003) *(26)* suggested using simulation software to estimate delay for intersections with actuated control. He also illustrated that using current analytical procedures to estimate delay at actuated controlled signalized intersection has the following limitations:

(1) The variability of traffic demand within a given control period cannot be fully considered. Analyses are typically using the average demand within a period;

(2) Unusual arrival and service patterns that do not follow traditional statistical distributions cannot be modeled; and

(3) The models cannot be used to analyze real-time traffic operations, as such operations are typically concerned with instantaneous and cyclic flows rather than average flows.

2.3.2 Delay Models at Unsignalized Intersection

There have been many studies on developing capacity and delay models to evaluate traffic operations at unsignalized intersections. Radwan and Kumares developed a delay-flow rate relationship for undivided and divided 4-lane highways *(27)*. 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 (turning movements), respectively.

The Highway Capacity Manual 2000 developed a procedure to estimate the delay, capacity, and level of service of unsignalized intersections *(28).* 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 *(29).* The two-stage priority situation as it exists at many unsignalized intersections within multilane major streets provides larger capacities and smaller delay as compared to intersections without central storage areas *(30).* A study by Robinson and Tian 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 *(31).* However, these theoretical models have not been calibrated against empirical data.

The 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). 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. The HCM 2000 uses the following model to estimate control delay at TWSC intersections:

$$d = \frac{3600}{c_{m,x}} + 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right] + 5 \quad \text{(2-5)}$$

where,

d = control delay (s/veh);

v_x = flow rate for movement x (veh/hr);

$C_{m,x}$ = capacity of movement (veh/hr); and

T = analysis time period (hr) (T=0.25 for a 15-min period)

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.

2.4 U-turn as an Alternative to Direct Left Turn

In roads designed with restrictive medians, left turn egress movements are only permitted at full median openings. As indicated before, left turn movements posed a lot of problems to roadway safety and operations. Many states have started taking strict restriction on median opening spacing to reduce the density of full median opening. The Access Management Manual presented that “when providing a full median opening on the fringe of an urban area, it is important to consider the potential for future signalization. A full median opening that is located where signalized intersection will interfere with efficient traffic progression may need to closed or reconstructed as a directional median opening”. A directional median opening means an opening in a restrictive median which provides U-turn only, and/or left-turn in movements. Replacing a full median opening with a

directional median opening will reduce conflict points, simplify driving tasks, and was found to significantly reduce crash rates (Figure 2-1)*(1)(7)(8)(9)*.

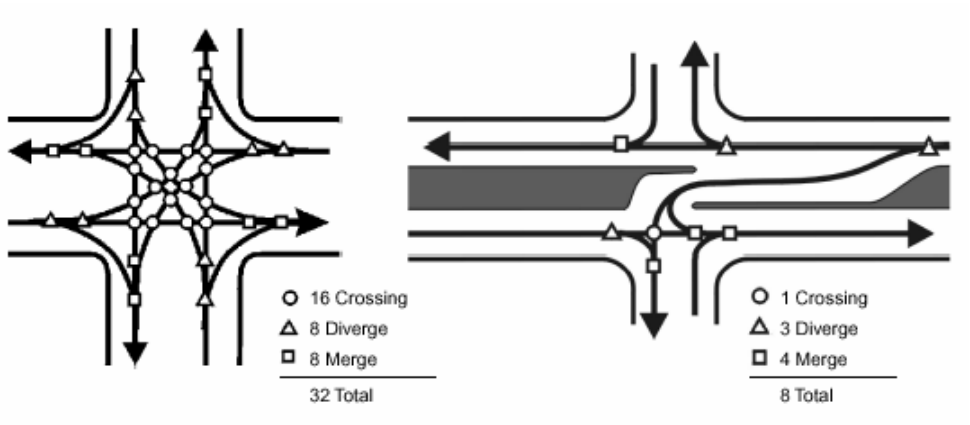


Figure 2-1 Vehicular conflict points at a typical four-way intersection versus a directional median opening.

(Sources: Access Management Manual)

Florida makes extensive use of directional median openings in the State Highway System. By closing existing median openings in some major arterial roads or replacing them with directional median openings, Florida prohibits left-turn exits onto major arterials. Left turn egress movements would be made by turning right onto the arterial road and then making U-turns at downstream median opening or signalized intersection (Figure 2-2).

Several studies have been conducted to evaluate the operational effects of providing U-turns at median openings as an alternative to direct left turns from a driveway. An analytical model was developed and calibrated in NCHRP Report 420 *(32)* to estimate the travel time savings when unsignalized left turns are diverted for various distances. It can apply to both suburban and rural environments where there are no nearby traffic signals. The key findings are as follows:

(1) A right turn followed by a U-turn will require up to one minute of travel time, assuming a diversion distance of about 1,320 ft;

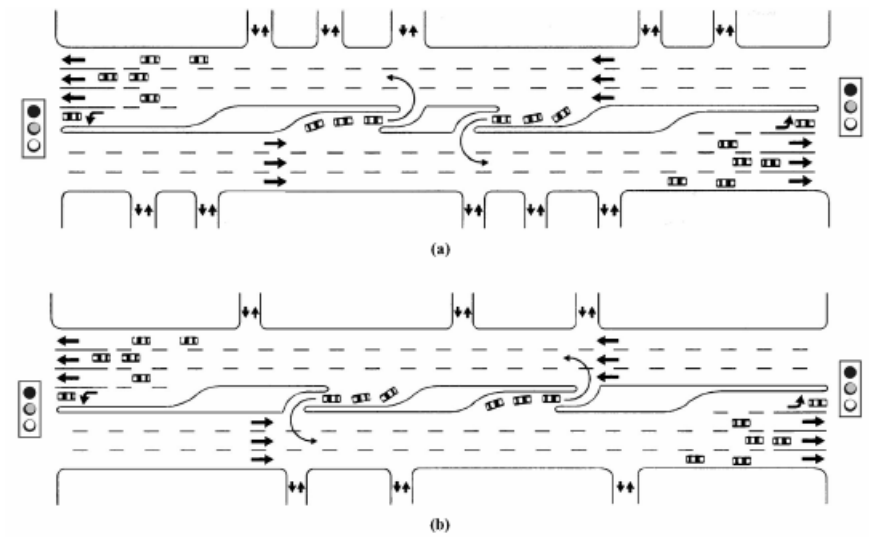


Figure 2-2 Unsignalized directional median openings. (a) downstream from the signalized intersections and (b) upstream from the signalized intersections.

(Sources: Access Management Manual)

(2) A single-stage left-turn exit (where medians are too narrow to safely store two or more vehicles) will involve the following delays (not including acceleration times), as shown in Table 2-1:

Table 2-1 Left-turn delay under different volume conditions

(Source: NCHRP 420)

Volumes(vph)		Delay per Vehicle (Seconds)
Artery *(Two directions)*	Left-Turn Exit	
1,000	50	20
1,000	100	25
2,000	50	200
2,000	100	530

.

These values suggest that when arterial traffic exceeds 375 to 500 vphpl on a four-lane facility the computed delays would exceed those associated with the right turn/U-turn movement. Higher volumes (700-900 vphpl) that are common along many suburban arterials would produce even higher left-turn egress delays in theory. In practice, motorists become impatient when gaps exceed 1 to 2 min and are attempt to avoid the direct left turn egress; and

(3) The two-stage left turn process, where medians can safely store waiting vehicles, reduces delays to left-turning traffic. Nevertheless, this process still results in long delays to left-turning vehicles when the volumes on the major street are relatively high (i.e., more than 2,000 vph), and the left turns exceeds 50 per hour. In these cases, even with substantial circuity (1,320 ft or 402m from the access drive to the U-turn median opening, or a 0.5 mi of additional travel) the right turn followed by a U-turn involves less time than calculated left-turn egress movements under moderate to high volumes.

The project sponsored by FDOT in 2001 (*Methodology to Quantify the Effects of Access Management Treatments on Roadway Operations and Safety*) provided very useful information on the access management treatment on – *Right-Turn followed by U-turn at Median Opening as an Alternative to the Direct Left Turn from Driveways and Side Streets* . The study took three basic approaches in evaluating the issue including operational evaluation, conflict data analysis, and crash data analysis. This project involved huge amount of field data collection where the extensive data were gathered at several appropriate locations using video cameras followed by lengthy data reduction process in the lab. Delay and travel time models were developed using collected data to quantify the relationship between delay and travel time to explanatory variables. Basic conclusions got from this project including:

(1) The curves based on delay and travel time models 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 a U-turn;

(2) 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 ;

(3) 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;

(4) 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, instead of a DLT.

2.5. U-turn at Signalized Intersections

As mentioned previously, the 2001 USF research has focused on U-turns at downstream median openings. In the real world, however, there are many other conditions where U-turns are provided such as at downstream signalized intersections. Considering different U-turn treatments, different state may have different state of practice. For example, in Wisconsin, U-turns are not legal at signalized intersections. U-turn movements are provided at “pre-U-turn” openings near signalized intersections. Michigan uses U-turn channels on highways with wide medians and prohibits all turning turns at signalized intersections. U-turn lanes can be provided downstream of signalized intersection. It is also called Michigan “U”, as shown in Figure 2-3. Increasingly, Florida is limiting unsignalized median openings to left turn ingress from the major arterials; hence, drivers wanting to turn left from a driveway must turn right onto the major road and then make a U-turn downstream.

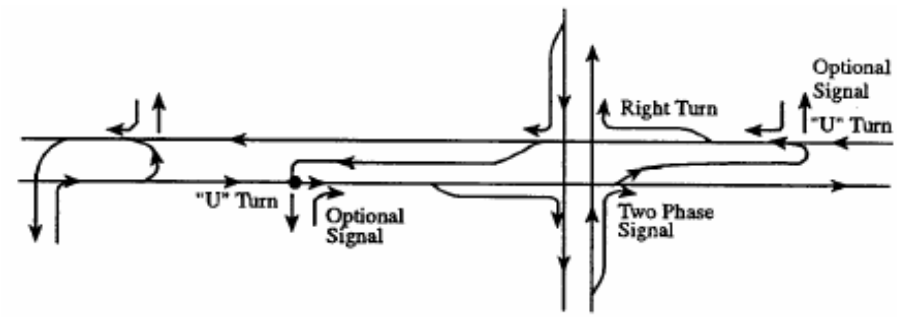


Figure 2-3 Michigan “U”

(Source: NCHRP 420)

In the Florida Median Handbook, Sokolow *(10)* identified three different U-turn approaches which have been widely implemented in Florida and nationally, including U-turns at signalized intersections, U-turns in advance of signalized intersection and U-turns after signal. Sokolow also indicated that providing U-turns in advance of a signalized intersection would result in two successive left-turn lanes and unless there was

a substantial length of full median width, drivers may mistakenly enter the U-turn lane. It was also recommended by the handbook that where medians are of sufficient width to accommodate dual left-turn lanes, U-turn could be provided from the inside left turn lane at signalized intersections. For this specific situation, Florida Median Handbook listed three issues that need to be considered: (1) “Right-on-red” restrictions for side streets. (2) Remember to look at signal operation. (3) Don’t let the signalization intersection work against U-turns.

NCHRP 420 analyzed three different U-turn approaches including providing U-turn lanes in advance of, at, or beyond signalized intersections. As indicated in this report:

(1) Left-turn lanes can be provided for U-turning vehicles in advance (i.e., upstream) of signalized intersections. This avoids concentrating development-related turning traffic at signalized junctions of major crossroads;

(2) Dual left-turn lanes can be provided at signalized intersections with the inner lane dedicated to U-turns. Many states now provide these lanes; however, they still require multiphase traffic signal controls; and

(3) Left- and U-turn lanes can be provided downstream of signalized intersection, thereby allowing two-phase traffic signal controls.

Another concern of providing U-turns at signalized intersections is that U-turn movements may have some adverse influence on signalized intersection capacity. A study conducted by Webster and Cobbe in 1966 *(33)* recommended the following relationship between radius and saturation flow rate for the exclusive left-turn movements:

$$s = \frac{2080}{1 + \frac{4.92}{R}} \quad \text{(2-6)}$$

Where:

s = saturation flow rate for exclusive left-turn movement (vphpl); and

R = radius of curvature (ft).

The equation shows that the saturation flow rate increases with increasing turning radius. Since U-turns usually have smaller turning radius than left turn movements, it is anticipated that U-turn may have lower turning speed than that of left-turn movement. Therefore U-turning vehicles may have some adverse impacts on left turn lane capacity.

Two previous research studies have substantiated this assumption. A study conducted at North Carolina State University in 1993 *(34)* evaluated the effects of U-turns on left-turn saturation flow rates. The study team selected four intersections with exclusive left-turn lanes and protected signal phasing and recorded saturation flow rates and U-turn percentages for 198 queues during weekday midday peaks. The data analysis showed that "a saturation flow reduction factor appears necessary for left-turn lanes that have large percentages of U-turns. Saturation flow rates were significantly lower when queues had more than 65% U-turns". However, the analyses also showed no correlation between saturation flow and the percentage of U-turns for queues with 50% or fewer U-turns. The results of this study suggest tentative saturation flow reduction factors of 1.0 for U-turn percentages below 65, 0.90 for U-turn percentages between 65 and 85, and 0.80 for U-turn percentages exceeding 85. A follow-up investigation should focus on intersections that have high percentages of U-turns, restrictive geometry, or high percentages of U-turning heavy vehicles.

Tsao and Chu (1995) recorded 600 headways of left-turning passenger cars and 160 headways of U-turning passenger cars in Taiwan *(35)*.Their study revealed that the average headways of U-turning passenger cars are significantly larger than those of left turning passenger cars. The effects of U-turning vehicles depend upon the percent of U-turning vehicles in the left-turn lane, as well as the order of formation in the traffic

stream. When preceded by a left-turning vehicle, the average headway of U-turning passenger car is 1.27 times that of left-turning passenger cars. When preceded by a U-turning vehicle, however, the average headway of U-turning passenger cars is 2.17 times that of left-turning passenger cars. The U-turn adjustment factors for varying percents of U-turning vehicles in left-turn lanes developed in this study is listed in the following Table 2-2:

Table 2-2 Adjustment Factor for U-turns

Percent of U-turn	0	2	4	6	8	10	15	20	25	30
Average Value	1	0.98	0.97	0.95	0.94	0.92	0.89	0.86	0.84	0.81

Conclusions yielded from these two studies can not be directly used in this project. The reasons are:

(1) Both of these studies assume that the discharge flow rate of the vehicle reaches saturation state after fourth or fifth discharged vehicle. The field measurement, however, shows that when U-turning vehicles are introduced in the left-turn traffic stream, the discharge flow rate does not display an easily identifiable steady maximum rate; and

(2) Considering the variations in drivers' behavior in different areas, conclusions from these two projects may not hold in Florida.

Based on the literature reviewed, generally there is no widely accepted procedure for estimating the effects of U-turns on signalized intersection capacity. In HCM, U-turns are treated as left-turns when estimating saturation flow rate. However, the operational effects of these two maneuvers are different. In practice, 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 downstream signalized intersections. The increased U-turning movements at signal may further degrade the signalized intersection and the effects should not be ignored.

2.6 Summary

After an extensive literature search, which included current rules and regulations, design standards and policies in Florida and nationally, and a computer search of the Transportation Research Information Service (TRIS) database, conclusions can be made that little documentation is available on operational effects of providing U-turns at downstream signalized intersections as an alternative to direct left turns from driveways or side streets. An extensive study on this specific access management treatment is needed, both for setting design policy, and in project-level design. Other findings include:

(1) Vehicle's delay at signalized intersections is largely affected by signalization conditions, including g/c ratio, cycle length, and demand flow rate;

(2) Providing U-turn movements in advance of or at downstream signalized intersection is a controversial topic. However, little documentation is available concerning the comparison of these two U-turn approaches; and

(3) There is no widely accepted procedure for estimating the effects of U-turns on signalized intersection capacity. Related studies have not been conducted in Florida before;

3 METHODOLOGY

The methodologies used in studying the operational effects of right-turn followed by U-turns at signalized intersection as an alternative to direct left turns are explained in this chapter. This chapter consists of three sections. The fist section explains the methodology used in the specification of delay and travel time models. The second part deals with driver's selection of RTUT or DLT on the basis of accessibility considerations. The third part of this chapter discusses the effects of U-turns on signalized intersection capacity.

3.1 Delay and Travel Time Models

Vehicle delay and travel time are very important parameters used by transportation professionals to evaluate the operational performance of intersections. The importance of vehicle delay and travel time is reflected in the use of these parameters in both design and evaluation practices. In addition, when implementing median modification projects, highway agencies often face some public concerns because some people believe that making a right turn followed by a U-turn may experience much longer delay and travel time as compared with those who would otherwise make direct left turns.

One of the major objectives of this project is to develop delay and travel time models of RTUT and DLT. In order to address public concerns, it is necessary to compare delay and travel time of RTUT versus DLT under specific traffic volume levels and roadway geometrics. In addition, from the decision maker's point of view, it is also necessary to quantify the relationship of delay and travel time to possible explanatory variables, including conflicting traffic flow rates, signalized intersection characteristics, and roadway geometric characteristics. This information is very useful for decision makers in determining what kind of median opening will be applicable under given traffic conditions and roadway geometrics.

