Evaluation of the Operational Effects of U-Turn Movement

by

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of the requirements for the degree of
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Keywords: capacity, delay, travel time, gap acceptance, follow-up time, adjustment factor, directional median opening, crash rate, access management

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DEDICATION

To my motherland: China

To my loving parents: Genyuan Liu and Hua Sun.
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EVALUATION OF THE OPERATIONAL EFFECTS OF U-TURN MOVEMENT

Pan Liu

ABSTRACT

In Florida, the increased installation of non-traversable medians and directional median opening has produced an increased number of U-turns on multilane highways. Arguments have been advanced by some opponents of median modification projects that the increased numbers of U-turns may result in safety and operational problems on multilane highways.

The primary objective of this study is to evaluate the operational effects of U-turn movement on multilane roadways. To achieve this research objective, extensive data were collected. Field measurements were conducted at 40 sites in the Tampa Bay area of Florida to collect traffic operations data. Besides, the crash histories of 179 selected roadway segments in central Florida were investigated. Statistical analysis was conducted based on the collected traffic operations data and crash data to quantitatively evaluate the operational performance of U-turn movement.

Delay and travel time were compared for different driveway left-turn alternatives that are widely used in Florida and nationally. Crash rate models were developed to evaluate how the separation distance between a driveway exit and the downstream U-turn bay impacts the safety performance of vehicles making right-turns followed by U-turns.
(RTUT). With the crash data analysis results, the minimum separation distances under different roadway conditions were determined to facilitate driver use of RTUTs. The capacity of U-turn movement was analyzed under two different situations: (1) U-turns are provided at a signalized intersection; and (2) U-turns are provided at an unsignalized intersection. Adjustment factors were developed to quantify the impacts of the presence of U-turning vehicles on the capacity of a signalized intersection. The critical gaps and follow-up time for U-turn movement at unsignalized intersections were estimated. With the estimated critical gaps and follow-up time, the Harders model was used to determine the capacity of U-turn movement at an unsignalized intersection. This study also looks extensively at the minimum roadway width and median width required by vehicles to perform U-turn maneuvers on 4-lane divided roadways. It was found that a roadway width of 46 ft is generally sufficient for most types of design vehicles (except heavy vehicles) to perform a continuous U-turn maneuver without impedance.
CHAPTER 1
INTRODUCTION

1.1 Background

During the past two decades, more and more state departments of transportation and local transportation agencies have started installing non-traversable medians and directional median openings on multilane highways. Since 1993, the Florida Department of Transportation (FDOT) mandated that all new or reconstructed multilane arterials with design speeds over 40 mph be designed with restrictive medians. By installing non-traversable medians and replacing full-median openings with directional median openings at various locations, Florida is limiting median openings to left-turns from the major arterials. Hence, drivers desiring to make direct left-turn egress (DLT) maneuvers from a driveway or a side street onto major arterials would need to turn right onto the major-street and then make U-turns (RTUT) at a downstream median opening or a signalized intersection, as shown in Figure 1-1.

The increasing installation of non-traversable medians and directional median openings reflects the increased attention given to access management. Access management is defined as the systematic control of the location, spacing, design, and operation of driveways, median openings, interchanges, and street connections to a roadway. It also
involves roadway design applications, such as median treatments and auxiliary lanes, and the appropriate spacing of traffic signals (TRB, 2003).

![Three Different Driveway Left-turn Alternatives](image)

Figure 1-1. Three Different Driveway Left-turn Alternatives

The purpose of access management is to provide vehicular access to land development in a manner that preserves the safety and efficiency of the transportation system (TRB, 2003). During the past few decades, more and more state departments of transportation came to realize the importance of access management to the modern traffic system and began to use various access management techniques to improve the traffic operations and safety along major arterials. Many states have developed or are considering developing their statewide comprehensive access management programs. In 1979, the nation’s first system-wide 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 an important legal foundation of Florida statewide access management program (Sokolow, G., 1993).

One of the major principles of access management is to install non-traversable medians and directional median openings. The purpose of using non-traversable and
directional median openings is to eliminate problems associated with left-turns and crossing movements on multilane highways. As a result of this design decision, drivers desiring to make direct left-turns at a driveway will be relocated to a downstream U-turn bay to make U-turns. Replacing a full median opening with a directional median opening will reduce conflict points from 32 to 8, as shown in Figure 1-2. Thus, it will simplify driving tasks and could significantly reduce crash rate (Vargas and Gautam, 1989).

In practice, however, the median modification projects, including installing non-traversable medians and replacing full median openings with directional median openings, could become controversial issues and sometimes they are difficult to be implemented. Some business owners believe that the loss of direct left-turn access would have some adverse impacts on their business. In addition, arguments have been advanced by some opponents of
median modification projects that the increased numbers of U-turning vehicles may result in safety and operational problems on multilane highways.

Recently, these issues are being hotly discussed. There have been considerable numbers of studies conducted in this area. Previous studies have demonstrated that the use of non-traversable medians and directional median openings have little or no overall adverse impacts on roadside business activities (Eisele et al., 1999, Rees et al., 2000, Williams, 2000, Levinson and Gluck, 2000, Patrick et al., 2002); and the increased numbers of U-turns at median openings and signalized intersections will not constitute major safety concerns (Kach, 1992; Levinson et al., 2000; Maki, 1996; Cluck et al., 1999; Lu et al., 2001; Lu et al. 2001, Potts et al., 2004; Carter et al., 2005).

Lu et al. compared the safety performance for two driveway left-turn treatments on 6-lane arterials. These two left-turn treatments are direct left-turns at a driveway and right turns followed by U-turns at a median opening (Lu et al., 2001). The research team examined crash history at 258 roadway segments and concluded that using right-turns followed by U-turns at a median opening as an alternative to direct left-turns from a driveway will reduce the crash rate by 26% and the injury/fatality crash rate by 32%.

A recent NCHRP research analyzed crash data at 481 conventional full median openings and 187 directional median openings and found that the crashes related to U-turn and left-turn maneuvers at median openings occur very infrequently (Potts et al., 2004). In urban arterial corridors, median openings experienced an average of 0.41 U-turn plus left-turn crashes per median openings per year. In rural arterial corridors, unsignalized median openings experienced an average of 0.20 U-turn plus left-turn crashes per median opening.
per year. Based on these limited crash frequencies, the researchers concluded that U-turns do not constitute a major safety concern at median openings.

Carter and Hummer examined U-turn crash history of 78 signalized intersections and found that 65 of the 78 sites did not have any collisions involving U-turns in the three-year study period. U-turn collisions at the remaining 13 sites ranged from 0.33 to 3.0 crashes per year (Carter et al., 2005). Researchers of that study concluded that U-turns do not have a large negative safety effect on signalized intersections.

During 2002 to 2004, a series of research projects concerning the safety and operational effects of U-turns were conducted by the University of South Florida (USF). The USF studies took three basic approaches in evaluating a widely used access management technique – using right-turns followed by U-turns as an alternative to direct left-turns from a driveways or a side street, including crash data analysis, conflict data analysis, and traffic operations data analysis. This dissertation presents some key findings of the USF studies. The focus of this dissertation is on the operational performance of U-turn movement.

1.2 Problem Statement

When a conventional full median opening is closed or replaced with a directional median opening; drivers desiring to make direct left-turns from a driveway onto a major-street will instead make right-turns followed by U-turns at a downstream U-turn bay. The U-turn bay could be located either at a median opening or at a signalized intersection. As compared with other turning movements, U-turn movement has a smaller turning radius. Consequently, vehicles making U-turns have slower turning speeds. Arguments have been
advanced by some opponents of median modification projects that using U-turns as alternatives to direct left-turns will remove the problems from driveways to downstream median openings or signalized intersections where the U-turn bays are located; and the increased number of U-turning vehicles may result in safety and operational problems on multilane highways.

There have been considerable numbers of studies conducted concerning the safety effects of U-turns. However, relatively fewer studies are available concerning the operational effects of U-turn movement. Current concerns with regard to the operational effects of U-turn movement include the following:

(1) drivers who lost the direct left-turn access could experience longer delay and travel time while making right-turns followed by U-turns;

(2) the increased numbers of U-turning vehicles could have some adverse impacts on the capacity of a signalized intersection;

(3) unsignalized intersections may not be able to handle large numbers of U-turning vehicles due to the limited capacity of U-turn movement, and the increased numbers of U-turns may result in congestion at unsignalized intersections; and

(4) some streets may have limited physical spaces such as narrow roadway widths and median widths to negotiate U-turns. Vehicles making U-turns at such locations may result in operational problems.

Besides, the separation distance between a driveway and the downstream U-turn bay is an important consideration for a driver deciding whether to make a right-turn followed by
a U-turn or a direct left-turn. If the separation distance is too short, vehicles making right-turns followed by U-turns do not have enough spaces to make comfortable lane changes; this may result in safety problems at the weaving section. On the other hand, a separation distance that is too great may result in a longer travel time and, thus, discourage drivers from making right-turns followed by U-turns. Currently, there are no regulations or guidelines for determining the optimal location of a U-turn bay to facilitate driver use of right-turns followed by U-turns.