3.1.1 Operations Analysis

3.1.1.1 Direct Left Turns

The Highway Capacity Manual 2000 identifies the priority of right-of-way given to each traffic stream at unsignalized intersection. Based on the definition, DLT egress from a driveway or minor 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. As a consequence, when wishing to make a DLT wait for longer periods, drivers 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, that can allow one or two vehicles to stop, a DLT maneuver may require four steps, as shown in Figure 3-1 and the specific steps are explained as follows.

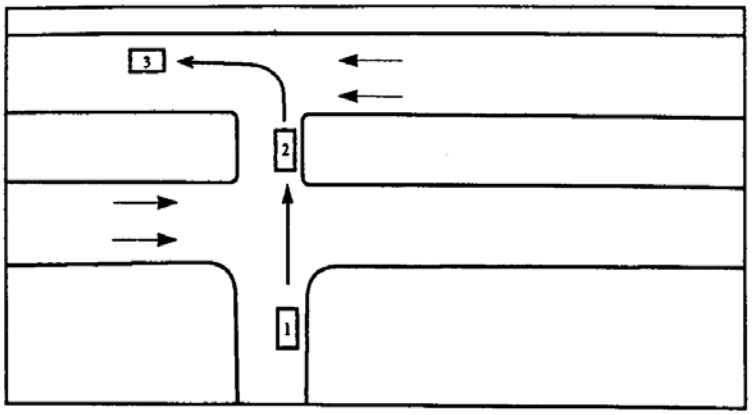


Figure 3-1 DLT Egress Movements (Source: NCHRP 4-20)

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;

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

$$TT_L = t_{L1} + t_{L2} + t_{L3} \quad (3\text{-}1)$$

$$TD_L = t_{L1} + t_{L2} \quad (3\text{-}2)$$

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}).

3.1.1.2 Right Turn plus U-turns

In order to eliminate problems associated with DLT movements, many states and transportation agencies have started installing restrictive medians and directional median openings in the state highway system. Left turn egress movements would be replaced by turning right onto the arterial road and then making U-turns at downstream median opening or signalized intersection. As shown in Figure 3-2, a vehicle making a RTUT at downstream signalized intersection also requires four steps.

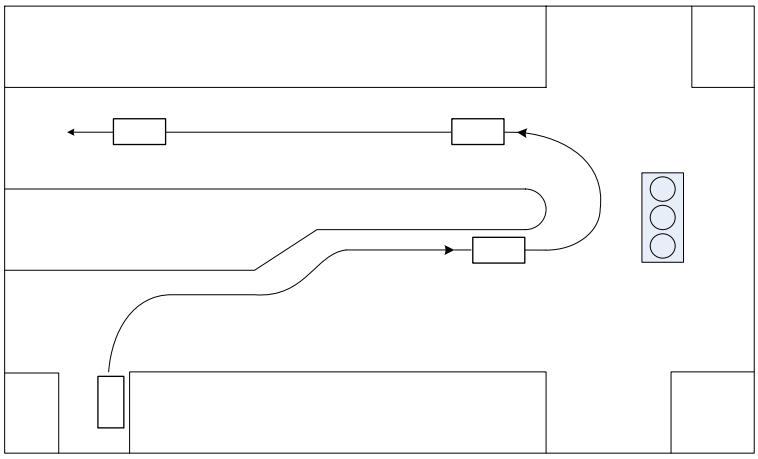


Figure 3-2 RTUT Movements

Step 1: Stopping at the driveway, and making a right turn onto major road when a suitable gap is available from left-side through-traffic. It is much easier for drivers to make a right turn than to make a left-turn egress at a driveway considering vehicles making a right turn do not need to yield to other turning movements at the unsignalized intersection. Usually, when the upstream signal for the major-road through-traffic turns red, there is a large gap for right turns from driveways. Drivers can easily make a right turn without interference with other turning maneuvers at median

opening. It is important to note that 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 a crash when their attention is focused on the major-road through traffic;

Step 2: Accelerating, weaving to the inside lane, and decelerating to a stop at the exclusive left turn lane of the downstream signalized intersection. 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. Sometimes when the left turn lane is not long enough, U-turning vehicles may be blocked by through traffic already queued at the traffic signal. It will cause extra delay to RTUT vehicle;

Step 3: Waiting until the signal turns green to make a U-turn. Delay of U-turns at signalized intersection is highly correlated with signalization conditions and demand flow rate. Field study found that U-turning vehicles could experience relatively long delay at signalized intersection especially when the signal has long cycle length and/or heavy left-turn movements are present at the signal. In addition, as mentioned previously, vehicles making a U-turn may have longer discharge headway and start up lost time as compared with those making left turns. Therefore U-turn movements may have some adverse impacts on left turn lane capacity. In this step, if U-turn is made during protected signal phase, drivers making U-turns do not need to yield to the through-traffic in other direction of the road. Therefore there is no conflict between U-turning vehicles and through-traffic. For the condition when U-turn is provided during permitted signal phase, drivers making a U-turn must wait until a suitable gap is available from downstream through-traffic and then make a U-turn. This condition is very similar to the condition where U-turn is provided at median opening. It is important to note that there is a potential conflict between U-turning vehicles and right-on-red vehicles in the other approach

of the road. Both U-turn drivers and right-turn drivers can easily overlook this conflict, which could result in a crash; and

Step 4: Accelerating to the operating speed of through-traffic. As compared with DLT movement, it does not result in speed reduction in through traffic if U-turn is made during protected signal phase.

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

$TT_{RU} = t_{RU1} + t_{RU2} + t_{RU3} + t_{RU4}$ ……...……. (3-3)

$TD_{RU} = t_{RU1} + t_{RU2}$……...…... (3-4)

$$t_{RU4} = \frac{l}{1.47 * V_T}$$ …… ……...…… . (3-6)

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 exclusive left turn lane of downstream signalized intersection (seconds)

t_{RU3} = average running time from leaving the driveway to stopping at the exclusive left turn lane (not including t_{R1} and t_{R2}) (seconds);

t_{RU4} = average running time of vehicles crossing the whole roadway section at the posted speed of through-traffic (seconds);

l_1 = weaving distance from the subjective driveway to the median U-turn opening (ft.)(as shown in Figure 3-3);

l_2 = length of left turn storage bay (ft)(as shown in Figure 3-3);

l = the distance form the studied driveway to the U-turn bay, including weaving distance and the left turn storage bay (ft), $l=l_1+l_2$;

v_T = speed limit on the major arterials (mph); and

1.47 = conversion factor from mph to ft/sec.

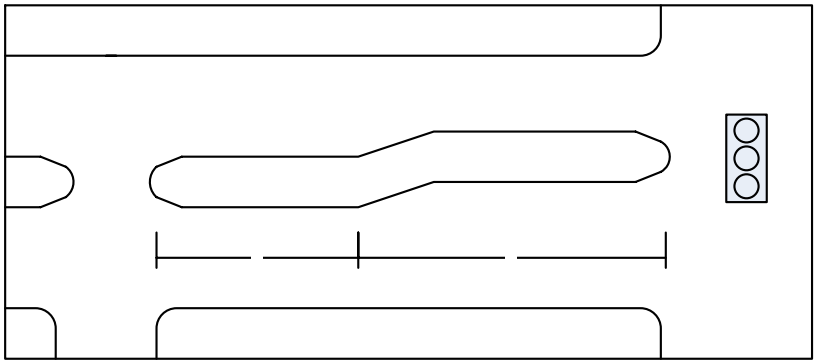


Figure 3-3 the Offset Distance from Subjective Driveway to Signalized Intersection

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 signalized intersection (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 whole roadway segment (weaving section plus exclusive left turn lane) at the operating speed of through-traffic. The average total delay and travel time were used to quantify operational effects of RTUT vs. DLT.

3.2 Driver Selection of RTUT

Driver's selection of RTUT or DLT may be different under various levels of traffic volume and roadway geometrics. An extensive study regarding driver's selection of RTUT is necessary which can provide decision maker's with useful information to select a suitable median treatment, including median opening closure and replacing a full median opening with a directional one, etc.

3.2.1 Driver Selection of RTUT at Signalized Intersection

The USF 2001 study*(3)* developed a regression model to describe the relationship between the percentage of drivers choosing RTUT at median opening and the combination of left-turn-in flow rate and major-road through-traffic flow rate. The regression results are given in the following table:

Table 3-1 Regression Results for Ratio of RTUT at median opening

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

$$Ratio = 0.23e^{0.004LTIN+0.0002TV-2.1SPLIT} \quad \text{(3-9)}$$

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.

The model shows that the percentage of RTUT movements increases with the increase of major-road through-traffic flow rate and left-turn-in flow rate from major road. In practice, when there is a suitable U-turn median opening downstream, some drivers prefer to make a RTUT rather than a DLT when the median storage space has been occupied by conflicting vehicles. This decision is encouraged when there are a large number of left-turn-in vehicles from the major road.

When U-turn movements are accommodated at downstream signalized intersection, drivers' choice behavior may be largely different from the condition where U-turn is provided at median openings. As indicated earlier, a driver making a U-turn at signalized intersection does not need to wait and find a suitable gap form the through-traffic in the

other direction of the road since U-turn is provided at protected signal phase. Some drivers may prefer this option with the perception that it is safer than to make a U-turn at other locations such as median opening. However, some drivers do not like making a U-turn at signalized intersections since they think it takes long time to wait for the signal to turn green.

Another factor that could influence driver's choice behavior is the offset distance from the driveway to the downstream signalized intersection. In the field, people generally do not like to make a right turn followed by a U-turn at downstream signalized intersection when the intersection is located out of the driver's sight distance. In contrast, if the distance is too short, drivers whishing to make a right turn onto major arterials need to select a suitable simultaneous gap in all through lanes and then make a direct entry into the inside lane, and wait until the signal turns green to make a U-turn. Sometimes, it is very difficult to perform this movement especially when through volume is very heavy and the driveway is blocked by through traffic already queued before the traffic signal. In these particular conditions, some drivers may make a right turn on to the major arterial road, cross the intersection, and then make a U-turn at a median opening downstream of the signalized intersection. It is a challenging topic to select a suitable offset distance from driveway to downstream intersection or median opening, especially when the potential effects of the offset distance on safety and operations of RTUT need to be considered. More details considering this topic would be included in another project.

3.2.2 Fitting a Binary Logistic Regression Model

Logistic regression is a technique for analyzing problems in which there are one or more independent variables which determine an outcome that is measured with a dichotomous variable in which there are only two possible outcomes. In the case of binary logistic

regression models, the relationship between a binary response variable and one or more explanatory variables are modeled.

For a binary response variable y, the linear logistic regression model has the following form:

$$Logit(p_i) = \log(\frac{p_i}{1-p_i}) = \alpha + \beta' X \quad \text{(3-9)}$$

where,

p_i = Prob. ($y_i = y_1/X_i$) is the response probability to be modeled, and y_1 is the first ordered level of y;

α = the intercept parameter;

β' = the vector of slope parameters; and

Xi = the vector of explanatory variables.

This logistic regression equation models the logit transformation of the ith individual's event probability, p_i, as a linear function of the explanatory variables. Logistic regression is widely used to estimate bounded fractional dependent variables. As compared with OLS regression, the predicted value from a logistic regression can be guaranteed to lie in the unit interval ($0 \le p_i \le 1$). It is natural to model its population regression as a linear function since $\log[p_i/(1-p_i)]$ can take on any real values as p_i varies between 0 and 1. In this study, a binary logistic regression model was developed to estimate the percentage of drivers selecting RTUT at downstream signalized intersection as an alternative to direct left turns from driveway under certain roadway traffic and geometric conditions.

3.3 The Effects of U-turns on Signalized Intersection Capacity

In some cases, when a full median opening is replaced by a directional median opening, drivers wishing to make direct left turns must be diverted to make a right turn followed by a U-turn at downstream signalized intersection. As indicated before, the increased U-turn movements at signalized intersection may have some adverse impacts on the intersection capacity, considering the operational characteristics of U-turning vehicles. This problem may become acute when there are a large percentage of U-turning vehicles in left-turn traffic stream, and the medians are not wide enough to accommodate dual left-turn lanes.

Saturation flow rate is one of the most critical factors in estimating capacity of a lane or lane group at signalized intersections. In this study, the effects of U-turns on signalized intersection capacity are estimated by applying U-turn adjustment factors for varying percentages of U-turning vehicles on left-turn saturation flow rate.

3.3.1 Procedures for Estimating Saturation Flow Rate

As indicated in HCM 2000, saturation flow rate is "the equivalent hourly rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal is available at all times and no lost time are experienced". Based on this definition, the saturation flow rate is the maximum flow rate that can pass through a given lane group under prevailing conditions. In estimating saturation flow rate, different adjustment factors are applied to address the impacts of prevailing conditions, including lane width and lateral clearance, number of lanes, heavy vehicles and grades, turning movements, interchange density, lane distribution, and environmental factors. A saturation flow rate for each lane group can be estimated according to the following equation:

$$s = s_0 N f_w f_{NV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb} \quad \text{(3-10)}$$

where,

s = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h);

s_0 = base saturation flow rate per lane (pc/h/ln);

N = number of lanes in lane group;

f_w = adjustment factor for lane width;

f_{HV} = adjustment factor for heavy vehicles in traffic stream;

f_g = adjustment factor for approach grade;

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;

f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area;

f_a = adjustment factor for area type;

f_{LU} = adjustment factor for lane utilization;

f_{LT} = adjustment factor for left turns in lane group;

f_{RT} = adjustment factor for right turns in lane group;

f_{Lpb} = pedestrian adjustment factor for left-turn movements; and

f_{Rpb} = pedestrian-bicycle adjustment factor for right-turn movements.

As an alternative to the estimation of saturation flow rate using Equation 3-10, saturation flow rate for each lane group can also be estimated by field measurement. As indicated in HCM, the measured values of prevailing saturation flow rate in the field will produce more accurate results. Discharge headway research is widely used in the field measurement of saturation flow rate at signalized intersections. In practice, when the green signal is initiated, headways between departing vehicles will be observed as vehicles cross the stop line. The first headway will be the time between the initiation of the green

signal and the crossing of the first vehicle over the stop line. The second headway is the time between the first and second vehicles crossing the stop line. Any reference points can be used when recording headways, as long as the identical point is maintained through measurement. Common practice is to measure the headways as the rear wheels of the reference vehicle cross the curb line. This study uses the rear wheel as the reference point in field measurement.

Most of previous studies indicate that discharge headway converges to a constant headway, which is usually achieved after the fourth to sixth discharged passenger car crossing the stop line after the beginning of the green signal. The constant headway is defined as the saturation headway. The relationship between saturation flow rate and saturation headway is shown in the following equation:

$$s = \frac{3600}{h} \quad \text{(3-11)}$$

where,

s = saturation flow rate (vphpl)

h = saturation headway (sec)

3600 = seconds/hour

3.3.2 The Effects of U-turns on Left-turn Saturation Flow Rate

In order to develop a procedure for estimating the effects of U-turns on signalized intersection capacity, a pilot survey was conducted at the early stage of this project. An intersection with exclusive left-turn lane and protected signal phasing was selected for this survey. The intersection is located on Fowler Avenue in Tampa, which is a six-lane principle arterial road. The signal is actuated controlled with an average cycle length of 149 sec.

The study team recorded discharge headways for 138 left-turning vehicles and 54 U-turning vehicles in 27 discharging queues during weekday peak hour. To focus on the characteristics of passenger-car flows, the data related to heavy vehicles and all vehicles behind a heavy vehicle are excluded from analysis. The queue discharge patterns for queues with different percentage of U-turning vehicles are shown in the following Figure 3-4:

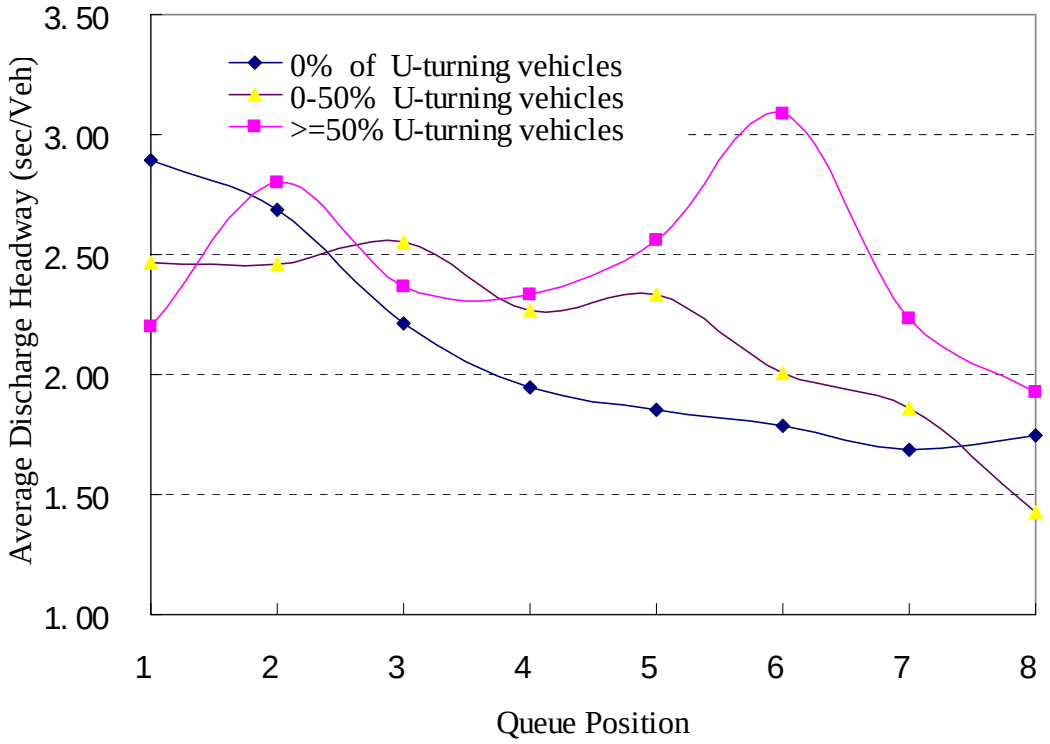


Figure 3-4 Comparison of Queue Discharge Patterns for Different Percentage of U-turning Vehicles in Left-turn Lane

Past studies generally indicate that when a vehicle queue is released by a traffic signal turning green, the discharge flow rate of the vehicles quickly reaches a steady state. The collected data matches this conclusion. As shown in Figure 3-4, the average discharge headways for left-turning vehicles converge to a relatively constant state from forth or fifth discharged vehicle after green onset. For the situations in which there are U-turning

plus left-turning vehicles in the discharging flow, however, the queue discharge patterns do not display an easily identifiable steady maximum rate. Field measurement found that it is very difficult to get saturation headway in the field measurement when there are U-turning vehicles in the discharging flow. In addition, the figure shows that the average discharge headway increases with the percentage of U-turning vehicles in discharging queue. This could be explained by the different turning characteristics of these two movements. As mentioned earlier, U-turn movement has shorter turning radius than left turn movement. Consequently it has lower turning speed as compared with left turn movement. Field observation found that there is a conflict point between U-turning vehicle and the preceding left-turning vehicles. In practice, when a vehicle is making a U-turn at signalized intersection, sometimes the following left turn vehicles have to make a break in order to avoid a rear end collision. In this condition, the saturation state is broken by the U-turning vehicle. Preceding vehicles may need to accelerate again before reaching a saturation state.

In HCM, U-turns are treated as left-turns in the current procedure for estimating saturation flow rates at signalized intersections while the operating characteristics of these two movements are different. Two past studies *(34) (35)* and the pilot survey have substantiated the assumption that the average discharge headway of U-turning vehicles is larger than that of left-turning vehicles. Moreover, U-turning vehicles cause greater effects to their succeeding vehicles than left-turning vehicles. Therefore, conclusion can be made that U-turn movements would reduce left turn saturation flow rate, especially, when the proportion of U-turning vehicles in left-turn lane is large. It is recommended in this project to apply an adjustment factor for U-turning movements when estimating left turn saturation flow rate.

3.3.3 Adjustment Factor for U-turn Movement

Similar to other turning movement adjustment factors, such as right-turn adjustment factor and left-turn adjustment factor, the U-turn adjustment factor also depends on a number of variables, including:

(1) Whether U-turns are made from exclusive left turn lanes or shared lanes;

(2) Type of phasing (protected, permitted, or protected-plus-permitted); and

(3) Proportion of U-turning vehicles in the left turn lane.

In this project, only the condition in which U-turn movements being accommodated at exclusive left turn lane with protected signal phasing was considered.

The study conducted by Tsao and Chu in 1996 use discharge headway research to estimate U-turns effects on traffic flow in left-turn lanes. The study recorded discharge headway for 600 left-turning vehicles and 160 U-turning vehicles in left-turn lanes. This research assumed that the discharge flow rate of the vehicle reaches saturation state after fourth or fifth discharged vehicle, and only the headways after the fifth discharged vehicle were recorded. In Tsao and Chu's study, U-turn adjustment factor was estimated using the following equations:

$$f_{UT} = \frac{1}{2}\left(\frac{h_{LL}}{h_{\max}} + \frac{h_{LL}}{h_{\min}}\right) \quad \text{(3-12)}$$

$$h_{\max}(a) = \left(1 - \frac{a}{100}\right)h_{LL} + \left(\frac{a}{100}\right)h_{UU} \quad \text{(3-13)}$$

$$h_{\min}(a) = \left(1 - \frac{2a}{100}\right)h_{LL} + \left(\frac{a}{100}\right)h_{LU} + \left(\frac{a}{100}\right)h_{UL} \quad \text{(3-14)}$$

where,

f_{UT} = the adjustment factor for U-turns;

$h_{min}(a)$ = the lower limit of average headway with a% of U-turning vehicles;

$h_{max}(a)$ = the upper limit of average headway with a% of U-turning vehicles;

h_{LL} = the average headway between two successive left-turning vehicles (sec);

h_{UU} = he average headway between two successive U-turning vehicles (sec); and

a = percentage of U-turning vehicles.

As mentioned earlier, this equation cannot be directly used in this project because:

(1) This study assumed that the discharge flow rate of vehicle reaches saturation state after the fourth or fifth discharged vehicle. The field measurement, however, indicates that when there are U-turning vehicles in the left-turn lane, the discharge flow rate does not display an easily identifiable steady maximum rate;

(2) Field observation found that the number of U-turning vehicles in the first four discharged vehicles would affect the discharge headways of succeeding vehicles. In the field, when there are U-turning vehicles in the first four discharged vehicles, following left turn vehicles cannot fully speed up to reach the maximum saturation state. This study did not consider this condition since only the headways after the fifth discharged vehicle were recorded; and

(3) Tsao and Chu's study was conducted in Taiwan. Considering the variations in drivers' behavior in different areas, the conclusion may not hold in USA.

In this project, the determination of a U-turn adjustment factor is achieved by analyzing the relationship between percentage of U-turning vehicles in the left turn lane and the average queuing discharge time of the whole discharging flow, including the first four discharged vehicles. As indicated before, U-turn movement has larger average discharge headway than left-turn movement. In addition, a U-turn movement will force succeeding left-turning vehicles to slow down to avoid a rear end collision. Therefore, when there are

U-turning vehicles in exclusive left turn lanes, the discharging queue will consume more green time than the queue with only left-turning vehicles. Theoretically, the difference increases with the percentage of U-turning vehicles in the discharging flow. To analyze the relationship between the percentage of U-turn vehicles in the left turn lane and the average queuing discharge time, a linear regression model was specified using field data from the pilot survey. The relationship is illustrated in the following Equation 3-15:

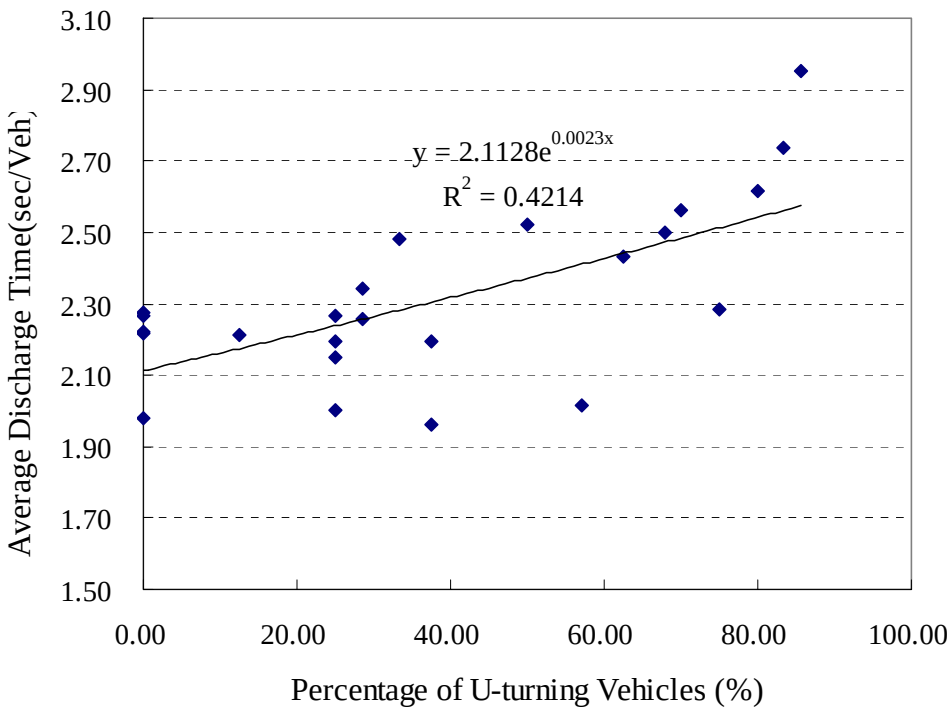


Figure 3-5 The Relationship between Average Discharge Time & the Percentage of U-turning Vehicles

$$h = h_0 e^{0.0023 P_{UT}} \quad \text{(3-15)}$$

where,

h = average queuing discharge time for U-turn and left-turn mix flow (sec);

h_0 = base average queuing discharge time for left-turn only flow (sec); and

P_{UT} = percentage of U-turning vehicles from inside left-turn lane (%).

The pilot survey shows that, when there is no U-turning vehicle in left-turn lane, the average queuing discharge time for each left turning vehicle is 2.11 sec. The average queuing discharge time increases with the percentage of U-turning vehicles. The relationship is shown in Equation 3-15. Based on the definition of the adjustment factors for turning movements, the U-turn adjustment factor for the left-turn saturation flow rate can be estimated by the following equation:

$$f_{UT} = \frac{\frac{3600}{h}}{\frac{3600}{h_0}} = \frac{h_0}{h} = \frac{2.1128}{2.1128e^{0.0023P_{UT}}} = \frac{1}{e^{0.0023P_{UT}}} \quad \text{……..……} \quad (3\text{-}16)$$

where,

f_{UT} = adjustment factor for U-turn movement;

. h = average queuing discharge time for U-turn and left-turn mix flow;

h_0 = base average queuing discharge time for left-turn only flow (sec); and

P_{UT} = percentage of U-turning vehicles from inside left-turn lane (%).

Equation 3-16 cannot be directly used in estimating the adjustment factor for U-turns because the sample size is limited and the R-square value is low. However, the procedure for estimating U-turn adjustment factors for different percentage of U-turning vehicles in left-turn lane can be used in future study.

4 DATA COLLECTION AND REDUCTION

This study consists of huge amount of field data collection work. During June 2002 to July 2003, field measurement was conducted on eight urban or suburban arterial street segments in Tampa Bay area in Florida, where extensive data were collected using video cameras. A total of more than 300 hours of field data was collected. This chapter discusses the detailed efforts of data collection and data reduction work.

The major objective of this project is to quantify the operational effects of right turns followed by U-turns at downstream signalized intersection as an alternative to direct left turns. The data needed to achieve this objective are listed as follows:

(1) Traffic volume: major-road through-traffic volume, left-turn-in volume from major-road, left-turn-out volume from driveway and side street and right turn followed by U-turn volume;

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

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

(4) Signal parameters: green arrow time, cycle length, queue discharge time, queue discharge headways for left-turning and U-turning vehicles, and left- turn volume from inside left turn lane;

(5) Geometric data: cross section, lane assignments, weaving distance, length of left-turn storage bay, and median type, and

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

4.1 Site Selection

Site selection work was conducted during June 2002 to November 2002. The major purpose of site selection is to find compatible site with high RTUT and DLT volumes. More specifically, the geometric criteria of selecting specific sites include:

(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

(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;

(6) The median width should be wide enough to store the left-turning vehicles, and

(7) The downstream signal should have exclusive left turn lane and protected left turn phasing in the studied approach. The condition in which U-turn

movements being accommodated at permitted left turn phase is not considered in this study.

Based on these criteria, eight sites located in Tampa Bay area were selected for field measurement. The selected sites are listed in Table 4-1.

Table 4-1 Description of Selected Sites

Site			N_1	N_2	Speed limit(mph)	Median type	g/C	l(ft)
	Arterial	Location						
1	Fowler Ave.	56th St.	6	Dual	50	D	0.17	465
2	Fowler Ave.	22nd St.	8	Single	45	F	0.16	645
3	Hillsborough Ave.	Webb Ave.	6	Single	45	F	0.14	300
4	Dale Mabry Hwy.	North Dale St.	6	Single	45	D	0.15	560
5	Bruce B. Downs Blvd.	Fletcher Ave.	6	Dual	45	F	0.16	900
6	54th St.	34th St.	6	Dual	45	F	0.28	550
7	54th St.	22nd N. St.	6	Single	40	F	0.11	390
8	Dale Mabry Hwy.	Maple Dale St.	6	Single	45	F*	0.15	525

Note: N1: # of through lanes; N2: # of exclusive left turn lanes at signalized intersection; D: directional median opening; F: Full median opening; l: the distance from driveway to downstream signalized intersection, including weaving distance and left-turn storage bay; and g/c: green cycle ratio. For actuated controlled signal, the g/c ratio here is defined as the maximum green arrow time for left turn phase divided by the average cycle length of the signalized intersection.

Site 1 is located in the city of Tampa, at Fowler Avenue and 56th Street. Fowler Avenue is a principle arterial road with three lanes in each direction. The studied driveway is located on Fowler Avenue and it serves a shopping plaza with a Publix Supermarket and many small businesses. The median opening across the driveway is a directional median opening,

which restricts left turn egress movements from driveway. The speed limit of the selected road segment is 50 mph.

Figure 4-1 Site 1 Fowler Avenue and 56th Street

Site 2 is located in the city of Tampa, at Fowler Avenue and 22nd Street. Fowler Avenue is a principle arterial road with four lanes in each direction. The studied driveway is located on Fowler Avenue and it is one of the driveways that serve the University Mall. The driveway has two lanes for egress of vehicles with one lane dedicated to DLT vehicles and the other one dedicated to right turning vehicles. The median opening across the driveway is a full median opening which allows almost all turning movements at un-signalized intersection. The speed limit of selected segment is 45mph.

Site 3 is located in the city of Tampa, at Hillsborough Avenue and Webb Avenue. Hillsborough Avenue is a principle arterial road, which has three lanes in each direction. The studied driveway is located on Hillsborough Avenue. The driveway serves a parking lot for a plaza includes a major bank and some small businesses. The median opening across the driveway is a full median opening. The speed limit of this road is 45mph.

Figure 4-2 Site 2 Fowler Avenue and 22nd Street

Site 4 is located in the city of Tampa at Dale Mabry Highway and North Dale Street. Dale Mabry Highway is a major highway divided by a raised median. This highway has three lanes for each direction. The studied driveway is on Dale Mabry Highway and it is one of the driveways that serve a major shopping plaza that includes many small businesses and retail stores. The median opening across the driveway is a directional median opening, which restricts left turn egress movements from the driveway. The speed limit is 45 mph at selected segment.

Figure 4-3 Site 3 Hillsborough Avenue and Webb Street

Site 5 is located in the city of Tampa at Bruce B. Downs Boulevard and Fletcher Avenue. Bruce B. Downs Boulevard is a major arterial road with three lanes in each direction. The driveway is one of the driveways that serve Target Plaza that consists of; Target, Eckerd and U Save supermarkets, fast food restaurants and many small businesses. The driveway is on Bruce B. Downs Boulevard and has two separate lanes for DLT and right-turn movements. There is a full median opening located across the driveway. The speed limit in this segment is 45 mph.

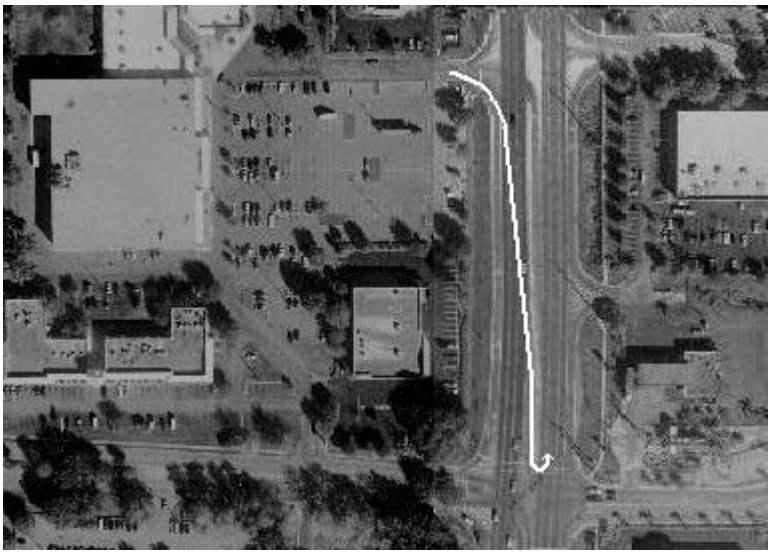


Figure 4-4 Site 4 Dale Mabry Highway and North Dale Street

Site 6 is located in the city of Saint Petersburg at 34th Street and 54th Street. 34th Street is major arterial with three lanes for northbound and southbound traffic. The driveway that was studied is one of the driveways that serve a major shopping plaza consists of a Publix Supermarket, some retail stores and many small businesses. The median across the driveway has a full median opening. The posted speed limit here is 45 mph.