It order to address the concerns with regard to the operational effects of U-turn movement, an extensive research is needed. The ambition of this dissertation is to address all these concerns mentioned above. In this study, field measurements were conducted at 40 selected sites in the Tampa Bay area of Florida, and crash data were investigated for 179 selected roadway segments in central Florida. Video cameras were set up in the field to collect traffic operations data. Over 1000 hours of traffic operations data were recorded. Statistical analysis was conducted based on field data to quantitatively evaluate the operational performance of U-turn movement. The research results will help traffic engineers and designers make decisions about the selection of different driveway left-turn alternatives on urban or suburban multilane highways, including: (1) direct left-turns at a driveway, (2) right-turns followed by U-turns at a median opening in advance of a signalized intersection, and (3) right-turns followed by U-turns at a signalized intersection. The research results will provide traffic engineers and designs efficient tools to analyze the capacity for U-turn movement at unsignalized intersections and signalized intersections. The research results
will also result in guidelines concerning the selection of the optimal location of a U-turn bay to facilitate driver use of right-turns followed by U-turns.

1.3 Research Objectives

The goal of this research is to evaluate the operational performance of U-turn movement. With the research results, concerns with regard to the operational performance of U-turn movement could be addressed. More specifically, the objectives of this study consisted of the following parts:

(1) To compare vehicle delay and travel time for various driveway left-turn alternatives which are widely used on Florida and nationally, including direct left-turns at a driveway, right-turns followed by U-turns at a downstream signalized intersection, and right-turns followed by U-turns at a downstream median opening in advance of a signalized intersection.

(2) To determine the optimal location of a U-turn bay to facilitate drivers use of right-turns followed by U-turns.

(3) To evaluate the effects of U-turning vehicles on the capacity of the exclusive left-turn lane at a signalized intersection.

(4) To develop a procedure for estimating the capacity of U-turn movement at unsignalized intersections.

(5) To evaluate the operational performance of U-turns on 4-lane divided roadways with narrow medians.
The research objectives and the expected result for each research objective is depicted in a flow chart in the following figure.

![Flow Chart](image)

**Figure 1-3. Research Objectives and Expected Results**

### 1.4 Scope of Research

This dissertation presents some key findings of three USF research projects about the safety and operational effects of U-turn movement. This research is focused on the operational performance of U-turning passenger cars. The operational effects of U-turning heavy vehicles were not considered. In addition, this research is limited to the urban and suburban environments. The operational performance of U-turn movement on multilane highways in a rural environment is not considered. The following basic requirements were applied when selecting sites for field measurements:
(1) The selected roadway segment should have a raised-curb median with either a full median opening or a directional median opening that can safely store waiting vehicles;

(2) Speed limit on the major-street is equal to or higher than 40 mph, because FDOT mandates that all new or reconstructed multi-lane arterials with design speeds over 40 mph be designed with restrictive medians; and

(3) There should be a left-turn storage lane at the median opening or signalized intersection where U-turns are provided. The condition where vehicles making U-turns from a major-road through-traffic lane is not considered in this study.

1.5 Outline of the Dissertation

This dissertation consists of eight chapters. Chapter 1 provides a brief introduction about the background and research objectives of this study. Chapter 2 describes a summary of the past studies conducted in the proposed area. Chapter 3 explains the methodology employed in achieving the research objectives. Chapter 4 focuses on the data collection and the data reduction procedures. In chapter 5, delay and travel time are compared for different driveway left-turn alternatives. Chapter 6 summarizes the results for crash data analysis. On the basis of the crash data analysis, the minimum separation distances between driveways and U-turn bays are determined to facilitate driver use of RTUTs. Chapter 7 presents the capacity analysis results for U-turn movement. Two different conditions were considered, including U-turns at unsignalized intersections and U-turns at signalized intersections. The
operational performance of U-turns on 4-lane divided roadways with narrow medians is also
evaluated in this chapter. Finally, Chapter 8 provides summaries, major conclusions and
recommendations resulting from this research.
CHAPTER 2
LITERATURE REVIEW

2.1 General

In this study, 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 regarding the operational performance of U-turn movement were also searched and reviewed. Generally, the references can be categorized into six parts, including: the current indirect left-turn treatments, the delay and travel time for vehicles making direct left-turns and right-turns followed by U-turns, the weaving issues related to right-turn followed by U-turn movement, the capacity of U-turn movement at signalized intersections, the capacity of U-turn movement at unsignalized intersections and the median and roadway width to facilitate U-turns.

2.2 Indirect Left-turn Treatments

Left-turn movement has long been considered as one of the major resources of traffic operations and safety problems on multilane highways. Past studies have indicated that left-turn maneuvers increase delay, conflicts, and crashes, and they reduce capacity and mobility in the major traffic. For example, as mentioned in the Access Management Manual, a total of
about 74% of access-related crashes were found to have left-turning vehicles involved in
(TRB, 2003).

In order to address the operational and safety issues related to direct left-turns, traffic
ingenieurs have often looked at other alternatives of facilitating left-turns such as median U-
turns also known as “Michigan U”, Bowtie, Super street, Paired Intersection, Jug handle and,
recently, right turns followed by U-turns. In Wisconsin, U-turns are not permitted at
signalized intersections. Instead, U-turn movements are provided at “pre-U-turn” median
openings near signalized intersections. Michigan uses U-turn channels on highways with
wide medians and prohibits all turning movements at signalized intersections. U-turn lanes
were provided downstream of signalized intersections. Two pictures regarding “Michigan
U” are given in Figure 2-1 and Figure 2-2. Increasingly, Florida is limiting unsignalized
median openings to left-turn ingress from the major arterials; hence, drivers desiring to make
direct left-turns onto the major-street from a driveway must turn right onto the major-street
and then make U-turns downstream.

Figure 2-1. Michigan “U” (Gluck et al., 1999)
In general, there are three different U-turn treatments including providing U-turns at signalized intersections, providing U-turns at a median opening in advance of a signalized intersection and providing U-turns at a median opening after a signalized intersection. As illustrated in the Florida Median Handbook (FDOT, 1997), U-turns can be provided at a signalized intersection when the median is of sufficient width and there is a low combined left-turn plus U-turn volume at the left-turn bay. Several issues need to be considered when U-turns are going to be provided at a signalized intersection: (1) the “right-turn-on red” restrictions should be considered for the side street; (2) the effects of U-turns on the operational performance of the signalized intersection should be considered; and (3) the signalization should not work against U-turns. Providing U-turns at a median opening in advance of a signalized intersection could alleviate the traffic congestion at signalized
intersections. However, it was found that providing U-turns in advance of a signalized intersection could result in two successive left-turn lanes. Thus, drivers desiring to make left-turns at the signalized intersection may mistakenly enter the U-turn lane. To overcome this problem, the U-turn median opening should be located at least 100 ft away from the signalized intersection.

In the NCHRP Report 420, there contains some discussions about the treatments of U-turns (Gluck et al., 1999). These U-turn treatments include:

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

2.3 Delay and Travel Time for DLT and RTUT

Delay and travel time are important measures of effectiveness (MOEs) for traffic operations. In practice, drivers often oppose being directed to make a right-turn followed by a U-turn due to the perception that it may result in longer delay at U-turn locations, or longer travel time due to the extra traveling distances. Several studies have been conducted to compare the delay and travel time for direct left-turns and right-turns followed by U-turns.
Gluck et al. developed and calibrated an analytical model to estimate the travel time savings when unsignalized left-turns are diverted for various distances (Gluck et al., 1999). The key findings of that study include the following:

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;

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. The values in Table 2-1 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; and

<table>
<thead>
<tr>
<th>Major-street (Two directions)</th>
<th>Left-Turn Exit</th>
<th>Delay per Vehicle (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1,000</td>
<td>50</td>
<td>20</td>
</tr>
<tr>
<td>1,000</td>
<td>100</td>
<td>25</td>
</tr>
<tr>
<td>2,000</td>
<td>50</td>
<td>200</td>
</tr>
<tr>
<td>2,000</td>
<td>100</td>
<td>530</td>
</tr>
</tbody>
</table>

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 veh/hr), 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.

Gluck et al.’s research is limited to the situation where vehicles make direct left-turns and right-turns followed by U-turns in a suburban or rural environment where there are no nearby traffic signals. The research results cannot be applied to the urban or suburban environments where the impacts from nearby traffic signals cannot be ignored.

As part of a research project conducted by Lu et al., delay and travel time were compared for two different driveway left-turn alternatives: direct left-turns at a driveway and right-turns followed by U-turns at a downstream median opening (Lu et al., 2001). The research team collected traffic data at ten selected sites in the Tampa-St. Petersburg-Clear Water metropolitan area in Florida. Based on the collected field data, linear regression models were developed to estimate the delay and travel time for DLT and RTUT under different traffic and roadway geometric conditions. The linear regression models are used to develop curves to compare the delay and travel time for direct left-turns and right-turns followed by U-turns. The key findings of that study include:

1. 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 veh/hr and the left-turn-in flow rate from the major-road is over 150 veh/hr; and
(3) There was about 15-22% less delay for the driveway left-turn drivers after a conventional full median opening was replaced with a directional median opening, and drivers were forced to make right-turns followed by U-turns at downstream median openings.

Lu et al.’s research has primarily focused on the situation where U-turns are provided at a median opening. The operational effects of U-turns at signalized intersections were not considered in that study. In addition, Lu et al.’s research is limited to the situation in which vehicles making U-turns on 6 to 8-lane divided roadways where vehicles making U-turn do not have any geometric restrictions such as a limited turning radius. The operational effects of U-turn movement on 4-lane divided roadways were not considered.