Site 7 is located in the city of Saint Petersburg at 34th Street and 22nd N. Street. 34th Street is major arterial with three lanes for northbound and southbound traffic. It is divided by a raised median. The driveway that was studied is one of the driveways that serve a major

shopping plaza consists of a Kash N Karry Supermarket, some retail stores and many small businesses. The median across the driveway has a full median opening. The RTUT movements are completed with a U-turn at the 34th Street and 22nd N. Street signalized intersection. The posted speed is 45 mph.

Figure 4-5 Site 5 Bruce B. Downs Boulevard and Fletcher Avenue

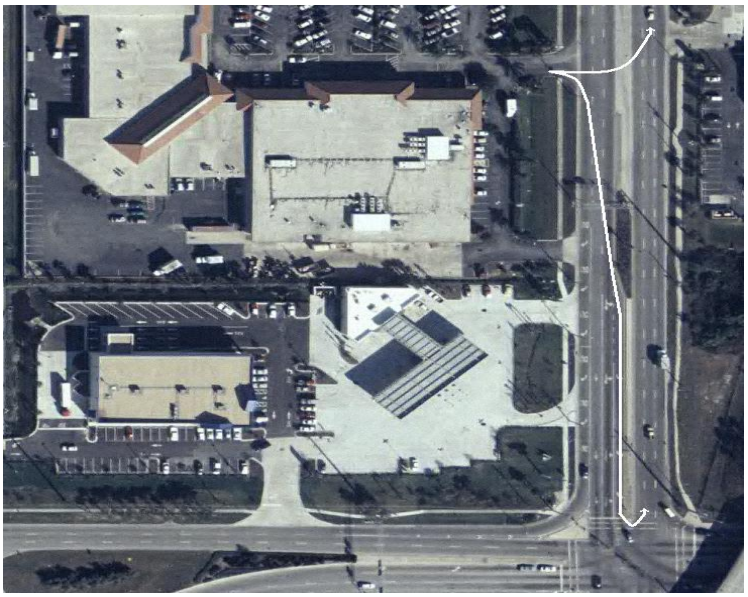


Figure 4-6 Site 6 34th Street and 54th Street

Site 8 is located in the city of Tampa at Dale Mabry Highway and Maple Dale Street. At this segment, Dale Mabry Highway is divided by a raised median with three lanes in each direction. The selected driveway is on Dale Mabry Highway and it serves the parking lot for Sam's Club Retail Store. The median opening in this site is different from that of other sites. The median opening here permit left egress from driveway while left-turn-in movements from major road are prohibited. The speed limit in this segment is 45 mph.

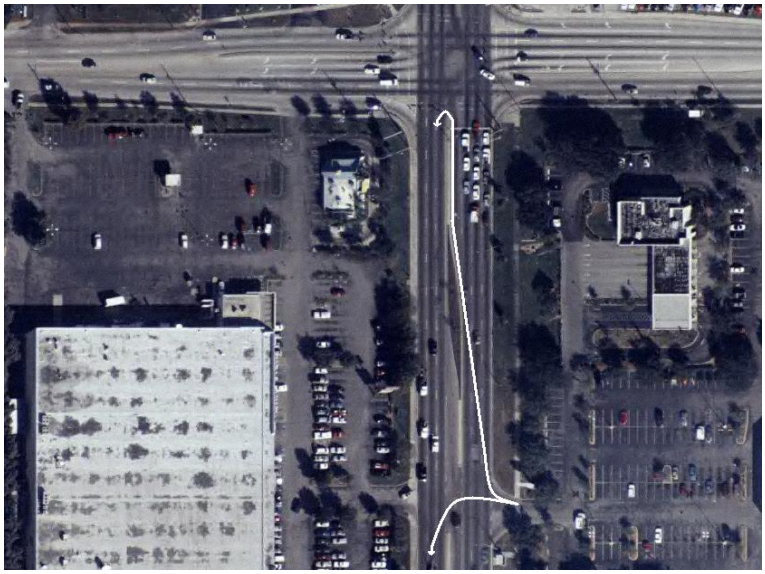


Figure 4-7 Site 7 54th Street and 22nd N. Street

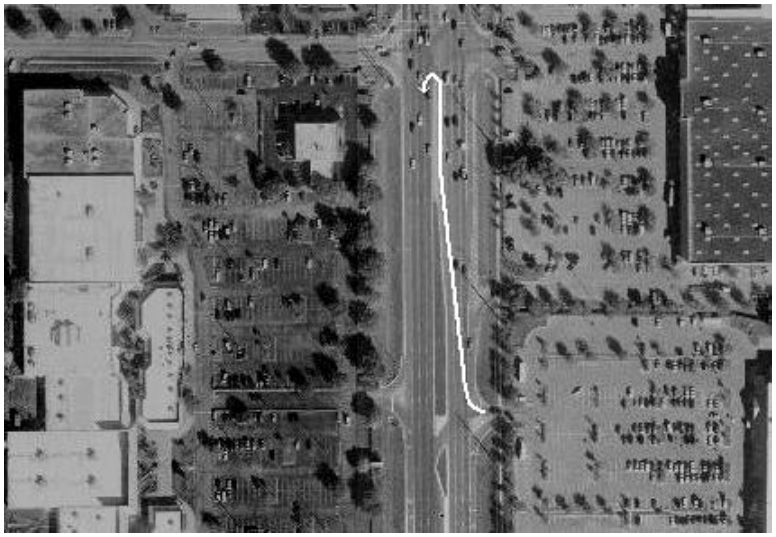


Figure 4-8 Site 8 Dale Mabry Highway and Maple Dale Street

4.2 Data Collection

In this study, equipments used for data collection include 5 video cameras, VCRs, batteries, inverters, and TVs. In order to cover the whole right turn followed by U-turn procedure and the signalized intersection parameters, the two-story scaffoldings were installed in the field. Figure 4-9 shows that cameras were set up at the top of a 15-feet high scaffolding. The equipments used for data collection are shown in Figure-10 and Figure-11. The basic cameras locations in the field are shown in Figure 4-12.

Figure 4-9 Equipments Setup in the Field

A typical data collection day generally starts at 7:00 in the morning. Before start recording, all video cameras were synchronized so that the data extracted from different videotapes can be matched. Data collection usually was conducted during weekday 7:00 AM to 7:00PM. More than 30 hours data were collected in each site. Data were not collected during inclement whether or when there were unusual traffic conditions in the road.

Figure 4-10 Equipments Setup in the Field

Figure 4-11 Equipments Setup in the Field

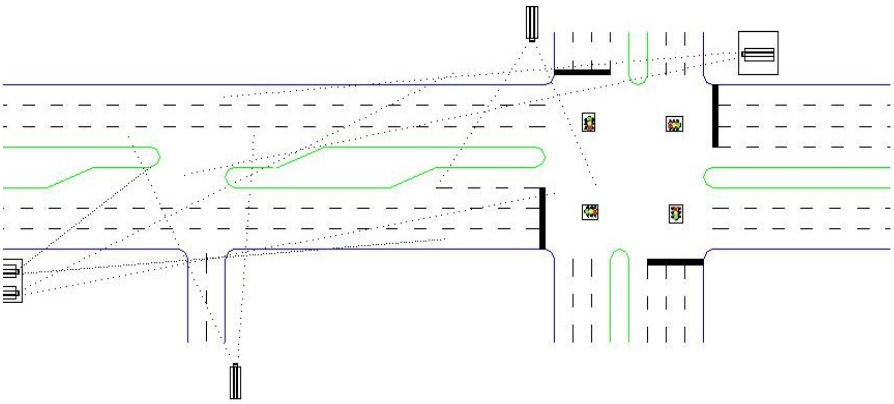


Figure 4-12 Basic Camera Locations in the Field.

4.3 Data Reduction

The collected videotapes were reviewed in office. In this project, the reduction of field data is very hard and timing consuming since there were more than 300 hours videotapes to be reviewed. Each videotape was reviewed for five to six times in order to gather different categories of data needed for further analysis. Each vehicle coming from the driveway making a DLT or a RTUT was tracked. Since all video cameras have already been synchronized in field, data collected by different video cameras can be matched. By reviewing videotapes, the following information was recorded:

(1) Waiting delay: waiting delay of DLT and RTUT vehicles at driveway; waiting delay of DLT vehicles at median opening; and waiting delay of RTUT vehicles at signalized intersection;

(2) Travel time: the total travel time of DLT and RTUT vehicles;

(3) Traffic volume: major-road through-traffic volume, left-turn-in volume from major-road, left-turn-out volume from driveway, and right turn followed by U-turn volume; and

(4) Signal parameters: green arrow time, cycle length, queue discharge time, queue discharge headways for left-turning and U-turning vehicles, and left turn volume from inside left turn lane.

Total delay of each vehicle at driveway is measured from a vehicle stops at the waiting queue until it exits the stop line. The definition of delay here consists of queue time and service time. This definition is a little bit different from the definition of average control delay in HCM, since vehicles' deceleration and acceleration were not considered when estimating delay in this project. The waiting delay of left-turns at a median opening was measured by recording the time from the vehicle stops at the median until it leaves the median. The waiting delay of U-turning vehicles at signalized intersection was recorded as the time from the vehicle stops at the inside left turn lane until it starts making a U-turn. By tracking each individual vehicle, the total travel time of each DLT or RTUT vehicle can also be recorded.

The reduction of field data is based on five-minute time interval. In each interval, the average total delay and travel time for DLT and RTUT vehicles were recorded. In addition, traffic volume data, including major-road through-traffic volume, left-turn-in volume from major-road, left-turn-out volume from driveway, right turn followed by U-turn volume, and left-turn volume from inside left-turn lane, were also counted based on this time interval.

5 ANALYSIS OF OPERATIONAL EFFECTS

5.1 General

In this study, the operational evaluation of right turns followed by U-turns at signalized intersections as alternatives to direct left turns is conducted through the following approaches:

(1) The comparison of the average delay of DLT and RTUT under various levels of traffic volume and roadway geometrics. This objective was accomplished through the development of delay models for these two maneuvers

(2) The comparison of the average total travel time of DLT and RTUT under various levels of traffic volume and roadway geometrics. This goal was attained by building travel time models for two maneuvers;

(3) The estimation of the percentage of drivers selecting RTUT when both choices are available. A binary logistic regression model was developed to achieve this objective; and

(4) The estimation of the effects of U-turns on signalized intersection capacity. This objective was achieved by applying the adjustment factors for varying percentages of U-turns on left-turn saturation flow rate.

In this chapter, the operations of two widely used U-turn approaches, U-turns at median opening in advance of signalized intersection and U-turns at signalized intersection, were also compared based on the models developed in this study and those from the USF 2001 study.

Models were built based on field data gathered from selected sites. As mentioned before, the reduction of field data was based on five-minute time interval. When specifying models, 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, such as higher adjusted R^2 value. In this study, statistical analysis was performed by the use of the SPSS software.

5.2 Average Delay

Delay is an important measure of effectiveness (MOE) of traffic operations. The HCM 2000 uses control delay as the criteria to evaluate the level of service (LOS) of signalized intersections. Control delay is defined as the proportion of total delay attributed to control measures, which includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. Delay defined in this study doesn't include vehicles' deceleration and acceleration time; because it is very difficult to measure deceleration and acceleration in the field without sophisticated tracking equipment. In this study, delay is defined as the total elapsed time from the time a vehicle stops at the end of the queue to the time the vehicle departs from the stop line. The average delay of DLT movements consists of delay at driveway and delay at median opening. The average delay of RTUT includes delay at driveway and delay at signalized intersection.

5.2.1 Delay Model for Direct Left Turn

Data collected from sites with full median openings were used to build delay model for direct left turn movements. As mentioned earlier in this chapter, the original data set at five-minute intervals were aggregated to fifteen-minute intervals when specifying models. Figure 5-1 shows the conflicting volumes affecting the delay of DLT movement.

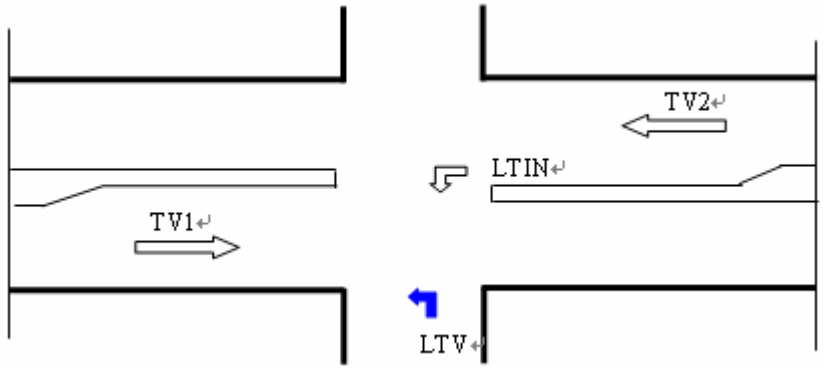


Figure 5-1 Traffic flows affecting the Delay of DLT

Statistical analysis showed that both linear and exponential forms are suitable to describe the relationship between the average delay of DLT movement and conflicting volumes. However, the exponential form was found to have better goodness of fit with field data. The delay model was described as Equation 5-1.

$$TD_L = e^{a_1 TV + a_2 SPLIT + a_3 DLTV + a_4 LTIN + a_0} \quad \text{..... (5-1)}$$

Where,

TD_L = average total delay of DLT (sec/ veh),

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

DLTV = 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

In total, 464 observations at fifteen-minute intervals were used to estimate the delay model for DLT movement. 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 statistical characteristics of collected data are given in Table 5-1. The final regression results are listed in Table 5-2.

Table 5-1 Descriptive Statistics of the Collected Data

	Average Total Delay	Flow Rate of TV	SPLIT	Flow Rate of DLT	Flow Rate of LTIN
Valid	465	465	465	465	465
Missing	0	0	0	0	0
Mean	38.1	3437	.5096	43.62	48.29
Median	36.4	3441	.5038	40.00	44.00
Mode	25.0	3200	.44(a)	36.00	36.00
Std. Deviation	16.1	614.0	.04886	22.39	20.76
Variance	260.5	377112.6	.002	501.41	431.176
Range	81.7	3080	.22	112.00	116.00
Minimum	9.6	1884	.39	8.00	8.00
Maximum	91.3	4964	.62	120.00	124.00

As shown in Table 5-2, all independent variables are significant at a 95 percent level of confidence. The adjusted R square value is 0.415, which implies that the selected independent variables can explain 41.5% of variations in the dependent variable. 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. According to these parameter estimates, the developed regression equation is:

$$TD_L = 10.63e^{0.0004TV - 0.935SPLIT + 0.004DLTV + 0.004LTIN} \quad \text{(5-2)}$$

Where,

TD_L = average total delay of DLT (sec/ veh),

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

DLTV = flow rate of DLT from a driveway (vph),

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

SPLIT = percentage of upstream through traffic flow rate,

SPLIT=TV1/ (TV1+TV2).

Table 5-2 Regression Results for Delay Models of DLT

Model Summary (b)

Mode	R	R Square	Adjusted R Square	Std. Error of the Estimate
	.648(a)	.420	.415	.33665

a Predictors: (Constant), LTIN, VOLUME, DLTV, SPLIT

b Dependent Variable: lnTD

ANOVA (b)

Model	Sum of Squares	df	Mean Square	F	Sig.
Regression	37.681	4	9.420	83.122	.000(a)
Residual	52.132	460	.113		
Total	89.813	464			

a Predictors: (Constant), LTIN, VOLUME, DLTV, SPLIT

b Dependent Variable: lnTD

Coefficients (a)

Model	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
(Constant)	2.364	.184		12.851	.000
TV	.0004	.000	.525	11.766	.000
SPLIT	-.935	.411	-.104	-2.277	.023
DLTV	.004	.001	.210	4.702	.000
LTIN	.004	.001	.185	4.198	.000

a Dependent Variable: lnTD

In Equation 5-2, the coefficients of DLTV (0.004) and LTIN (0.004) are much greater than the coefficient of TV (0.0004). This implies that DLT and LTIN flow rate have greater

impact on the delay of left-turning vehicles 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 the corresponding upstream flow rate (TV1). This could be explained by the fact that when the median space is occupied by other turning vehicles, left-turning vehicles must wait at the driveway even if suitable gaps are available at the upstream through-traffic stream.

Based on Equation 5-2, curves for the average delay of DLT under different levels of traffic volumes can be developed. Figure 5.2 shows a group of curves for average delay of DLT assuming the left-turn-in flow rate from the major road is 100 vph, split is 0.5, and the flow rate of DLT is 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.

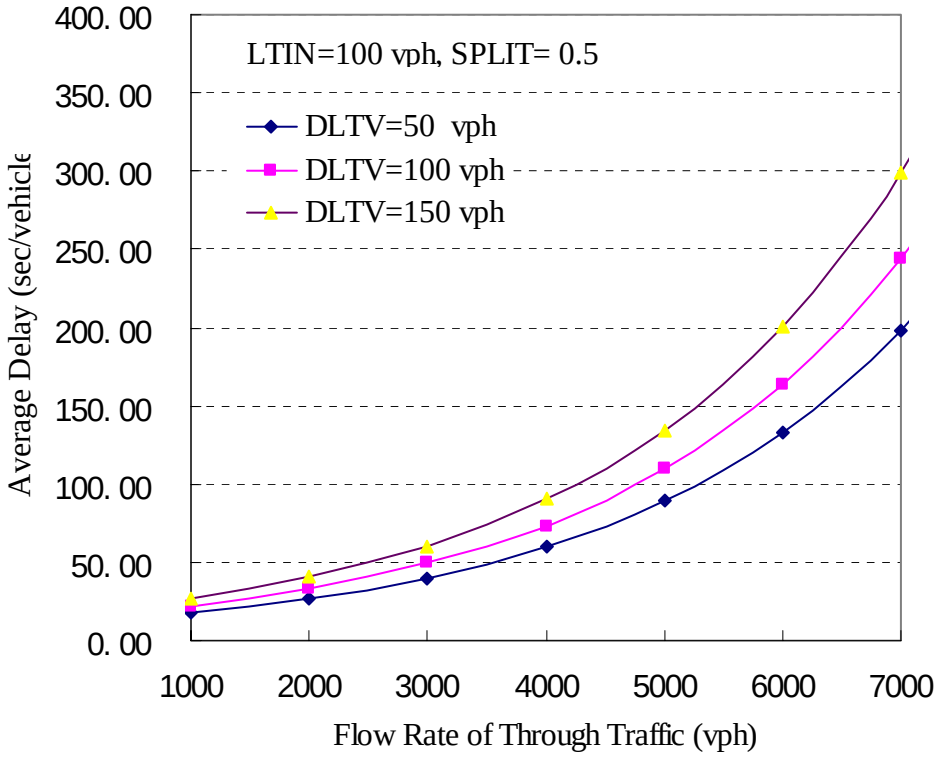


Figure 5-2 Curves for the Average Total Delay for DLT

(LTIN=100 vph, Split=0.5)

5.2.2 Delay Model for Right Turn Followed by U-turn

In this study, the total delay of RTUT includes delay at driveway and delay at signalized intersection. Field measurement found that the delay of U-turning vehicles at signalized intersection was decided by the signalization conditions and demand flow rate, including g/c ratio, cycle length, and left-turn flow rate from inside exclusive left-turn lane. Variables expected to affect the delay of U-turning vehicles at driveway include major-road through-traffic flow rate, split and RTUT flow rate.

Another variable that could affect RTUT vehicles' delay at driveway is the offset distance from the subjective driveway to downstream signalized intersection. As shown in Figure 5-3, when this distance is shorter than the left turn deceleration lane on the major road (Type A), drivers wishing to make a RTUT will select a suitable simultaneous gap in all through lanes and then make a direct entry into the left turn deceleration lane. When the distance is medium or long (Type B), drivers do not have to wait for a simultaneous gap in all three through lanes, since they can easily select a suitable gap, turn into the right-side lane, accelerate to an appropriate speed, and then weave to the exclusive left turn lane. Therefore, RTUT vehicles' delay at driveway may decrease with the increase of the offset distance from the subjective driveway to downstream U-turn bay. On the other hand, when the offset distance is too long, vehicles making a RTUT will experience longer travel time than those would otherwise make a DLT. Therefore, the key point here is to find the optimal distance from the subjective driveway to downstream U-turn bay. In practice, it is difficult to define a suitable distance from the driveway to downstream intersection or median opening since there are several different factors that need to be considered, including traffic operations and transportation safety. This specific topic will be addressed in another research project.

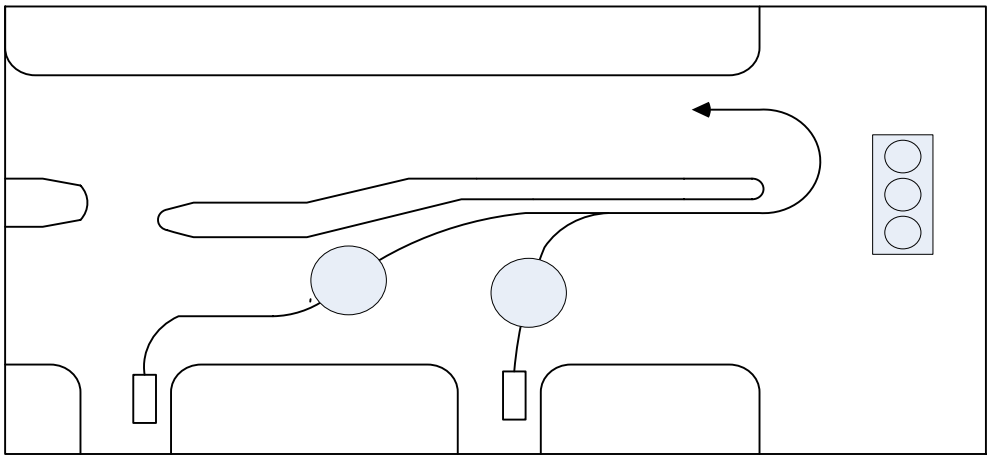


Figure 5-3 Two Different Weaving Patterns

The average total delay model for RTUT movement is described as follows:

$$TD_{RU} = e^{a_1 TV + a_2 SPLIT + a_3 RUV + a_4 LTV + a_5 G/C + a_6 C + a_7 L + a_0} \quad \text{(5-3)}$$

Where,

TD_{RU} = average total delay of RTUT (sec/ veh),

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

RUV = flow rate of RTUT from a driveway (vph),

G/C = g/c ratio for exclusive left turn phase,

C = Cycle length (sec); cycle length used in pretimed signal control, or average cycle length for actuated control;

LTV = left-turn flow rate from inside left turn lane;

L = the distance from driveway to downstream signalized intersection,

SPLIT = percentage of upstream through traffic flow rate,

SPLIT=TV1/ (TV1+TV2), and

a_0, a_1, a_2, a_3, a_4, a_5, a_6, a_7 - parameters

The dependent variable in this model is the average total waiting delay per vehicle making a right turn followed by a U-turn at downstream signalized intersection during a fifteen-minute interval. In this study, g/c ratio is defined as the green arrow time for

left-turn phase divided by the cycle length of selected signal. If the study site is an actuated signal with varying cycle and phase length, g/c ratio is defined as the maximum green arrow time for left-turn phase divided by average cycle length. This definition is different from that of some other studies, which generally use average green arrow time when defining g/c ratio of actuated controlled signal. However, this study found that the maximum green arrow time is a better indicator of actuated controlled signal capacity, and has better statistical characteristic such as adjusted R^2 value when incorporated into delay and travel time models.

A total of 610 observations at fifteen-minute intervals were used to perform the regression analysis. The statistical characteristics of collected data are given in Table 5-3. The regression results are listed in Table 5-4.

As shown in Table 5-4, all independent variables are significant at a 95 percent level of confidence. The adjusted R square value is 0.46, which implies that the selected independent variables can explain 46% of variations in the dependent variable. 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. According to these parameter estimates, the final developed regression equation is:

$$TD_{RU} = 28.73e^{0.00016TV+0.427SPLIT+0.003RUV+0.002LTV-3.483G/C+0.0059C-0.00056L} \quad \text{..... (5-4)}$$

Where,

TD_{RU} = average total delay of RTUT (sec/ veh),

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

RUV = flow rate of RTUT from a driveway (vph),

G/C = g/c ratio for exclusive left turn phase,

C = Cycle length (sec); cycle length used in pretimed signal control, or average cycle length for actuated control;

LTV = left-turn flow rate from inside left turn lane;

L = the distance from driveway to downstream signalized intersection,

SPLIT = percentage of upstream through traffic flow rate,

SPLIT=TV1/ (TV1+TV2), and

a_0, a_1, a_2, a_3, a_4, a_5, a_6, a_7 - parameters

Table 5-3 Descriptive Statistics of the Collected Data

	TD	VOLUME	SPLIT	G/C	C	L	RUV	LTV
Valid	610	610	610	610	610	610	610	610
Missing	0	0	0	0	0	0	0	0
Mean	76.1	3575.66	.48	.16	122.2	577.93	18.27	107.69
Median	74.8	3516.00	.47	.15	106.67	525.00	16.00	104.00
Mode	78.00	3332	.41	.15	106.67	525	16.00	100.00
Std. Deviation	19.1	493.83	.07	.014	19.81	149.40	5.76	42.17
Variance	363.7	243862	.004	.00	392.44	22320.7	33.17	1778.7
Range	112.2	3320	.31	.17	57.68	600	40.00	268.00
Minimum	44.00	1588	.33	.11	106.67	300	12.00	8.00
Maximum	156.3	4908	.63	.28	164.35	900	52.00	276.00

As shown in Equation 5-4, the coefficient of TV is very small (0.00016), which implies that the average total delay of RTUT is not sensitive to the change in flow rate of through-traffic. The coefficient for G/C is negative, which suggests that providing a large g/c ratio for left-turn phase will reduce RTUT delay at signal. Obviously, a long cycle length will result in long waiting delay for vehicles in left-turn lane. Therefore, the coefficient for C is positive. Another finding from this equation is that the increase of distance from driveway to downstream signalized intersection reduces average total delay of RTUT. As mentioned earlier in this chapter, when this distance is long enough, the driver making a right turn does not have to wait for a simultaneous gap in all three through lanes since they can easily select a suitable gap, turn into the right-side lane, accelerate to an appropriate speed, and then weave to the exclusive left turn lane. Therefore, in this

condition, the RTUT drivers will have less delay at driveway as compared with the condition where the distance is too short.

Table 5-4 Regression Results for Delay Models of RTUT

Model Summary

Mode	R	R Square	Adjusted R Square	Std. Error of the Estimate
	.683(a)	.466	.460	.17992

a Predictors: (Constant), L, VOLUME, RUV, SPLIT, C, LTV, G/C

ANOVA (b)

Model	Sum of Squares	df	Mean Square	F	Sig.
Regression	17.032	7	2.433	75.164	.000(a)
Residual	19.487	602	.032		
Total	36.519	609			

a Predictors: (Constant), L, VOLUME, RUV, SPLIT, C, LTV, G/C

b Dependent Variable: lnTD

Coefficients (a)

Model	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
(Constant)	3.358	.096		35.157	.000
VOLUME	.000	.000	.327	9.959	.000
SPLIT	.427	.123	.115	3.479	.001
G/C	-3.483	.653	-.199	-5.333	.000
C	.006	.000	.474	13.013	.000
RUV	.003	.001	.076	2.386	.017
LTV	.002	.000	.407	11.169	.000
L	-.001	.000	-.346	-9.208	.000

a Dependent Variable: lnTD

Based on Equation 5-4, different curves can be developed under different volume and roadway geometric conditions. Curves in Figure 5-4 are developed assuming a g/c ratio of 0.15, cycle length of 120 sec, SPLIT of 0.5, left-turn flow rate from inside left turn lane of 100 vph, and a distance from driveway to downstream signal of 560 ft. In this figure, the x-axis represents the flow rate of major-road through-traffic; and the y-axis refers to the average total waiting delay per vehicle making a right turn followed by a U-turn at downstream signalized intersection during a fifteen-minute interval.

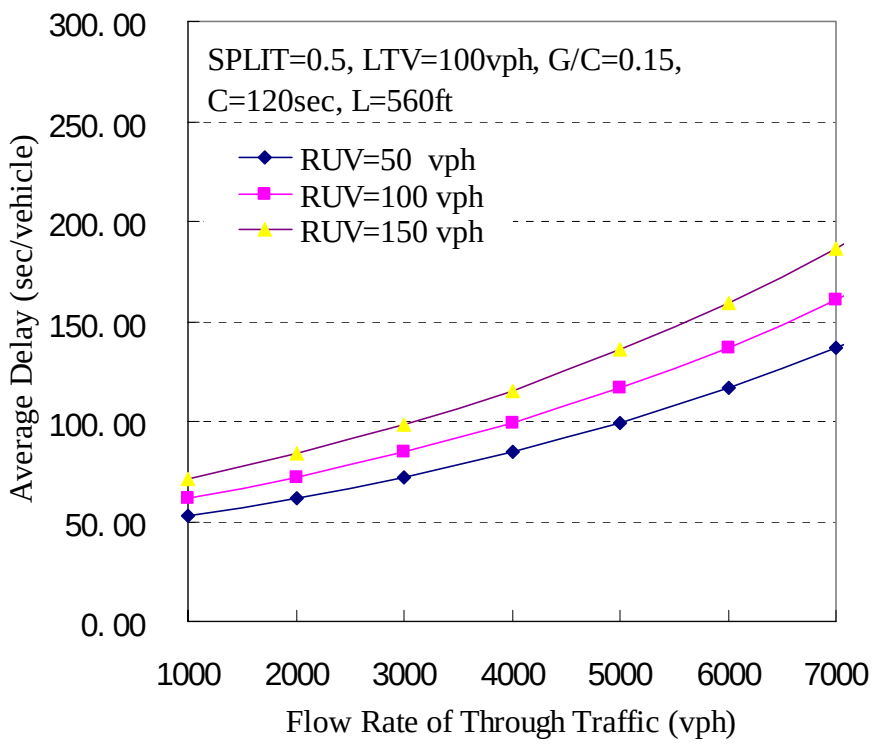


Figure 5-4 Curves for the Average Total Delay for RTUT

(SPLIT=0.5, LTV=100vph, G/C=0.15, C=120sec, L=560ft)

5.2.3 Delay Comparison of DLT and RTUT

One of the major objectives of this project is to compare delay of DLT and RTUT under specific traffic and roadway geometric conditions. In practice, DLT will have less delay than RTUT when conflicting volumes are very light. On the other hand, when conflicting

volumes increase, the delay of DLT experiences a sharp increase because of the restrained median storage and the gap acceptance characteristics of DLT movement. Therefore, RTUT could have longer delay than DLT under high levels of traffic volumes. In order to determine under what volume conditions, DLT would have more delay than RTUT, the curves in Figure 5-2 and Figure 5-4 were combined together. Figure 5-5 shows the break points for flow rate of through traffic when comparing average delay for DLT and RTUT.

As shown in Figure 5-5, the breakpoints for delay of these two left turn options can be estimated as follows:

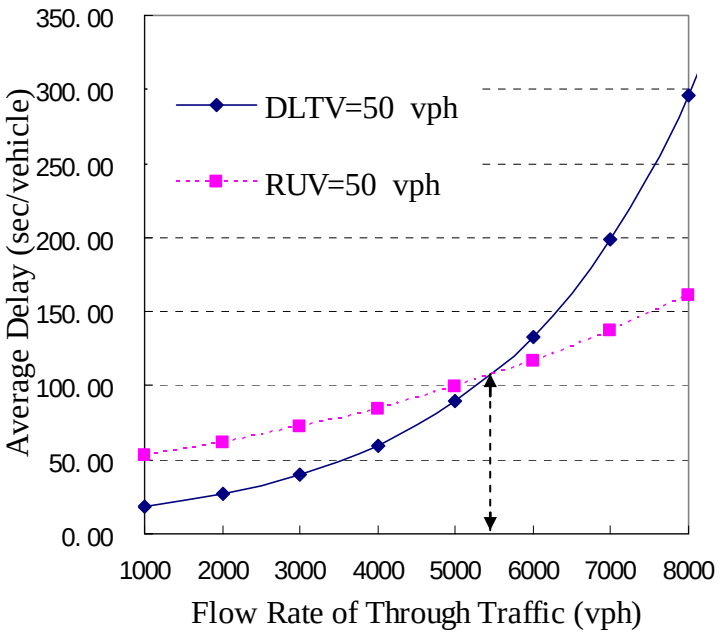
(1) When both DLT and RTUT flow rates are equal to 50 vph, the average total waiting delay of RTUT is greater than that of DLT until the major-road through-traffic flow rate is greater than 5500 vph;

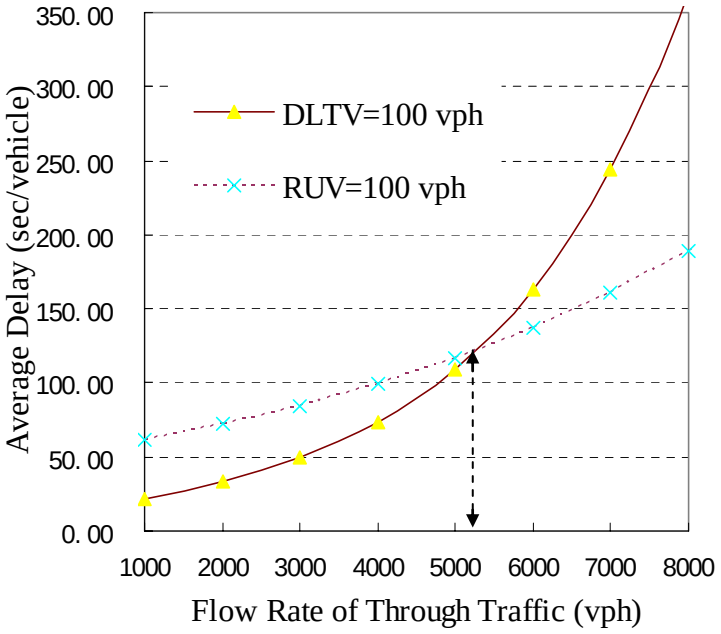
(2) When both flow rates are equal to 100 vph, RTUT has less delay than DLT when the through-traffic flow rate is more than 5200 vph; and

(3) When both flow rates are equal to 150 vph, RTUT will suffer less delay when the through-traffic flow rate is about 5000 vph.