2.4 Weaving Issues Related to RTUT

The safety and operational performance of vehicles making RTUTs is very likely to be influenced by the length of the separation distance between the driveway and the downstream U-turn location. Even though several studies have been conducted concerning the safety and operational effects of U-turns, most of them have not focused on the effects of the separation distance. The NCHRP Report 420 analyzed the weaving patterns for vehicles making RTUTs under various separation distances (Cluck et al., 1999). These different weaving patterns are shown in Figure 2-3. In general, there are three different types of weaving patterns for RTUT, including:
(1) When the separation distance is short, which is less than the left turn deceleration lane on the major road, many drivers will select a suitable simultaneous gap in all through lanes and then make a direct entry into the left turn deceleration lane (Type A Weaving);

(2) When the separation distance is medium, which is not long enough for a RTUT maneuver make a comfortable lane change, many drivers will select a suitable simultaneous gap in all through lanes and then make a direct entry into the most inside lane (Type B Weaving); and

(3) When the separation distance is sufficiently long, drivers will select a suitable gap, turn into the right-side lane, accelerate to appropriate speed, and then make a lane change into the most inside lane (Type C Weaving).

Figure 2-3. Weaving Patterns for RTUT (Gluck et al., 1999)

There have been considerable numbers of studies conducted to analyze weaving on freeways. However, most of these methods are not directly applicable to analyze weaving that occurs in the non-freeway environments. The only previous study concerning the
weaving issues related to RTUT was conducted by Zhou et al. in 2003. Researchers of that study developed an analytical model for determining the optimal location of mid-block U-turn median openings on multilane divided roadways where the signalized intersections are coordinated (Zhou et al., 2003). A case study of that study showed that the average delay of U-turns will significantly decrease and the capacity of U-turns will increase if the U-turn median opening is located at an optimal location downstream of driveway. Zhou et al.’s study focused on selecting an optimal distance between a driveway and a downstream mid-block U-turn median opening such that the waiting delay of vehicles making RTUTs could be minimized. The findings of that study provided very useful insights on traffic operations and safety of right turn plus U-turns design. However, that study did not look specifically at the crash data and traffic conflicts occurred at weaving sections. Further work need to be conducted to evaluate the impacts of various separation distances on the safety performance of RTUT.

2.5 Capacity of U-turn Movement at Signalized Intersections

In the current edition of the Highway Capacity Manual (HCM), U-turns are treated as left-turns for estimation of the saturation flow rate (TRB, 2000). However, the operational effects of U-turns and left-turns are different. U-turning vehicles have smaller turning radii than left-turning vehicles. Consequently, vehicles making U-turns usually have lower turning speeds than those making left-turning. It is possible that the increased numbers of U-turns at signalized intersections could have some adverse impacts on the intersection capacity.
Adams and Hummer evaluated the effects of U-turns on left-turn saturation flow rates (Adams and Hummer, 1993). The research team selected four intersections with exclusive left-turn lanes and protected signal phasing and recorded the saturation flow rates and U-turn percentages for 198 queues during midday peaks on weekdays. The data analysis showed that “a saturation flow reduction factor appears necessary for left-turn lanes that had large percentages of U-turns. Saturation flow rates were significantly lower when queues have more than 65% U-turns”. However, the analyses also showed no correlation between the saturation flow and the percentage of U-turns for queues with 50% U-turning vehicles or less. The results of that study suggested 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. The researchers of that study also recommended that a follow-up investigation focus on intersections that have high percentages of U-turns, restrictive geometries, or high percentages of U-turning heavy vehicles.

Tsao and Chu recorded 600 headways of left-turning passenger cars and 160 headways of U-turning passenger cars in Taiwan (Tsao and Chu, 1996). It was found 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 on the percent of U-turning vehicles in the left-turn lane, as well as the order of formation in the traffic stream. When it is preceded by a left-turning vehicle, the average headway of a U-turning passenger car is 1.27 times that of a left-turning passenger car. When it is 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. Tsao and Chu assumed that the discharge flow rate of the vehicle reaches a
saturation state after the fourth or fifth discharged vehicle, and only the headways after the fifth discharged vehicle were recorded.

2.6 Capacity of U-turn Movement at Unsignalized Intersections

The only previous study concerning the capacity of U-turn movement at unsignalized intersections was conducted by Al-Masaeid in 1999 (Al-Masaeid, 1999). In Al-Masaeid’s study, regression models were developed to estimate the capacity and delay of U-turn movement at median openings and to investigate the effect of different relevant factors that might affect the estimated capacity and delay. The equations of the regression models are shown as follows (cf. Al-Masaeid, 1999):

\[ C = 1545 - 790e^{\frac{q_c}{3600}} \]  \hspace{1cm} (2-1)

where \( C \) represents the capacity of U-turn movement at a median opening (veh/hr), and \( q_c \) represents the conflicting traffic flow (veh/hr); and

\[ TD = 6.6e^{\frac{q_c}{1200}} \]  \hspace{1cm} (2-2)

where \( TD \) represents the average total delay for U-turning vehicles at a median opening (s), and \( q_c \) represents the conflicting traffic flow (veh/hr).

Al-Masaeid also estimated the critical gap and move-up time for U-turns and used them to calculate capacity on the basis of the 1994 edition of the Highway Capacity Manual. The author compared the results of gap acceptance model and regression model and
concluded that the gap acceptance model provided reasonable results for estimating the capacity of U-turns. Al-Masaeid’s study provided very useful information about the capacity of U-turn movement at unsignalized intersections. However, the author did not explain the procedures for estimating the critical gap and follow-up time for U-turns. In addition, Al-Masaeid’s study was conducted in Jordan; the results may not reflect the behaviors of motor vehicle drivers in the United States.

2.7 Median and Roadway Width to Facilitate U-turns

The minimum median and roadway width required to facilitate U-turning vehicles are key factors in determining whether U-turn movements can be permitted at a median opening or a signalized intersection. The AASHTO Green Book (A Policy on Geometric Design of Highways and Streets) contains some guidelines on the relationship between median width and U-turn maneuvers. As indicated in the AASHTO Green Book, medians of 5.0 m (16 ft) and 15 m (50 ft) or wider are needed to permit passenger car and single-unit truck traffic, respectively, to turn from the inner lane (next to the median) on one roadway to the outer lane of a two-lane opposing roadway. Also, a median left-turn lane is highly desirable in advance of the U-turn opening to eliminate stopping on the through lane. This scheme would increase the median width by approximately 3.6 m (12 ft) (AASHTO, 2001). The minimum widths of medians to accommodate U-turns by different design vehicles turning from the lane adjacent to the median are shown in Figure 2-4.

Wherever possible, a newly designed divided highway should have a median width that can accommodate normal left-turns and passenger car U-turns by using a sufficient
intersection design and a median storage lane that will protect and store the design-hour turning volume. If adequate median width does not exist for accommodating U-turns, then adding extra pavement width, through use of a taper, a flare or on the shoulder for example should be considered (FDOT, 1997). Two examples of the using curb and gutter to facilitate U-turns are shown in Figure 2-5 and Figure 2-6.

![Minimum Median Widths to Accommodate U-turns](image)

**Figure 2-4. Minimum Median Widths to Accommodate U-turns (AASHTO, 2001)**

Another treatment to facilitate the larger turning path of U-turning vehicles along narrow medians is the use of loons. As defined in the NCHRP Report 524, a loon is an expanded paved apron on the shoulder opposite a median crossover, as shown in Figure 2-7 (Potts et al., 2004). The purpose of installing loons is to provide additional space for larger vehicles (particularly trucks) to negotiate U-turns, and thus, to allow the installation of conventional or directional median openings along narrow medians. The provision of loons
to serve U-turns by large vehicles is a new technique that formalizes past use of paved shoulders for the same purpose.

Figure 2-5. Flare to Allow Design P-Vehicle to Make U-turn on 4-Lane Divided Roadway with Curb and Gutter (Florida, 1997)

Sisiopiku and Aylsworth-Bonzelet evaluated the operations, placement, and safety of existing loons at directional crossovers in Michigan. It was found that loons provide commercial vehicles with the extra pavement necessary to complete the U-turn maneuver; and the consistent placement of advance warning signs preceding the indirect crossover and associated loon assists in driver behavior. The research team of that study investigated crash data analysis at 7 crossovers installed with loons and indicated that directional crossovers with loons experienced a high percentage of fixed-object and sideswipe crashes. As a result of that study, the researchers developed the guidelines for the design and placement of loons using computer simulation.
Figure 2-6. Design for P-Vehicle U-turn on 4-Lane Divided Roadway with Curb and Bus Stop (Florida, 1997)

Figure 2-7. Conventional Median Opening with Left-turn Lanes and Loons at Three-leg Intersection (Potts et al., 2004)
2.8 Summary

There have been considerable numbers of studies conducted to evaluate the safety effects of U-turn movement. However, relatively fewer efforts have been made to understand the operational effects of U-turn movement. Previous studies concerning the delay and travel time effects associated with right-turns followed by U-turns have not focused on the situation where U-turns are accommodated at signalized intersections. There are limited numbers of studies concerning the capacity of U-turn movement at a signalized intersection. There is only one previous research conducted concerning the capacity of U-turn movement at unsignalized intersections. There are no regulations or guidelines for determining the optimal locations of U-turn bays on multilane highways to facilitate driver use of right-turn followed by U-turn. With the increasing installation of non-traversable medians and directional median openings, it is important to have a better understanding of the operational effects of U-turn movement.
CHAPTER 3

METHODOLOGY

3.1 General

This chapter documents the methodologies that are used to achieve the research objectives of this study. This chapter consists of four sections. The first section explains the definition of delay and travel time for different driveway left-turn alternatives which are currently used in Florida and nationally, including direct left-turns at a driveway, right-turns followed by U-tours at a median opening in advance of a signalized intersection and right-turns followed by U-turns at a signalized intersection. The second part of this chapter introduces the current capacity analysis procedures at signalized intersections. The third part of this chapter is focused on explaining the capacity estimation procedures at unsignalized intersections. The last part of this chapter explains the crashes that could occur at the weaving sections while vehicles making right-turns followed by U-turns.

3.2 Delay and Travel Time

Vehicle delay and travel time are important measures of effectiveness for traffic operational performance. There are many different definitions for vehicle delay. In the
second edition of the Traffic Engineering, the most frequently used definitions of delay include (Roess et al., 1998):

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

2. **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.

3. **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.

4. **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.

In the Highway Capacity Manual, control delay is used as the criteria for determining the level of service for both signalized intersections and unsignalized intersections. In HCM, 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 that can be 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 (TRB, 2000).

In this study, control delay and travel time are used as two measures of effectiveness for comparing the operational effects of three different driveway left-turn alternatives which are most commonly used in Florida and nationally. These driveway left-turn alternatives include direct left-turns at a driveway, right-turns followed by U-turns at a median opening in advance of a signalized intersection and right-turns followed by U-turns at a signalized intersection. The definitions of delay and travel time for a direct left-turn movement and a right-turn followed by a U-turn movement are explained in the following sections.

3.2.1 Delay and Travel Time for DLT

On multilane highways with non-traversable medians that can safely store left-turning vehicles, the direct left-turn movement is a two-stage left-turn process. As shown in Figure 3-1, a typical two-stage left-turn process requires three steps:

(1) Stopping and waiting at the driveway;
(2) Selecting a suitable gap in the traffic stream approaching from the major-street; crossing the major street and coming to a stop at the median opening; and
(3) Selecting a suitable gap in the traffic stream approaching from the right; and completing the left-turn movement.

Based on the analysis, delay for a complete direct left-turn maneuver is equal to delay at the driveway plus the delay at the median opening. Delay for a left-turning vehicle at the
driveway was measured from the time when a car stopped at the waiting queue until it crossed the stop line of the driveway. Delay for a left-turning vehicle at the median opening was measured from the time when the left-turning vehicle stopped at the median opening until the time when it left the median opening. The travel time for a complete direct left-turn maneuver is equal to the total delay plus the running time from when a vehicle leaves the driveway until it stops at the median opening.

![Figure 3-1. A Direct Left-turn Egress Movement](image)

### 3.2.2 Delay and Travel Time for RTUT

When a conventional full median opening is closed or replaced with a directional median opening, drivers desiring to make a direct left-turn from a driveway onto the major-street would instead make a right-turn followed by a U-turn at a downstream U-turn location. As shown in Figure 3-2, a typical right-turn followed by a U-turn movement requires four steps:
(1) Stopping and waiting at the driveway;

(2) Making a right-turn onto the major-street when a suitable gap is available from the left-side through-traffic;

(3) Accelerating to the operating speed of major-street, weaving to the inside lane, and decelerating to a stop in the exclusive left-turn lane at the downstream U-turn location; and

(4) If U-turns are accommodated at a signalized intersection, waiting until the signal turns green to complete the U-turn movement. If U-turns are accommodated at a median opening in advance of a signalized intersection, waiting until there is a suitable gap in the major-street to make a U-turn.

Figure 3-2. A Right-turn Followed by a U-turn Movement

Based on the analysis, delay for a complete RTUT procedure includes delay at the driveway and delay at the U-turn bay. Vehicle delay at the driveway was measured from the
time when a car stopped at the waiting queue until it crossed the stop line of the driveway. Delay for a U-turning vehicle at the U-turn bay was measured from when a car stopped at the U-turn bay until it started making a U-turn. The total travel time for a complete right-turn followed by U-turn procedure is equal to the total delay plus the travel time drivers spend at the weaving section.

The travel time a driver spends at the weaving section while making a right-turn followed by a U-turn consists of two parts: (1) the elapsed time from the time when a vehicle leaves the driveway until the time when it stops at the U-turn bay; and (2) the elapsed time from the time when a vehicle starts making a U-turn until the time when it finishes traversing the separation distance from U-turn bay to subject driveway at the speed of major-street through traffic, as shown in Figure 3-3.

![Figure 3-3. Vehicle Travel Time at the Weaving Section](image-url)
3.3 Capacity of U-turn Movement at Signalized Intersections

At a signalized intersection, U-turning vehicles are usually provided from the inside exclusive left-turn lane, as shown in Figure 3-4. Consequently, the capacity of U-turn movement at a signalized intersection depends on the capacity of the inside exclusive left-turn lane. The following two sections explain the current capacity analysis procedures at signalized intersections. A pilot survey was conducted at the early stage of this study to help to better understand the factors that could affect the capacity of U-turn movement at signalized intersections. The findings of the pilot survey are briefly introduced in the following sections.

Figure 3-4. U-turn Movement at a Signalized Intersection

3.3.1 Capacity Analysis at Signalized Intersections

In the current edition of the Highway Capacity Manual, saturation flow rate plays a very importation role in estimating the capacity of a lane or a lane group at signalized
intersections. In the Highway Capacity Manual, saturation flow rate is defined as “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” (TRB, 2000). Based on this definition, the saturation flow rate is the maximum flow rate that can pass through a given lane group under prevailing conditions.

The computations of saturation flow rate begin with the selection of a base saturation flow rate. The base saturation flow rate is the saturation flow rate for a lane under “ideal” conditions. Usually, the base saturation flow rate is assumed to be 1900 passenger cars per hour per lane (pc/hr/ln). In estimating saturation flow rate, different adjustment factors are applied to account for the prevailing conditions that do not meet the requirements of “ideal”. These adjustment factors include lane width and lateral clearance factors, number of lanes factors, heavy vehicles factors, grades factors, turning movement factors, interchange density factors, lane distribution factors, and environmental factors. A saturation flow rate for each lane group can be estimated according to the following equation (cf. TRB, 2000):

\[
s = s_0 N f_w f_{NV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lph} f_{Rph}
\]

where, \( s \) = saturation flow rate for a particular lane group (veh/hr),

\( s_0 \) = base saturation flow rate per lane (pc/hr/ln),

\( N \) = number of lanes in a lane group,

\( f_w \) = adjustment factor for lane width,

\( f_{HV} \) = adjustment factor for heavy vehicles in traffic stream,
\[ f_g = \text{adjustment factor for approach grade}, \]
\[ f_p = \text{adjustment factor for existence of a parking lane and parking activity adjacent to lane group}, \]
\[ f_{bb} = \text{adjustment factor for blocking effect of local buses that stop within intersection area}, \]
\[ f_a = \text{adjustment factor for area type}, \]
\[ f_{LU} = \text{adjustment factor for lane utilization}, \]
\[ f_{LT} = \text{adjustment factor for left turns in lane group}, \]
\[ f_{RT} = \text{adjustment factor for right turns in lane group}, \]
\[ f_{lpb} = \text{pedestrian adjustment factor for left-turn movements}, \]
\[ f_{rpb} = \text{pedestrian-bicycle adjustment factor for right-turn movements}. \]

As an alternative to estimating the saturation flow rate using Equation 3-10, saturation flow rate for each lane group can also be estimated by field measurement. The saturation flow rate that was measured in the field will usually produce more accurate results. Discharge headway method is the most commonly used method for the field measurement of saturation flow rate at signalized intersections.

In HCM, headway is defined as the time between successive vehicles as they pass a point on a lane or roadway (TRB, 2000). In practice, when the green signal is initiated, headways between departing vehicles can be observed as vehicles cross the stop line. The first headway is 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.

Many previous studies have demonstrated that the discharge headways will converge to a constant headway. The “constant headway” state 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, which is used to calculate the saturation flow rate. The relationship between saturation flow rate and saturation headway is shown in the following equation (cf. TRB, 2000):

\[
s = \frac{3600}{h}
\]

(3-2)

where, 
\( s = \) saturation flow rate (vphpl), and 
\( h = \) saturation headway (s),

\[
3600 = \text{seconds/hour.}
\]

With the saturation flow rate, the capacity of a particular lane or a lane group can be estimated using the following equation (cf. TRB, 2000):

\[
c_i = s_i \frac{g_i}{c}
\]

(3-3)

where, 
\( c_i = \) capacity of lane group i (veh/hr),
\( s_i = \) saturation flow rate for lane group i (veh/hr), and
\( g_i/c = \) effective green ratio for lane group i.
3.3.2 Pilot Survey

In the current edition of the Highway Capacity Manual, U-turns are treated as left-turns for estimation of the saturation flow rate (TRB 2000). However, the operational effects of U-turns and left-turns are different. A pilot survey was conducted at the early stage of this study to help us better understand the factors that could affect the capacity of U-turn movement at signalized intersections. A signalized intersection with dual exclusive left-turn lanes and protected signal phasing was selected for this survey. The intersection is located on Fowler Avenue in Tampa, Florida. The street is a 6-lane principle arterial road. The signal is actuated controlled with an average cycle length of 149 sec. The research team recorded discharge headways for 138 left-turning vehicles and 54 U-turning vehicles in 27 left-turn queues during weekday peak period.