5.2.4 Delay Comparison of two U-turn Approaches

In this chapter, the operations of two widely used U-turn approaches, U-turns at median opening in advance of signalized intersection and U-turns at signalized intersection, were compared based on the models developed in this study and those from the project performed by the University of South Florida in 2001. Curves for average delay of RTUT at median openings and RTUT at downstream stream signalized intersections were developed.


350. 00
300. 00
250. 00
200. 00
150. 00
100. 00
50. 00
0. 00
Average Delay (sec/vehicle)
DLTV=50 vph
RUV=50 vph
1000
2000
3000
4000
5000
6000
7000
8000
Flow Rate of Through Traffic (vph)


350. 00
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50. 00
0. 00
Average Delay (sec/vehicle)
DLTV=100 vph
RUV=100 vph
1000
2000
3000
4000
5000
6000
7000
8000
Flow Rate of Through Traffic (vph)

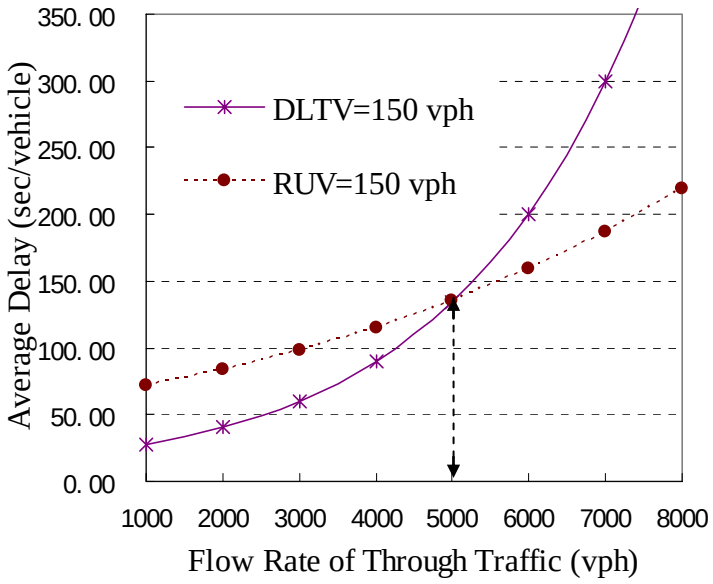


Figure 5-5 Comparison of Average Delay of Two Movements

Figure 5-6 shows a group of curves for average delay of RTUT assuming the g/c ratio is 0.15, cycle length is 120 sec, SPLIT is 0.5, left-turn flow rate from inside left turn lane is 100 vph, the distance form driveway to downstream signal is 560 ft, and the flow rate of RTUT 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 RTUT.

As shown in this figure, providing right turn followed by U-turn at downstream signalized intersection will suffer longer delay than the condition where U-turn is accommodated at downstream median opening in advance of signalized intersection.

5.3 Average Total Travel Time

In this project, the average total travel time of DLT is defined as the sum of average total waiting delay and the time for DLT vehicles crossing the through lanes. The average total travel time for RTUT includes the average total waiting delay, the running time from

vehicle leaving the driveway until it stops at exclusive left turn bay, plus the travel time from U-turn bay back to the median opening at driveway.

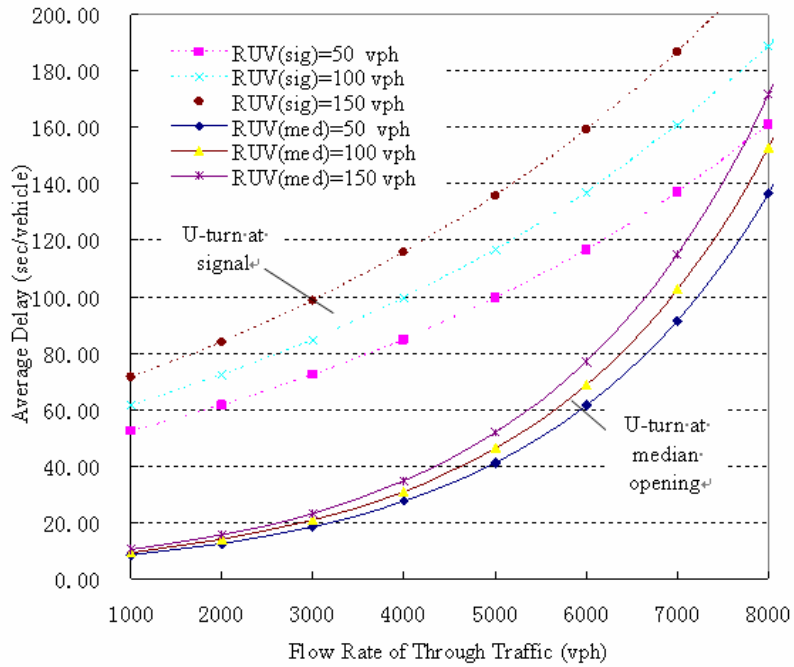


Figure 5-6 Comparison of Average Delay of Two U-turn Approaches

5.3.1 Travel Time Model for DLT

Data collected from the sites with full median openings were used to build travel time model for direct left turn movements. The dependent variable is the average total travel time for DLT movements at fifteen-minute intervals. The independent variables include 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. The same datasets for the delay models were used to develop the travel time model for DLT and RTUT.

$$TT_L = e^{a_1 TV + a_2 PLIT + a_3 DLTV + a_4 LTIN + a_0} \quad \text{(5-5)}$$

Where,

TT_L = average total travel time of DLT (sec/ veh);

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

DLTV = 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, and

a_0, a_1, a_2, a_3, a_4 = parameters

A total of 459 observations at fifteen-minute intervals were used to perform the regression analysis. The statistical characteristics of collected data are given in Table 5-5. The final regression results are listed in Table 5-6.

Table 5-5 Descriptive Statistics of the Collected Data

	TT	VOLUME	SPLIT	DLTV	LTIN
Valid	459	459	459	459	459
Missing	0	0	0	0	0
Mean	44.03	3437.75	.51	43.55	48.18
Median	42.08	3441.33	.50	40.00	44.00
Mode	33.00	3200.00	.44(a)	36.00	36.00
Std. Deviation	16.06	615.05	.049	22.29	20.78
Variance	257.86	378282.70	.002	496.71	431.70
Range	82.50	3080.00	.22	112.00	116.00
Minimum	14.50	1884.00	.39	8.00	8.00
Maximum	97.00	4964.00	.62	120.00	124.00

As shown in Table 5-6, all independent variables are significant at a 95 percent level of confidence. The adjusted R square value is 0.423, which implies that the selected independent variables can explain 42.3% of variations in dependent variable. The independent variable SPLIT has a negative coefficient, which suggests that the downstream through-traffic flow rate (TV2) has a greater impact on the total travel time than corresponding upstream flow rate (TV1). According to these parameter estimates, the final developed regression equation was shown in Equation 5-6.

Table 5-6 Regression Results for Travel Time Models of DLT

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.654(a)	.428	.423	.33473

a Predictors: (Constant), LTIN, VOLUME, DLTV, SPLIT

ANOVA(b)

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	38.041	4	9.510	84.882	.000(a)
	Residual	50.867	454	.112		
	Total	88.908	458			

a Predictors: (Constant), LTIN, VOLUME, DLTV, SPLIT

b Dependent Variable: lnTD

Coefficients(a)

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	2.369	.184		12.862	.000
	TV	.000	.000	.532	11.811	.000
	SPLIT	-.996	.416	-.110	-2.395	.017
	DLTV	.004	.001	.212	4.772	.000
	LTIN	.004	.001	.195	4.441	.000

a Dependent Variable: lnTD

$$TT_L = 10.69e^{0.00038TV - 0.996SPLIT + 0.0042DLTV + 0.0041LTIN} \quad \text{(5-6)}$$

Where,

TT_L = average total travel time of DLT (sec/ veh);

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

DLTV = flow rate of DLT from a driveway (vph);

LTIN = flow rate of left-turn-in from major road (vph); and

SPLIT = percentage of upstream through traffic flow rate.

Based on Equation 5-6, curves for the average total travel time of DLT can be developed. Figure 5-7 shows a group of curves for average total travel time of DLT assuming the left-turn-in flow rate from the major road is 100 vph, split is 0.5, and the flow rate of DLT is 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 travel time of DLT.

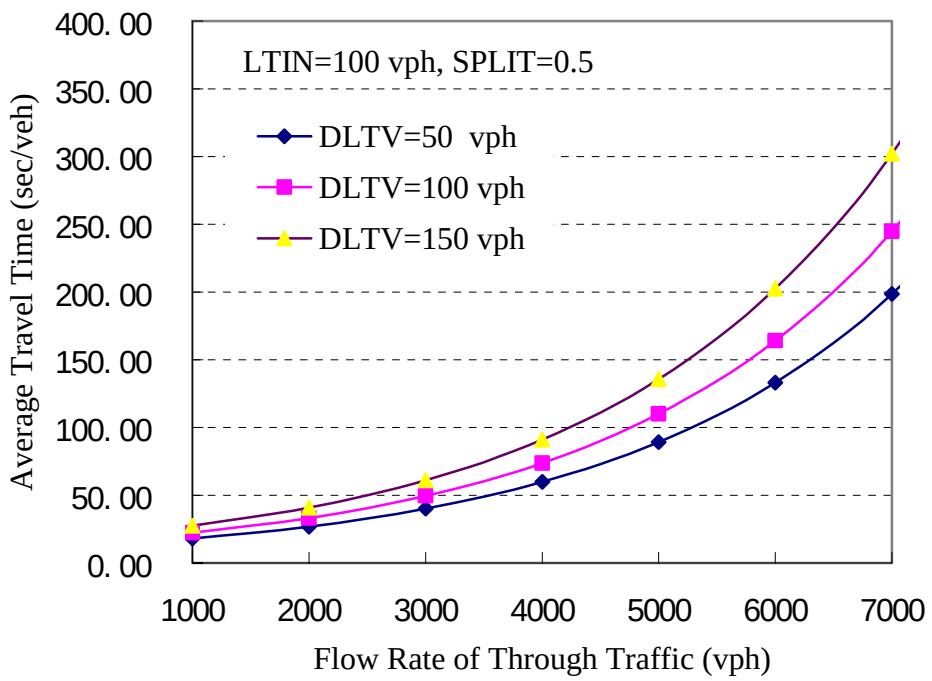


Figure 5-7 Curves for the Average Total Travel Time for DLT

5.3.2 Travel Time Model for RTUT

The average total travel time of RTUT includes the average total waiting delay, the running time from vehicle leaves driveway until it stops at exclusive left turn bay, plus the travel time from U-turn bay back to median opening at driveway. The statistical

characteristics of collected data are given in Table 5-7. The regression results are listed in Table 5-8.

The t-stat indicated that most of the independent variables are significant at a 95% level of confidence except G/C, which is significant at an 80% level of confidence. The coefficient of SPLIT is positive, which indicates that the upstream through-traffic flow rate (TV1) has a greater impact on the travel time than corresponding downstream stream flow rate (TV2). The coefficient of L is very small (-0.00046), which suggests that travel time of RTUT is not sensitive to the distance from driveway to downstream signalized intersection. As mentioned early in this chapter, the RTUT will take less delay at driveway when the downstream signal is located at a suitable distance from driveway. Therefore, when L increased, the total travel time of RTUT could decrease because of the reduced delay at the driveway. This conclusion may not be hold when this distance is getting too long. In this condition, it will take long time for RTUT to traverse the weaving section, and the total travel time of RTUT could increase. The maximum L of the selected sites is 900 ft. It is not easy to find a suitable distance from driveway to downstream signalized intersection or U-turn median opening, because the number of factors that needs to be considered besides delay and travel time of RTUT. Another research project concerning this topic will be conducted later. The empirical equation based on the regression results is as follows:

$$TT_{RU} = 137.8e^{0.00013TV+0.192SPLIT+0.0023RUV+0.0014LTV-0.691G/C+0.0061C-0.00032L-0.032SPEED} \quad \text{... (5-7)}$$

where,

TT_{RV} = average total travel time of RTUT (sec/ veh);

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

RUV = flow rate of RTUT from a driveway (vph);

LTV = left-turn flow rate from inside left turn lane;

G/C = g/c ratio for exclusive left turn phase;

C = Cycle length (sec); cycle length used in pretimed signal control, or average cycle length for actuated control;

SPLIT = percentage of upstream through traffic flow rate;

L = distance from driveway to downstream signalized intersection (ft); and

SPEED = speed limit along the arterial (mph).

Table 5-7 Descriptive Statistics of the Collected Data

	TT	VOLUME	SPLIT	RUV	LTV
Valid	610	610	610	610	610
Missing	0	0	0	0	0
Mean	106.19	3567.96	.48	18.39	109.60
Median	104.43	3514.67	.47	16.00	104.00
Mode	85.93(a)	3440	.41(a)	16.00	100.00
Std. Deviation	18.80	496.25	.07	5.81	43.39
Variance	353.33	246267.6	.004	33.78	1882.96
Range	99.87	3328	.31	40.00	268.00
Minimum	64.21	1580	.33	12.00	8.00
Maximum	164.08	4908	.63	52.00	276.00
Sum	64776.72	2176457	289.88	11215.33	66857.33

Based on Equation 5-7, curves for the average total travel time of RTUT can be developed. Figure 5-8 is an example which assumes that the g/c ratio is 0.15, cycle length for downstream signal is 120 sec, SPLIT is 0.5, left-turn flow rate from inside left turn lane is 100 vph, and the distance form driveway to downstream signal is 560 ft. Three different curves represent different volume conditions in which flow rate of RTUT is 50, 100, and 150 vph respectively. In Figure 5-8, the x-axis resents the flow rate of two-directional through-traffic on the major road. The y-axis represents the average total travel time of RTUT.

Table 5-8 Regression Results for Travel Time Models of RTUT

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
	.676(a)	.457	.450	.12992

a Predictors: (Constant), LTV, SPLIT, RUV, VOLUME, C, G/C, L, SPEED

ANOVA(b)

Model	Sum of Squares	df	Mean Square	F	Sig.
Regression	8.550	8	1.069	63.321	.000(a)
Residual	10.144	601	.017		
Total	18.695	609			

a Predictors: (Constant), LTV, SPLIT, RUV, VOLUME, C, G/C, L, SPEED

b Dependent Variable: lnTT

Coefficients(a)

Model	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
(Constant)	4.926	.221		22.247	.000
VOLUME	.000	.000	.356	10.309	.000
SPLIT	.192	.090	.072	2.135	.033
G/C	-.691	.504	-.058	-1.370	.171
C	.006	.000	.684	13.886	.000
L	.000	.000	-.287	-5.466	.000
SPEED	-.032	.006	-.324	-5.247	.000
RUV	.002	.001	.075	2.309	.021
LTV	.001	.000	.355	9.222	.000

a Dependent Variable: lnTT

5.3.3 Travel Time Comparison of DLT and RTUT

Given the travel time models for DLT and RTUT, the average total travel time of these two movements can be compared under different traffic and roadway geometric conditions.

Figure 5-9 is an example assuming that g/c ratio is 0.15, cycle length for downstream signal is 120 sec, SPLIT is 0.5, left-turn flow rate from inside left turn lane is 100 vph, and the distance form driveway to downstream signal is 560 ft. In this figure, the curves from Figure 5-7 and 5-8 were combined together.