In the field, when the green signal was initiated, the headways between departing vehicles were observed as vehicles crossed the stop line. The first headway was measured as the time between the initiation of the green signal and the crossing of the first vehicle over the stop line. The second headway was measured as the time between the times that the first and the second vehicles crossed the stop line. Usually any reference point could be used when the headways were recorded, as long as the identical point was maintained for all measurements. In this study, the rear wheel of the turning vehicle was used as the reference point in field measurements. To focus on the characteristics of passenger car flows, the data related to heavy vehicles and all vehicles behind a heavy vehicle were excluded from the analysis.
The collected data were classified into three different categories on the basis of different percentages of U-turning vehicles in the left-turn lane. Within each category, the average discharge headways by different queue positions were calculated, as shown in Table 3-1. The queue discharge patterns for the queues with different percentages of U-turning vehicles were shown in Figure 3-5.

Table 3-1. Discharge Headway by Queue Position

<table>
<thead>
<tr>
<th>$P_U^a$</th>
<th>Queue Position</th>
<th>Range of Discharge Headway (s)</th>
<th>Average Discharge Headway (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_U &gt; 50%$</td>
<td>1</td>
<td>1.00 ~ 3.00</td>
<td>2.20</td>
</tr>
<tr>
<td>2</td>
<td>1.98 ~ 3.59</td>
<td>2.80</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2.00 ~ 2.93</td>
<td>2.37</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.42 ~ 3.39</td>
<td>2.33</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1.52 ~ 3.81</td>
<td>2.56</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>2.21 ~ 3.93</td>
<td>3.09</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>1.71 ~ 2.64</td>
<td>2.23</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>1.90 ~ 1.96</td>
<td>1.93</td>
<td></td>
</tr>
<tr>
<td>$0% &lt; P_U \leq 50%$</td>
<td>1</td>
<td>1.84 ~ 4.33</td>
<td>2.47</td>
</tr>
<tr>
<td>2</td>
<td>1.89 ~ 3.40</td>
<td>2.46</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.95 ~ 3.21</td>
<td>2.55</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.23 ~ 3.50</td>
<td>2.27</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1.71 ~ 3.28</td>
<td>2.34</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>1.59 ~ 2.45</td>
<td>2.01</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>1.27 ~ 2.68</td>
<td>1.86</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>1.21 ~ 1.62</td>
<td>1.43</td>
<td></td>
</tr>
<tr>
<td>$P_U = 0%$</td>
<td>1</td>
<td>1.50 ~ 3.92</td>
<td>2.89</td>
</tr>
<tr>
<td>2</td>
<td>1.95 ~ 3.55</td>
<td>2.69</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>1.50 ~ 2.80</td>
<td>2.22</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>1.53 ~ 2.62</td>
<td>1.95</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>1.48 ~ 2.09</td>
<td>1.85</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>1.33 ~ 2.48</td>
<td>1.78</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>1.39 ~ 1.89</td>
<td>1.68</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>1.77 ~ 1.89</td>
<td>1.83</td>
<td></td>
</tr>
</tbody>
</table>

$^a$ $P_U = \text{Percentage of U-turning vehicles in the left-turn lane.}$
As mentioned before, the traditional headway method assumes that when a vehicle queue is released by a traffic signal that has turned green, the discharge flow rate of the vehicles quickly reaches a steady state. As shown in Figure 3-5, the average discharge headways for the vehicles in those queues with 0% of U-turning vehicles converge to a relatively constant state from the forth or fifth discharged vehicle after green onset. However, when left-turning vehicles are mixed with U-turning vehicles in the left-turn lane, the queue discharge patterns do not display an easily identifiable steady maximum rate; and it was difficult to measure saturation headway for the mixed traffic stream.

In addition, the curves in Figure 3-5 show that the average discharge headway increases with the percentage of U-turning vehicles in the left-turn lane. This could be
explained by the fact that the turning speeds of vehicles making U-turns and left-turns are different. Generally, a U-turn movement has a shorter turning radius than a left-turn movement. Consequently, a vehicle making a U-turn may have a lower turning speed. As a result, U-turning vehicles may consume more of the available green time and more of the lane’s available capacity than vehicles making left-turns. Field observation found that U-turning vehicles are often in conflict with the following left-turning vehicles. When a vehicle is making a U-turn at a signalized intersection, the following left-turning vehicles must sometimes apply the brakes because of the speed difference. Therefore, vehicles making U-turns will cause the headways of the following left-turning vehicles to be increased; and the increased headway must be considered in the overall capacity reduction because of vehicles making U-turns.

Two important findings were made on the basis of the results of the pilot survey, including: (1) the increased U-turns at signalized intersections adversely affect the capacity of the left-turn lane; and the effects increase with the increase in the percentage of U-turning vehicles in the left-turn lane; and (2) past studies assumed that the discharge flow rate reaches a saturation state after the fourth or the fifth discharged vehicle. The field measurement, however, indicates that if left-turning vehicles are mixed with U-turning vehicles in the left-turn traffic stream, the discharge flow rate does not display an easily identifiable steady maximum rate. Therefore, the traditional headway method, which measures the saturation headway of U-turning vehicles and left-turning vehicles in the field, may not be suitable for estimation of the effects of U-turning vehicles on the left-turn traffic stream.
3.4 Capacity of U-turn Movement at Unsignalized Intersections

U-turn movement is usually provided from an exclusive left-turn lane at an unsignalized intersection. Vehicles making U-turns at an unsignalized intersection are in conflict with the major-street through traffic stream and the right-turning traffic stream from the driveway, as shown in Figure 3-6. Usually, U-turning vehicles should yield to the right-of-way of the major-street through traffic stream. Drivers need to wait for a suitable gap in the major-road traffic stream to perform the U-turn maneuver. In the 2000 edition of the Highway Capacity Manual, a procedure was developed for estimating the capacity and level of service of minor traffic streams at Two-way Stop-controlled (TWSC) intersections. However, the HCM methodology does not contain the procedure for estimating the capacity and level of service of U-turn movement.

Figure 3-6. U-turn Movement at an Unsignalized Intersection
Gap acceptance model has been widely used for estimating the capacity of a minor movement at an unsignalized intersection. Critical gap and follow-up time are two key parameters in a gap acceptance model. One of the objectives of this study is to estimate the capacity of U-turn movement at unsignalized intersections. This objective was achieved by estimating the critical gap and follow-up time for U-turn movement. The following two sections document the current capacity analysis procedures at unsignalized intersections. The methodologies applied in this study to estimating the critical gap and follow-up-time for U-turn movement are also explained.

3.4.1 Capacity Analysis at Unsignalized Intersections

Gap acceptance models have been widely used for estimating the capacity of minor traffic streams at unsignalized intersections. Most of the gap acceptance models are derived from the Siegloch capacity formula. Siegloch proposed a queuing model for estimating the capacity of a minor traffic stream at an unsignalized intersection. The equation of the model is given as (cf. Siegloch, 1973):

\[ c = q_p \int_{t=0}^{\infty} f(t)g(t)dt \]  

(3-4)

where, \( c \) = capacity of a minor traffic stream (veh/hr),

\( q_p \) = major stream traffic volume (veh/hr),

\( f(t) \) = probability density function for the distribution of gaps in the major stream,

\( g(t) \) = the number of vehicles which can enter into the major traffic stream
during a gap of size $t$, and

\[ t = \text{the length of a gap in the major traffic stream (s)}. \]

On the basis of the Siegloch capacity formula, different assumptions are made about the $f(t)$ and $g(t)$, resulting in different capacity models. In the 2000 edition of the Highway Capacity Manual, a procedure was developed for estimating the capacity of minor traffic streams at Two-way Stop-controlled (TWSC) intersections. The potential capacity is considered the basis to determine the capacity and level of service of a minor traffic stream at a TWSC intersection. The potential capacity of a specific minor movement is estimated by a gap acceptance model developed by Harders in 1968 (Harders, 1968). In Harders model, the headway distribution for the major-street traffic stream is assumed to be a negative exponential distribution. The probability density function of the negative exponential distribution is given as follows (cf. Harders, 1968):

\[
f(t) = q_p e^{-q_p t}
\]  

(3-5)

where $q_p$ is the major stream traffic volume (veh/hr); and $t$ is the length of a gap in the major traffic stream (s). In Harders model, it is assumed that the departure function, $g(t)$ is a stepwise constant function. The number of vehicles which can enter into the major traffic stream during a gap of size $t$ is estimated using the following equation (cf. Kyte et al., 1996):

\[
g(t) = \sum_{n=0}^{\infty} n \mu_n(t)
\]  

(3-6)
where \( p_n(t) \) is the probability that \( n \) minor stream vehicles enter into the major traffic stream during a gap of size \( t \). The equation of \( p_n(t) \) is given by (cf. Kyte et al., 1996):

\[
p_n(t) = \begin{cases} 
1 & \text{for } t_c + (n-1) \times t_f \leq t < t_c + n \times t_f \\
0 & \text{elsewhere}
\end{cases}
\] (3-7)

where \( t_c \) is the critical gap for the minor movement (s), and \( t_f \) is the follow-up time for the minor traffic stream.

Harders model is developed by combining Equation 3-4, Equation 3-5 and Equation 3-6. The equation of Harders model is given as follows (cf. Harders, 1968):

\[
c_{p,x} = q_p \frac{e^{-q_p t_{c,x}/3600}}{1 - e^{-q_p t_{f,x}/3600}}
\] (3-8)

where, \( c_{p,x} \) = potential capacity of minor movement \( x \) (veh/hr),
\( q_p \) = flow rate for major traffic stream (veh/hr),
\( t_{c,x} \) = critical gap for minor movement \( x \) (s), and
\( t_{f,x} \) = follow-up time for minor movement \( x \) (s).

What Harders model estimates is the potential capacity of a particular minor traffic stream at an unsignalized intersection. The potential capacity is defined as the capacity of a specific movement under some ideal conditions including:

1. Traffic from nearby intersections does not back into the subject intersection;
2. A separate lane is provided for the exclusive use of each minor-street movement;
(3) An upstream signal does not affect the arrival pattern of the major-street traffic; and
(4) No other minor movements impede the subject movement.

In HCM, several procedures have been developed to account for the prevailing conditions that do not meet the requirements of the potential capacity. Capacity adjustment factors have been developed to account for the impedance effects of other minor movements on the capacity of a particular minor movement. In addition, a procedure is developed to quantify the effects of the upstream signalized intersections. The details are not documented herein.

3.4.2 Critical Gaps and Follow-up Time

Critical gap and follow-up time are two key parameters in most of the current gap acceptance models. In HCM, the critical gap is defined as the minimum time interval in the major-street traffic that allows intersection entry for one minor-street vehicle (TRB, 2000). It is assumed that the critical gap is a constant value for a specific driver. A particular driver will accept any gaps larger than his critical gap and reject any gaps smaller than his critical gap. There are many different methods for estimating the critical gap. The method selected by HCM is the “maximum likelihood method” which was first proposed by Miller and Pretty in 1968 (Miller and Pretty, 1968).

Kyte et al. compared several different ways for the estimation of critical gaps and concluded that the maximum likelihood method and Hewitt’s method gave the best results (Kyte et al., 1996). The maximum likelihood method assumes that a driver’s critical gap is
always smaller than his accepted gap and greater than his largest rejected gap. For a sample of n drivers, the log-likelihood function of n drivers having accepted and largest rejected gaps of \((a_i, r_i)\) is given by (cf. Tian et al., 1999):

\[
L = \sum_{i=1}^{n} \ln[F(\ln(a_i)) - F(\ln(r_i))]
\]  

(3-9)

where \(a_i\) is the gap accepted by driver \(i\), and \(r_i\) is the largest gap rejected by driver \(i\). Miller and Pretty assumed that drivers’ critical gap was log-normally distributed. Therefore, the \(F(x)\) here is the cumulative distribution function for the normal distribution (Miller and Pretty, 1968). In order to find the parameters that maximize the \(L\), the following equations should be solved (cf. Tian et al., 1999):

\[
\frac{\partial L}{\partial \mu} = \sum_{i=1}^{n} \frac{\partial F(\ln(a_i))}{F(\ln(a_i)) - F(\ln(r_i))} - \frac{\partial F(\ln(r_i))}{\partial \mu} = 0
\] 

(3-10)

\[
\frac{\partial L}{\partial \sigma^2} = \sum_{i=1}^{n} \frac{\partial F(\ln(a_i))}{F(\ln(a_i)) - F(\ln(r_i))} - \frac{\partial F(\ln(r_i))}{\partial \sigma^2} = 0
\] 

(3-11)

where \(\mu\) is the mean of the distribution of the logarithms of the individual driver’s critical gaps, and \(\sigma^2\) is the variance of the distribution of the logarithms of the individual driver’s critical gaps. Troutbeck developed a program to solve these two equations iteratively (Troutbeck, 1992). The mean critical gap \(t_c\) and the variance of critical gaps \(s^2\) for a minor movement can then be computed by (cf. Tian et al., 1999):
\[ t_c = e^{u+0.5\sigma^2} \]  
(3-12)

\[ s^2 = t_c^2(e^{\sigma^2} - 1) \]  
(3-13)

In HCM, the followed-up time is defined as the time between the departure of one vehicle from the minor street and the department of the next vehicle using the same major-street. Thus, \( t_j \) is the headway that defines the saturation flow rate for the approach if there are no conflicting vehicles on movements of higher ranks (TRB, 2000). Siegloch’s method can be used for the estimation the follow-up time (Siegloch, 1973). However, this method requires a continuous queuing for the minor traffic stream, which is difficult to be observed in the field. The method used by HCM was described in the NCHRP Report 3-46 (Kyte et al., 1996). The follow-up time was directly observed in the field by measuring the exit-queue time of two vehicles using the same gap. The follow-up time measured by this method is, in fact, the headway between the first discharged vehicle and the second discharged vehicle, which is usually greater than the saturation headway considering the existence of the start-up lost time.

### 3.5 Crash Rate at the Weaving Section

As mentioned previously, the separation distances between driveway exits and downstream U-turn locations could significantly impact the safety performance of vehicles making right-turns followed by U-turns. If the separation distances are too short, vehicles making RTUTs do not have enough space to make comfortable lane changes; this may cause
safety problems at weaving sections. The crashes that may occur at the weaving section while vehicles making right-turns followed by U-turns include:

1. **Angle Crash/Right-Turn Crash**: occurs when drivers accept too small a gap in the major-road through traffic, make a direct entry into the left-turn deceleration lane. Vehicles making RTUT in this condition will place vehicles on the major road with an increased potential of an angle crash;

2. **Sideswipe Crash**: occurs when a vehicle from the outside lane of the major road weaves to the inside lane before stopping at the U-turn location. If the separation distance is not long enough, vehicles do not have enough space to make a comfortable lane change. Some drivers in this condition may change lane in an aggressive way, placing the major-street vehicles with an increased potential of sideswipe collisions; and

3. **Rear-end Crash**: occurs when a right turning vehicle is already on the major road and begins to accelerate. If the separation distance is too short, vehicles do not have enough space to accelerate to the operating speed of through-traffic, thus, the major-street vehicles are encountered with an increased potential of rear-end collisions.

In total, there are 36 different types of crashes in the crash database maintained by FDOT. When conducting crash data analysis, it is very difficult to identify if a particular crash occurred at a selected roadway segment has a RTUT vehicle involved in. Therefore, the analysis of total crashes at the selected roadway segments could provide some biased results. To overcome this problem, we only selected the roadway segments with large
numbers of vehicles making RTUTs for crash data analysis. Thus, vehicles making RTUTs had considerable effects on the safety performance of the selected roadway segments. Besides, at the selected roadway segments, only some particular types of crash data were used for analysis. These crashes include Angle Crash/Right-Turn Crash, Sideswipe Crash and Rear-end Crash. It is assumed that the occurrence of these crashes is affected by vehicles making right-turns followed by U-turns. The occurrence of some other crashes, such as left-turn crashes and head-on crashes are not likely to be affected by vehicles making right-turns followed by U-turns. Consequently, these crashes were withdrawn from crash data analysis.

Crash rate is an important measure of effectiveness for the safety performance of a roadway segment. The purpose of using crash rate as the measure of effectiveness is to reduce the influence of traffic volume on the safety analysis results. Crash rate is usually defined as crash frequency divided by exposure. In this study, the crash rate for a roadway segment is defined as crashes per million vehicle miles traveled (crash/MVM), as shown in the following equation:

\[
\text{Crash Rate} = \frac{1,000,000 \times A}{365 \times T \times V \times L}
\]  

(3-14)

where, 

\( A \) = the number of reported crashes,

\( T \) = the time frame of the analysis (years),

\( V \) = the average ADT volume of the segment at three years time period,

and

\( L \) = the length of the selected roadway segment (miles).
CHAPTER 4
DATA COLLECTION AND REDUCTION

4.1 Introduction

This chapter summarizes the detailed efforts of data collection and data reduction work. Two different types of data were collected, including traffic operations data and crash data. In order to collect traffic operations data, extensive field measurements were conducted at 40 selected sites in the Tampa Bay area of Florida. Crash data were analyzed for 179 selected roadway segments in central Florida. The crash data at selected roadway segments were obtained from the FDOT crash database.

4.2 Traffic Operations Data

During 2002 to 2004, the USF research team spent more than 12 months in the field collecting traffic operations data. Extensive field measurements were carried out at 40 selected roadway segments in central Florida. The selected roadway segments are located on urban or suburban multilane highways. Out of the selected study sites, 24 sites were located on 4-lane divided roadways with 2 lanes in each direction, while 16 sites were located on 6 to 8-lane divided roadways with 3 to 4 lanes in each direction. The selected sites were
classified into 4 groups based on the number of through-traffic lanes and the locations of U-turn bays, as shown in Table 4-1.

Table 4-1. Selected Sites for Field Measurements

<table>
<thead>
<tr>
<th>Number Of Lanes in Both Directions</th>
<th>Location of U-turn Bay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Median Opening</td>
</tr>
<tr>
<td>4-lane</td>
<td>16</td>
</tr>
<tr>
<td>6 to 8-lane</td>
<td>8</td>
</tr>
</tbody>
</table>

The equipments used for field data collection include four video cameras, four VCRs, three batteries, three inverters, three TVs, two scaffolds, one measuring wheel and two Hi-star NC-97 portable traffic analyzers. Some of the equipments used for field data collection are shown in Figure 4-1. Video cameras were set up on scaffolds to achieve adequate viewing height. A picture of the setting of a scaffold is shown in Figure 4-2. The scaffolds were located at least 300 ft away from the subject driveway. Therefore, drivers making a left-turn or right-turn from the subject driveway usually will not realize the existence of the scaffolds. The locations of the scaffolds and cameras in the field are shown in Figure 4-3 and Figure 4-4.