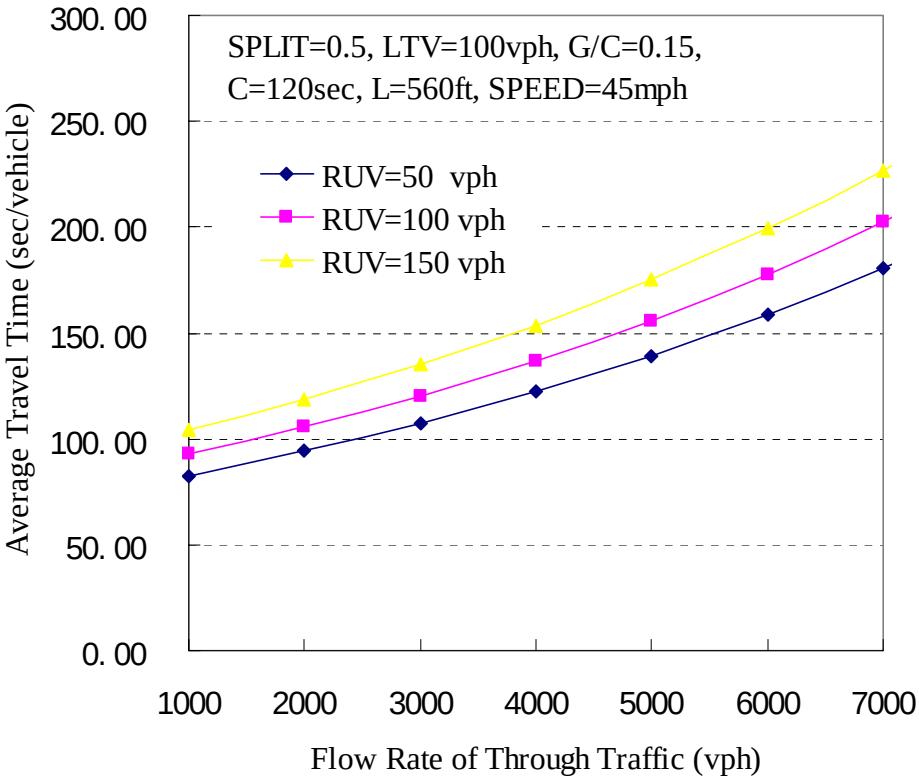


Figure 5-8 Curves for Average Total Travel Time for RTUT
(SPLIT=0.5, LTV=100vph, G/C=0.15, C=120sec, L=560ft, and SPEED=45mph)

As shown in Figure 5-9, the breakpoints for average total travel time of these two movements can be estimated 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 around 6600 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 about 6300 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 6000 vph.

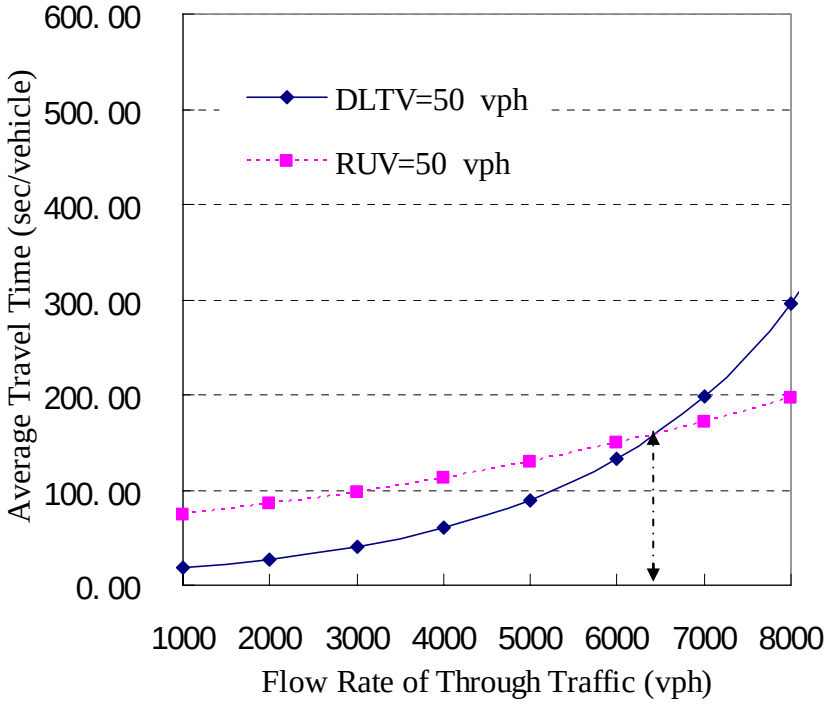
5.3.4 Travel Time Comparison of Two U-turn Approaches

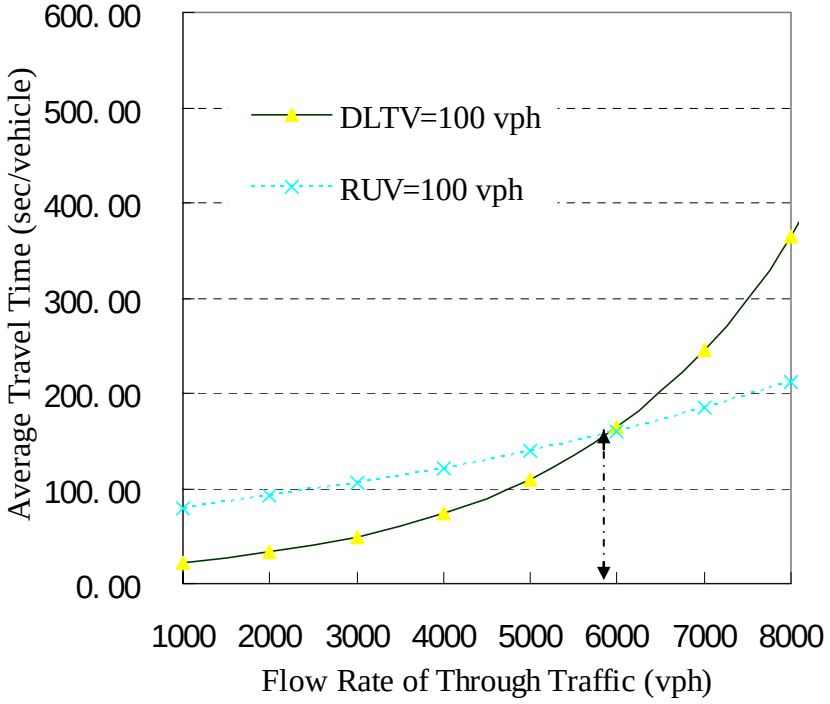
The operations of two widely used U-turn approaches, including U-turn at median opening in advance of signalized intersection and U-turn at signalized intersection, are analyzed by comparing the travel time model for RTUT developed in this project and the model from USF 2001 study. Figure 5-10 shows curves based on Equation 5-7 and the model from USF 2001 study as an example of comparison.

As illustrated in Figure 5-10, vehicles making a right turn followed by a U-turn at downstream signalized intersection will experience longer travel time than the condition in which U-turn is accommodated at downstream median opening in advance of signalized intersections.

5.4 Amount of RTUT under Both Choices

Based on the results from the USF 2001 study, 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. Therefore, the drivers' selection of a RTUT or a DLT will be affected by traffic volume conditions.


600. 00
500. 00
400. 00
300. 00
200. 00
100. 00
0. 00
Average Travel Time (sec/vehicle)
DLTV=50 vph
RUV=50 vph
1000
2000
3000
4000
5000
6000
7000
8000
Flow Rate of Through Traffic (vph)


600. 00
500. 00
400. 00
300. 00
200. 00
100. 00
0. 00
Average Travel Time (sec/vehicle)
DLTV=100 vph
RUV=100 vph
1000
2000
3000
4000
5000
6000
7000
8000
Flow Rate of Through Traffic (vph)

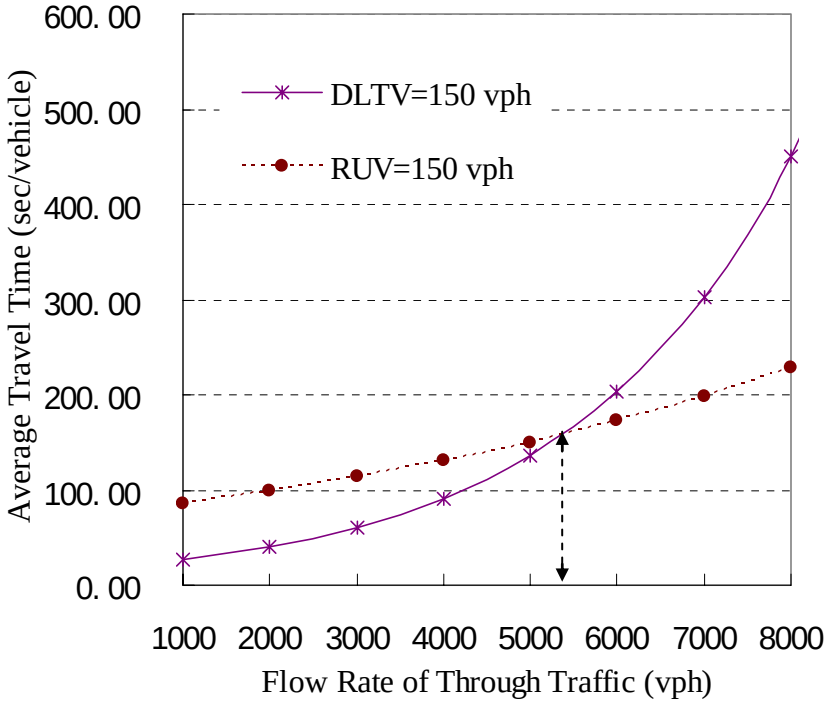


Figure 5-9 Comparison of Average Total Travel Time of Two Movements

For the condition in which U-turn is accommodated at signalized intersection, drivers' choice behavior may be different. As indicated earlier, drivers making U-turns at signalized intersections do not need to wait and find a suitable gap form the through-traffic in the other direction of the road, since U-turn is accommodated at protected signal phase. Some drivers may prefer this option with the perception that it is safer. However, there are also some drivers do not like making a U-turn at signalized intersection since they think it will result in longer delay and travel time.

In this study, a binary logistic regression model was developed to estimate under what traffic and roadway geometric conditions, more drivers would select RTUT rather than DLT. The reason for choosing logistic regression lies in the bounded nature of the dependent variable (the percentage of drivers selecting RTUT rather than DLT always varies between 0 and 1). In this model, 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 5-8.

RATIO= (Number of RTUT)/ (Number of RTUT+ Number of DLT)…..…… (5-8)

Data collected from sites 2, 3, 5, 6, and 7 were used to build this model, because these sites permit both of the DLT and RTUT movements. A total of 381 observations at fifteen-minute intervals were used to develop this model. Only intervals when both DLT and RTUT were chosen to perform the regression analysis. The regression results are given in Table 5-9

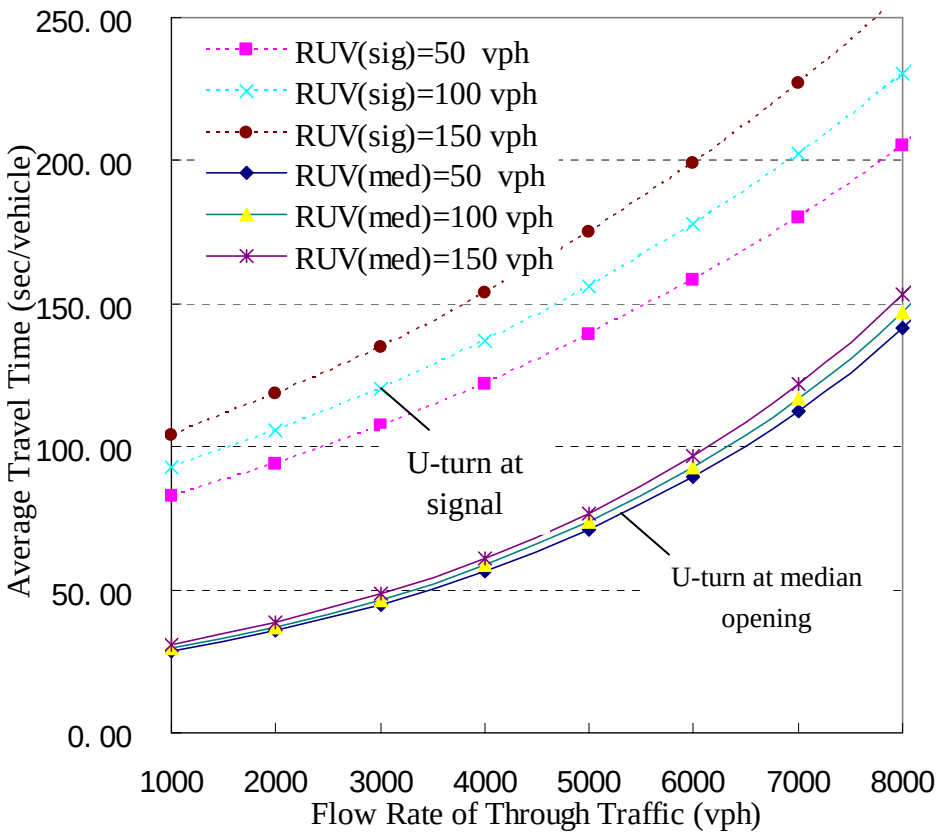


Figure 5-10 Comparison of Average Total Travel Time of Two U-turn Approaches

Table 5-9 Regression Results for Ratio of RTUT

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
	.712(a)	.507	.502	.45102

a Predictors: (Constant), SPLIT, L, TVOLUME, LTIN

ANOVA(b)

Model	Sum of Squares	df	Mean Square	F	Sig.
Regression	78.734	4	19.683	96.764	.000(a)
Residual	76.485	376	.203		
Total	155.219	380			

a Predictors: (Constant), SPLIT, L, TVOLUME, LTIN

b Dependent Variable: lnP

Coefficients(a)

Model	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
(Constant)	-1.162	.233		-4.99	.000
L	-.003	.000	-.931	-15.48	.000
LTIN	.014	.004	.864	14.03	.000
TV	6.262E-05	.000	.051	1.22	.224
SPLIT	1.933	.359	.202	5.38	.000

a Dependent Variable: lnP

T-stat indicated that most of the independent variables are significant at a 95% confidence level except the flow rate of through traffic, which is significantly at 75% level of confidence. The coefficient for SPLIT is positive, which indicates that the upstream through-traffic flow rate (TV1) has a greater impact on drivers' decision than corresponding downstream flow rate (TV2). In practice, when the downstream signal turns red, DLT vehicles waiting at median opening can easily find a gap from downstream through traffic and then make a left turn. This is why downstream traffic does not

significantly affect drivers' selection of RTUT as compared with upstream through-traffic flow rate. As shown in this model, the coefficient of the independent variable L, which represents the distance from driveway to downstream signalized intersection, is negative. In practice, some drivers prefer to make a RTUT rather than a DLT when the downstream signal is close to the driveway. This conclusion may not hold when this distance is too short, for example, shorter than the length of left-turn storage bay. In this condition, drivers wishing to make a RTUT are often blocked by through traffic already queued at the traffic signal. Drivers must cross the intersection, and then make a U-turn at a median opening downstream of the signalized intersection. This is another U-turn approach which needs to be considered and evaluated. More information regarding the distance from driveway to signalized intersection and U-turns after signal will be incorporated into another project. Based on the regression analysis, the final equation for estimating percentage of drivers selecting RTUT was:

$$\ln\left(\frac{RATIO}{1-RATIO}\right) = -1.162 + 0.014LTIN + 0.00006TV + 1.933SPLIT - 0.003L \quad \text{(5-9)}$$

Where,

Ratio = percentage of RTUT at fifteen-minute intervals

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

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

SPLIT = percentage of upstream through traffic flow rate

L = the distance from driveway to signalized intersection (ft)

Curves were developed based on Equation 5-9, assuming that SPLIT is 0.5, the distance from driveway to downstream signalized intersection is 560ft, and the flow rate of major road through traffic is made equal to 2000, 4000, and 6000 vph in both directions, respectively. As shown in Figure 5-11, the x-axis represents the flow rate of left-turn-in from major road. The y-axis represents the percentage of drivers choosing right turn

followed by a U-turn at downstream signalized intersection rather than making a direct left-turn from driveway when both choices are available.

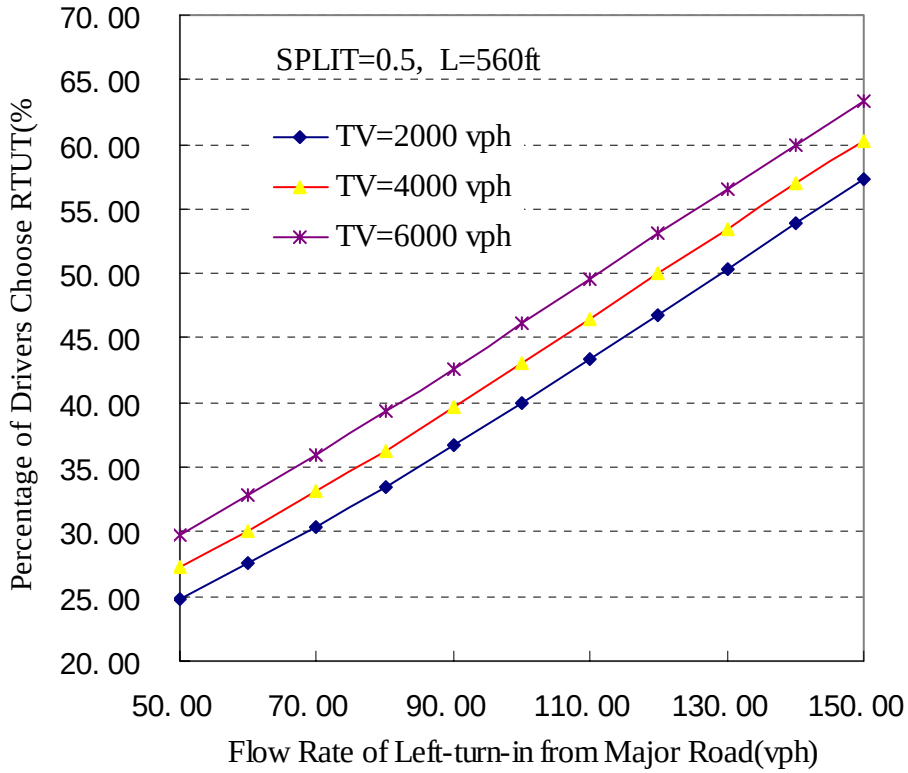


Figure 5-11 Ratio of RTUT vs. Left Turn in Volume and Through Volume

Based on the Figure 5-11, it is clear that:

(1) When the flow rate of major road through traffic is equal to 2000 vph, more drivers will select RTUT when the flow rate of left-turn-in from major road is about 130 vph or higher;

(2) When the flow rate of major road through traffic is equal to 4000 vph, the ratio is getting close to 50 percent when the flow rate of left-turn-in from major road is about 120 vph or higher; and

(3) When the flow rate of major road through traffic is equal to 6000 vph, the ratio is greater than 50 percent when the flow rate of left-turn-in from major road is larger than 110 vph.