A typical field data collection day started at 7:00 in the morning. Before recording began, the video cameras were synchronized so that the data extracted from different videotapes could be matched. Data collection was usually conducted during weekday 7:00 in the morning to 7:00 in the afternoon. The research team spent at least one week in each site to collect traffic operations data. An average of 30 hours traffic data were recorded in each
site. Traffic operations data were not collected during inclement weather or under unusual traffic conditions on the road.

A total of over 1000 hours of traffic operations data were recorded in the field. The recorded videotapes were later reviewed in the laboratory for obtaining traffic operations data. While reviewing videotapes, the following information were gathered, including traffic volumes, delay and travel time, turning time for U-turns, queue discharge time, gap acceptance data and follow-up time data. Geometric data at each site were directly measured.

Figure 4-1. Some of the Equipments Used for Field Data Collection
in the field. We will explain the detailed procedures for collecting each type of traffic operations data in the following sections.

Figure 4-2. Cameras Setup on a Scaffold

Figure 4-3. Locations of Scaffolds and Cameras in the Field
4.2.1 Delay and Travel Time

Delay and travel time data were measured at 29 selected sites in the Tampa area of Florida. The general criteria for selecting study sites for collecting delay and travel time data include the following:
(1) The selected roadway segment should have a raised-curb median with either a full median opening or a directional median opening that can safely store waiting vehicles;

(2) Speed limit on the major-street should be 40 mph or higher; because the Florida Department of Transportation mandates that all new multilane highways with design speeds of 40 mph or greater be designed with a restrictive median.

(3) The median width should be wide enough to safely store left-turning vehicles;

(4) The subject 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 right-turning vehicles and left-turning vehicles do not interfere with each other; and

(5) The driveway volumes should be high so that there were a considerable number of vehicles making RTUTs and/or DLTs.

Among the selected sites, 13 sites are located on 4-lane divided roadways with 2 lanes in each direction, while 16 sites are located on 6 or more-lane divided roadways with at least 3 lanes in each direction. Operational effects of various driveway left-turn alternatives on 4-lane roadways and on 6 to 8-lane roadways were analyzed separately. The reason for considering the 4-lane and 6 to 8-lane conditions separately lies in the fact that vehicles usually could easily make U-turns on 6 to 8-lane roadways with at least 3 lanes receiving U-turns. On 4-lane roadways, however, vehicles making U-turns may have some geometric concerns such as limited turning path. The selected sites are classified into 4 different
categories based on the number of through traffic lanes and the locations of U-turn bays, as shown in Table 4-2.

Table 4-2. Selected Sites for Delay and Travel Time Analysis

<table>
<thead>
<tr>
<th>Number of Lanes in Both Directions</th>
<th>Location of U-turn Bay</th>
<th>Median Opening</th>
<th>Signalized Intersection</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-lane</td>
<td></td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>6 or more-lane</td>
<td></td>
<td>8</td>
<td>8</td>
</tr>
</tbody>
</table>

Video cameras were setup in the field to record traffic data. The recorded videotapes were later reviewed in the laboratory. From videotapes, each vehicle coming from the driveway making a DLT or a RTUT was tracked. The following information was gathered while reviewing videotapes:

1. Traffic volumes: major-road through-traffic volume, direct left-turn volume, driveway volume and right-turn followed by U-turn volume;
2. Delay: delay for left-turning vehicles and right-turning vehicles at the driveway, delay for left-turning vehicles at the median opening, delay for U-turning at the median opening or signalized intersection;
3. Travel time: the travel time for left-turning vehicles crossing the major-street through lanes, and the travel time drivers spend at weaving sections while making right-turns followed by U-turns; and
4. Turning time: the turning time for a vehicle making a U-turn maneuver at a median opening.
The reduction of field data was based on a fifteen-minute time interval. In each time interval, the average delay and average travel time for vehicles making DLTs or RTUTs were calculated.

4.2.2 Queue Discharge Time

In this study, the effects of U-turn movement on the capacity of an exclusive left-turn lane at a signalized intersection were quantified by analyzing the relationship between the percentage of U-turning vehicles in the left-turn lane and the average queue discharge time for each turning vehicle. Data were collected at three signalized intersections in the Tampa area of Florida. To separate the effects of U-turning vehicles from other factors that may influence intersection capacity, the following criteria were used in the selection of the study sites:

(1) The lane widths were 12ft;
(2) The approach grade was level;
(3) There was no parking adjacent to a travel lane within 250 ft of the stop line;
(4) The intersections were located in a non-central business district area;
(5) The intersections had exclusive left-turn lanes and protected left-turn phasing for left turns;
(6) There was insignificant disturbance from a bus stop;
(7) There was insignificant disturbance from the right-turning vehicles during the left-turn phase in the other approach of the intersection (right-turning vehicles are supposed to yield to U-turning vehicles when U-turns are accommodated at protected
left turn phase; if significant disturbance was observed, the data were excluded from analysis); and

(8) The selected street segment needed to have at least three traffic lanes (including through traffic lanes and exclusive right-turn lane in the other approach) in each direction. Passenger cars can normally make U-turns along a divided six-lane road without any geometric restrictions.

The selected sites are listed in Table 4-3. The traffic flow data and signal timing data were recorded by using two video cameras. Data collection typically started at 4:00 in the afternoon. Data collection was conducted during weekday peak periods. Data were not gathered during inclement weather or under unusual traffic conditions. The following information was gathered by reviewing the videotapes: (a) the number of U-turning vehicles and left-turning vehicles in each queue; and (b) the discharge time required for each queue, which was measured as the time that elapsed from the time that the green signal was initiated until the time that the rear wheel of the last vehicle in the queue crossed the stop line.

**Table 4-3. Selected Sites for Capacity Analysis at Signalized Intersection**

<table>
<thead>
<tr>
<th>Signalized Intersection</th>
<th>N&lt;sub&gt;1&lt;/sub&gt;&lt;sup&gt;a&lt;/sup&gt;</th>
<th>N&lt;sub&gt;2&lt;/sub&gt;&lt;sup&gt;b&lt;/sup&gt;</th>
<th>N&lt;sub&gt;3&lt;/sub&gt;&lt;sup&gt;c&lt;/sup&gt;</th>
<th>Left-turn Phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fowler Ave. &amp; 56&lt;sup&gt;th&lt;/sup&gt; St.</td>
<td>Dual</td>
<td>3</td>
<td>0</td>
<td>P&lt;sup&gt;d&lt;/sup&gt;</td>
</tr>
<tr>
<td>Bruce B. Downs Blvd. &amp; New Tampa Blvd.</td>
<td>Single</td>
<td>2</td>
<td>1</td>
<td>P</td>
</tr>
<tr>
<td>Bruce B. Downs Blvd. &amp; Cross Creek Blvd.</td>
<td>Single</td>
<td>2</td>
<td>1</td>
<td>P</td>
</tr>
</tbody>
</table>

<sup>a</sup> Number of exclusive left-turn lanes;  
<sup>b</sup> Number of through-traffic lanes in each direction;  
<sup>c</sup> Number of exclusive right-turn lanes from other approach of the intersection;  
<sup>d</sup> Protected signal phasing.
The discharge time for each queue was recorded by using a RadioShack LCD Stopwatch, which could record the discharge time with 0.01-s accuracy. To focus on the characteristics of passenger car flows, the data related to heavy vehicles and all vehicles behind a heavy vehicle were excluded from the analysis. Additionally, only those vehicles that had come to a complete stop before the initiation of green signal were included in the analysis. In total, the research team recorded the queue discharge times for 260 queues, including 571 U-turning vehicles and 1,441 left-turning vehicles.

### 4.2.3 Critical Gaps and Follow-up Time

Data were collected at six selected unsignalized intersections for measuring the critical gap and follow-up time for U-turn movement. The following criteria were considered in the site selection process:

1. The selected street should be a 4-lane divided street with 2 lanes in each direction;
2. There should be a left-turn storage lane at the median opening. Vehicles can stop and queue at the left-turn lane and then wait for a suitable gap in the major-street traffic stream to perform the U-turn maneuver; and
3. The U-turn volume at the selected site should not be too small in order to record as many observations as possible during a certain period of time.

The selected sites are shown in Table 4-4. The width of median nose at selected sites varies from 3 ft to 45 ft, as shown in Table 1. At site 1, site 2, and site 3, there are more than 70% of passenger cars need to encroach on to the shoulder to perform the U-turn maneuver.
At site 4, site 5, and site 6, there are more than 65% of passenger cars make U-turns from the left-turn storage lane into the lane next to the shoulder. Less than 15% of passenger cars at those sites need to encroach on to the shoulder to perform the U-turn maneuver. The selected sites were then divided into two groups, including the “narrow median group” (site 1, site 2, and site 3) and the “wide median group” (Site 4, site 5, and site 6). The critical gap and follow-up time for U-turn movement under these two different conditions were analyzed separately.