5.5 The Effects of U-turns on Signalized Intersection Capacity

In this project, the effects of U-turns on signalized intersection capacity were estimated by applying adjustment factors for varying percentages of U-turning vehicles on left-turn saturation flow rate. The pilot survey conducted at early stage of this project indicated that U-turning vehicles have some adverse impacts on intersection capacity, and this effects increase with the increase of percentage of U-turning vehicles from inside left-turn lane. In addition, the pilot survey found that when U-turning vehicles are accommodated in the exclusive left-turn lane, the queue discharge patterns do not display an easily identifiable steady maximum rate. Therefore traditional headway research, which assumes that discharge flow rate reaches saturation state after the fourth or fifth discharged vehicle, cannot be directly used in estimating the adjustment factor for U-turning vehicles.

A procedure was developed to estimate the relationship between the percentage of U-turning vehicles in the left-turn lane and the average queue discharge time for each turning vehicle. The average queue discharge time was defined as the queue discharge time divided by the number of vehicles in the discharged queue as shown in Equation 5-10:

$$h = \frac{T}{N_u + N_l} \quad \text{(5-10)}$$

where,

h = average queue discharge time for each vehicle (sec);

T = queue discharge time (the time from the beginning of green until the rear axle of the last vehicle in queue crosses the stop line) (sec);

N_u = the number of U-turning vehicles in queue; and

N_l = the number of left-turning vehicles in queue.

The study team selected three intersections with exclusive left-turn lanes and protected signal phasing, and recorded discharge time for 260 queues, including 571 U-turning vehicles and 1441 left-turning vehicles. These data are used to build an empirical model which estimates the relationship between the percentage of U-turning vehicle and the average queue discharge time. Analysis showed that a second degree polynomial regression model is appropriate to describe the relationship. Figure 5-12 presents the distribution of collected data. The regression results are listed in Table 5-12. The average queue discharge time model was described as Equation 5-11.

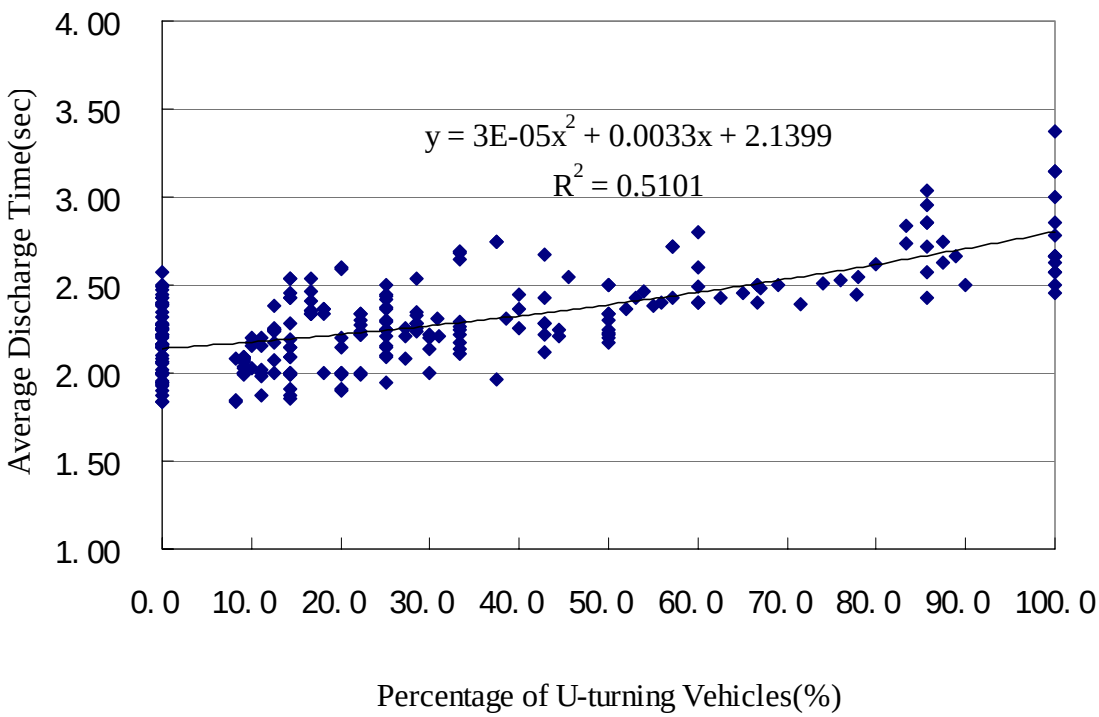


Figure 5-12. Average Queue Discharge Time versus the Percentage of U-turning Vehicles in Queue

$$h = 0.000033P_{UT}^{2} + 0.0033P_{UT} + 2.1399 \quad \text{(5-11)}$$

where,

h = average queue discharge time for U-turn and left-turn mix flow (sec);

P_{UT} = percentage of U-turn vehicles from inside left-turn lane (%);

$$P_{UT} = \frac{N_u}{N_u + N_l}$$, and

a_1, a_2, a_3, a_4 -parameters.

The dependent variable in this model is the average queue discharge time for each vehicle. Considering the intercept, which represents the base average queue discharge time assuming no U-turning vehicles in left-turn flow, this model provides a reasonable value of 2.14 sec. It is important to note that the definition of average queue discharge time in this model is different from that of the saturation headway, which is very difficult to be measured when there are U-turning vehicles in discharging queue. The adjusted R square value of this model is about 0.51.

Based on the definition of adjustment factors for turning movements, the U-turn adjustment factor for the left-turn saturation flow rate can be estimated by the following equation:

$$f_{UT} = \frac{\frac{3600}{h}}{\frac{3600}{h_0}} = \frac{h_0}{h} = \frac{2.1399}{0.00003P^2{}_{UT} + 0.0033P_{UT} + 2.1399} \quad \text{……. (5-12)}$$

where,

f_{UT} = adjustment factor for U-turn movement;

h = average queuing discharge time for U-turn and left-turn mix flow;

h_0 = base average queuing discharge time for left-turn only flow (sec); and

P_{UT} = percentage of U-turn vehicles from inside left-turn lane (%).

Table 5-10 the Regression Results for the Average Queue Discharge Time

Variables Entered/Removed(b)

Model	Variables Entered	Variables Removed	Method
	Put, Put2(a)	.	Enter

a All requested variables entered.

b Dependent Variable: h

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
	.714(a)	.510	.506	.18425

a Predictors: (Constant), Put, Put2

ANOVA(b)

Model	Sum of Squares	df	Mean Square	F	Sig.
Regression	9.085	2	4.542	133.813	.000(a)
Residual	8.724	257	.034		
Total	17.809	259			

a Predictors: (Constant), Put, Put2

b Dependent Variable: h

Coefficients(a)

Model	Unstandardized Coefficients		Standardized Coefficients	t	Sig.
	B	Std. Error	Beta		
(Constant)	2.140	.021		100.32	.000
Put2	3.337E-05	.000	.355	2.480	.014
Put	.0033	.001	.367	2.564	.011

a Dependent Variable: h

By using the Equation 5-12, the U-turn adjustment factor for different percent of U-turning vehicles are calculated and listed as follows:

Table 5-11 Adjustment Factor for U-turn movements

P_{UT} (%)	5	10	20	30	40	50	60	70	80	90	100
f_{UT}	0.99	0.98	0.96	0.94	0.92	0.90	0.87	0.84	0.82	0.79	0.76

From this table, it is clear that U-turning vehicles have considerable effects on the traffic flow in a left-turn lane, especially when the percent of U-tuning vehicles is high (>40%). Therefore, when estimating the left-turn lane capacity, it is essential to account for the effects of U-turning vehicles. This effect can be quantified by applying adjustment factors for U-turn movements on left-turn saturation flow rate, as those listed in Table 5-11.

5.6 Summary

Four major conclusions could be yielded in this chapter including:

(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) By comparing delay and travel time models developed in this chapter with those from the 2001 project, it is clear that providing RTUT at median opening in advance of signalized intersection will experience less delay and travel time as compared with the condition where U-turns are accommodated at signalized intersections;

(3) The driver selection of a RTUT or a DLT on the basis of accessibility is affected by both traffic flow and roadway geometrics. A binary logistic regression model was developed to estimate the relationship between the percentage of RTUT and explanatory variables; and

(4) The effects of U-turns on left-turn lane capacity can not be ignored especially when there is a large percentage of U-turning vehicles (>40%). A second degree polynomial regression model was developed to estimate the relationship between the average queue discharge time for each vehicle and the percent of U-turning vehicles in left-turn lane. Adjustment factors for varying percents of U-turning vehicles in left-turn lane are established by using this model.

6 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

Florida uses restrictive medians and directional median openings in the State Highway System to manage left turn egress movements from driveways and side streets. By installing raised curb medians and replacing full median openings with directional median openings in some places, direct left turn movements are substituted by making a right-turn followed by a U-turn at downstream median opening or signalized intersection.

This report is one of the reports that evaluate the safety and operational effects of a widely used access management treatment: U-turns at downstream signalized intersections as alternatives to direct left turns. The focus of this report is on the operational effects of U-turns at signalized intersection, on six to eight lanes urban or suburban arterials. The primary objectives of this study were to explore methodologies for evaluating the operational effects of U-turns at signalized intersections as alternatives to direct left turns; and to provide information on the potential operational impacts of these alternatives under various conditions.

To achieve these objectives, field studies were conducted at eight roadway segments in the Tampa Bay area in Florida. Over 300 hours of traffic data were collected using video cameras. While reviewing videotapes, each vehicle coming from the driveway making DLT or RTUT was tracked. Delay and travel time for each vehicle making DLT or RTUT were recorded. Other information gathered in the field study included traffic volumes, signal parameters, and roadway geometrics.

Delay and travel time models were developed based on collected field data. The delay and travel time of DLT and RTUT were determined as a function of conflicting volumes, signalization conditions, and roadway geometrics. Curves were developed based on

regression results depicting operational differences between vehicles making a DLT versus those making a RTUT. The curves demonstrated the break points at which drivers making a right turn followed by a U-turn at downstream signalized intersection experienced less delay and travel time than those attempting to make a direct left turn through a median opening onto a major road.

In this project, the operations of two widely used U-turn approaches: U-turn at median openings in advance of signalized intersections and U-turn at signalized intersection, were also evaluated by comparing the delay and travel time models developed in this study with those from the USF 2001 study. Based on the comparison, it is clear that providing RTUT at median openings in advance of signalized intersections will experience less delay and travel time as compared with the condition where U-turns are accommodated at signalized intersections.

Drivers' selection of RTUT or DLT may be affected by traffic characteristics such as through-traffic volume, left-turn-in volume, and others. In addition, field measurement found that drivers' choice of RTUT is also affected by the offset distance from driveway to downstream signalized intersections. A binary logistic regression model was developed to estimate the percentage of the drivers would like to make a RTUT rather than a DLT under some particular traffic and roadway geometric conditions. The findings indicate that the left-turn volume off major road onto minor road had significant impacts towards increasing the amount of RTUT. Additionally, the regression model developed in this study also indicated that fewer drivers would select RTUT when the offset distance from driveway to downstream signalized intersection is relatively long (>1500 ft).

In order to estimate the effects of U-turns on signalized intersection capacity, a second-degree polynomial regression model was developed to determine the relationship between the average queue discharge time for each vehicle and the varying percentages of

U-turning vehicles in the left-turn traffic stream. Adjustment factors for varying percentages of U-turning vehicles on left-turn saturation flow rate were also developed by using this model, as shown in Table 5-11. The results could be directly used in estimating the capacity reduction due to the presence of U-turning vehicles at a signalized intersection.

6.2 Conclusions

This study developed a procedure to estimate the operational effects of a widely used median treatment: U-turns at signalized intersection as alternatives to direct left turns. Though this study, conclusion can be made that U-turns at signalized intersection could have better operational performance than direct left turns under certain traffic and roadway geometric conditions. More specifically, the findings of this study include:

(1) The curves based on delay and travel time models indicated that under high major road through traffic (>5500 vph in both directions) and driveway volume (>150 vph) conditions, vehicles making a direct left turn could experience longer delay and travel time than those making a right turn followed by a U-turn at downstream signalized intersection. The difference can be quantified by using the delay and travel time models developed in this study;

(2) When major road and driveway volume are not high, vehicles making a right turn followed by a U-turn at downstream signalized intersection will experience longer delay and travel time than those making a direct left turn from the driveways and side streets. The break point, which indicates the specific volume condition in which DLT will have more delay than RTUT, can be estimated by using the delay and travel time models developed in this project;

(3) Considering the selection of different U-turn approaches, providing U-turns at median opening in advance of signalized intersection, at some locations, may provide more efficient traffic flow as compared with the condition in which U-turns are accommodated at signalized intersection;

(4) The percentage of RTUT movements increases with the increase of left-turn-in flow rate from major-road and major-road through-traffic flow rate; and decreases with the distance from driveway to downstream signalized intersection; and

(5) When estimating the left-turn lane capacity, it is essential to account for the effects of U-turning vehicles; especially when the percentage of U-turning vehicles is large (>40%). This effect can be quantified by applying adjustment factors for U-turn movements on left-turn saturation flow rate. In this project, a second-degree polynomial regression model was developed to estimate the relationship between the average queue discharge time for each vehicle and the percent of U-turning vehicles in left-turn lane. Adjustment factors for varying percents of U-turning vehicles in left-turn lane are established by using this model.

6.3 Recommendations

The findings of this study are helpful in providing local and state transportation agencies with recommendations for the design and selection of median treatments in six to eight lanes urban or suburban arterial roads. The potential median treatments include the installation of restrictive medians, closing median openings, and replacing a full median opening with a directional one. Delay and travel time models 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 under which the right turns followed

by U-turns take less time than direct left turns from driveways or side streets. Adjustment factors for varying percentages of U-turning vehicles developed in this study can be directly used in estimating the capacity of a signalized intersection where U-turns are accommodated.

The percentage of drivers selecting RTUT was found to be greatly affected by Left-turn-in volume from major road. For example, when the flow rate of major road through traffic is equal to 4000 vph, more drivers (>50%) tend to select RTUT when the flow rate of left-turn-in from major road is higher than 120 vph. In addition, the left-turn-in volume also has a dramatic impact on the delay of left turn out from driveways. Field measurement found that DLT drivers often refuse to yield to left-turn-in vehicles especially when the vehicle's delay at driveway is increasing. Sometimes it will cause accident. These findings indicated that left-turn-in volume could be an important indicator when considering replacing a full median opening with a directional one.

This study found that vehicles making a right turn followed by a U-turn at downstream median opening in advance of signalized intersection experience less delay and travel time as compared with the condition where U-turns are accommodated at signalized intersections It was also found that providing U-turns at signalized intersection will not only increase U-turning vehicle's delay and travel time, but also adversely affect the capacity of signalized intersection. From the author's point of view, providing U-turns at a median opening in advance of signalized intersection has more potential operational benefits than accommodating U-turns at a signalized intersection, as long as the distance from U-turn median opening to signal is not too close. However, it is also important to note that the operational performance of RTUT or DLT is not the only criterion for design and selecting median treatment. For example, vehicles making a right turn followed by a U-turn will experience long delay and travel time if the offset distance from driveway to downstream median opening or signalized intersection is too long. Field measurement

found that an offset distance of 600 ft is suitable from the operations point of view; in that it will not cause extra long delay and travel time to U-turning vehicles. However, such distance may not be enough from safety point of view. In this condition, decisions should not be made only from operations standpoint. In general, when selecting a median treatment, safety should have the first priority in decision making.

Several issues were not addressed in this study including operational effects of U-turns at four-lane road, the selection of the optimum offset distance from driveway to downstream signalized intersection or median opening, and the operational evaluation of another widely used U-turn approach: right turns followed by U-turns at a median opening after downstream signalized intersection. These issues would be addressed in future study.

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