A video camera was used to collect traffic data in the field. The recorded video tapes were reviewed in the laboratory for obtaining gap acceptance and follow-up time data. A computer program was written to measure the available gaps on the major-street and the response of U-turning vehicles to these gaps. The first rejected/accepted gap was measured from the time when a U-turning vehicle arrives at the median opening until the time when a major-street vehicle arrives at the median opening. This time interval can also be defined as a “lag”. A “gap” was measured as the time interval between two consecutive major-street vehicles passing a reference line in the major-street. Only when there was insignificant disturbance from other minor movements, the gap acceptance data for U-turns were collected. The largest rejected gap and the accepted gap for each U-turning vehicle were saved into a database. If one individual driver did not reject any gaps and accepted the first available gap in the major-street traffic stream, this data was omitted. In total, the research team recorded the rejected and accepted gaps for 387 U-turning vehicles. Some other data were also gathered while reviewing video tapes, including the major-street through traffic volume in the direction that is in conflict with U-turning vehicles, the service delay for each
U-turning vehicle at the median opening, and the queue discharge time of several consecutive U-turning vehicles using a same gap.

Table 4-4. Selected Sites for Critical Gaps and Follow-up Time

<table>
<thead>
<tr>
<th>Site</th>
<th>Street</th>
<th>City</th>
<th>MW (ft)</th>
<th>Distribution of U-turns (%)</th>
<th>L^d</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Thonotosassa Rd.</td>
<td>Plant City</td>
<td>3</td>
<td>1.2 25.6 73.2</td>
<td>2140 ft</td>
</tr>
<tr>
<td>2</td>
<td>Thonotosassa Rd.</td>
<td>Plant City</td>
<td>5</td>
<td>0   28.1 71.9</td>
<td>&gt;1 mile</td>
</tr>
<tr>
<td>3</td>
<td>US 301</td>
<td>Brandon</td>
<td>8</td>
<td>1.4 25.1 73.5</td>
<td>729 ft</td>
</tr>
<tr>
<td>4</td>
<td>US 301</td>
<td>Brandon</td>
<td>25</td>
<td>10.2 77.8 12</td>
<td>1480 ft</td>
</tr>
<tr>
<td>5</td>
<td>Gunn Hwy</td>
<td>Tampa</td>
<td>45</td>
<td>19.3 79.4 1.3</td>
<td>2650 ft</td>
</tr>
<tr>
<td>6</td>
<td>Gunn Hwy</td>
<td>Tampa</td>
<td>21</td>
<td>20.1 68.2 11.7</td>
<td>1117 ft</td>
</tr>
</tbody>
</table>

^a Vehicles making U-turns into the lane next to the median

^b Vehicles making U-turns into the lane next to the outside shoulder

^c Vehicles encroach on to the shoulder to perform the U-turn maneuver

^d The distance from U-turn location to upstream signal

4.3 Crash Data

In this study, crash history at 179 roadway segments was investigated. The roadway segment was defined as an urban or suburban arterial segment with non-traversable medians. The roadway segments begin at a driveway/side street and continue downstream toward a median opening or a signalized intersection which accommodates U-turns. Figure 4-5 presents the definition of a roadway segment for crash data analysis.

The driveway/side streets selected are those active access points that have high ingress and egress volumes. To avoid interference between driveways, conditions of one U-turn bay shared by several major driveways along the arterial were not studied. The selected
sites can be divided into four groups based on the number of through traffic lanes and the locations of U-turn bays, as shown in Table 4-5. All of the selected sites were considered to have similar operational or design characteristics, including:

(1) The selected sites are located on multilane arterials with non-traversable medians;
(2) At the selected sites, there are no other major driveways between the subject driveway and the downstream U-turn location;
(3) Speed limit on the major-street is equal to or higher than 45 mph, because FDOT mandates that all new or reconstructed multi-lane arterials with design speeds over 40 mph be designed with restrictive medians; and
(4) The driveway volumes at selected sites are high and direct left-turn access at subject driveway is not permitted. Therefore, there are a considerable number of vehicles making RTUTs at selected sites.

Figure 4-5. Definition of a Roadway Segment for Crash Data Analysis
Table 4-5. Selected Sites for Crash Data Analysis

<table>
<thead>
<tr>
<th>Number of Lanes in Both Directions</th>
<th>Location of U-turn Bay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Median Opening</td>
</tr>
<tr>
<td>4-lane</td>
<td>35</td>
</tr>
<tr>
<td>6 or more-lane</td>
<td>79</td>
</tr>
</tbody>
</table>

Most of the traffic and geometric information such as the post speed limit, signal installation and separation distance were determined from field observations. The separation distance at the selected sites varies from 73 ft to 1150 ft with an average of 429 ft. The aerial photographs of two sample sites were given in Figures 4-6 and 4-7. Figure 4-6 depicts a site where the U-turn is provided at a directional median opening. The minor street, Beckett Way, serves a big residential area. The separation distance at this site was found to be 377 ft, as shown in Figure 4-6. There are two minor driveways between the subject driveway and the U-turn location. However, the traffic volume at these two minor driveways was found to be very low. Figure 4-7 depicts a site where U-turns are provided from the exclusive left-turn lane at a signalized intersection. The subject driveway is a major driveway which serves a large business plaza. This kind of driveway usually has larger driveway volume. The separation distance at this selected site was found to be 275 ft.

The crash data at selected roadway segments were obtained from the FDOT crash database. FDOT maintains a very large crash database generated by merging crash data from the Department of Highway Safety and Motor Vehicles with roadway information from FDOT. This database is updated yearly. All police reported crashes with a fatality, an injury, and high property damage occurred on state roads are included in this database.
A SAS (Statistical Analysis System) program was written to extract crash data from the FDOT crash database. It is commonly believed that three years would usually provide a sufficient number of crashes for analysis while reducing the possibility of extraneous factors influencing the crash data. In this study, crash data of three consecutive years, from 2001 through 2003, were used for the analysis process.

Figure 4-6. Aerial Photo of a Selected Site: U-turns at a Median Opening
Figure 4-7. Aerial Photo of a Selected Site: U-turns at a Signalized Intersection
CHAPTER 5
DELAY AND TRAVEL TIME COMPARISON

5.1 Introduction

This chapter presents the results for delay and travel time analysis. Delay and travel time is compared for three different driveway left-turn alternatives that are widely implemented in Florida and nationally. These driveway left-turn alternatives include the following: (1) direct left-turns at a driveway; (2) right-turns followed by U-turns at a downstream signalized intersection; and (3) right-turns followed by U-turns at a downstream median opening in advance of a signalized intersection. A travel time model is developed to predict the travel time drivers spent at different weaving sections while making RTUTs. With the delay comparison results and the travel time model, the total travel time can be compared for various driveway left-turn alternatives under different levels of conflicting traffic volumes and different roadway geometric conditions.

5.2 Delay Comparison

Delay is an important parameter used by transportation professionals as the criteria to evaluate the level of service for signalized intersections and unsignalized intersections. One
of the objectives of this study is to compare delay for various driveway left-turn alternatives under different levels of driveway volumes and major-road through-traffic volumes. The data used for delay comparison was measured in the field. Vehicle delay at a driveway was measured from the time when a car stopped at the waiting queue until the time it crossed the stop line. Delay for a left-turning vehicle at a median opening was measured from the time when a car stopped at the median opening until the time it left the median opening. Delay for a U-turning vehicle at a signalized intersection or an unsignalized median opening was measured from the time when a car stopped at a U-turn bay until the time it started making a U-turn. Based on these definitions, delay in this study consists of waiting delay and queuing delay. The deceleration time and acceleration time were considered as parts of the vehicle travel time, and therefore, were not considered in the delay comparison.

Descriptive statistics for delay data were shown in Table 5-1. The collected average vehicle delay varies from 20 s per vehicle to 79 s per vehicle. The following roadway conditions were considered when comparing delay for different driveway left-turn alternatives: (1) the major-street is a 4-lane divided roadway with 2 lanes in each direction; and (2) the major-street is a 6-lane or an 8-lane divided roadway with at least 3 lanes in each direction. Delay for different driveway left-turns alternatives on 4-lane roadways and on 6 to 8-lane roadways were analyzed separately. The reason is that vehicles making direct left-turns on 6 to 8-lane roadways need to cross at least 3 lanes before they stop at the median openings. On 4-lane divided roadways, vehicles making direct left-turns only need to cross 2 lanes at a time. Thus, it is easier to make direct left-turns on 4-lane divided roadways than on 6 to 8-lane roadways.
The average vehicle delay was compared for various driveway left-turn alternatives under different volume conditions. The results are shown in Table 5-2. The collected data were divided into 6 different volume scenarios based on the levels of the driveway volume and the major-road through-traffic volume. In each scenario, the average delay for vehicles making DLTs or RTUTs was calculated. A total of 2997 observations were used to compare vehicle. Each observation herein represents the vehicle delay data collected in the field within a 15-min time interval.

As shown in Table 5-2, when the volume levels in the major-street and the driveway are low, vehicles making RTUTs at a median opening result in 1 to 3 s less delay as compared with those making DLTs at a driveway. With the increase of the driveway volume and the major-road through-traffic volume, delay for direct left-turn movement increases rapidly. When the volume levels in the major-street and driveway are high, vehicles making DLTs at a driveway could result in up to 24 s more delay as compared with those making RTUTs at a median opening.
Table 5-2. Delay Comparison for Various Driveway Left-turn Alternatives

<table>
<thead>
<tr>
<th># of Through Lanes</th>
<th>Traffic Volume (veh/hr)</th>
<th>Average Delay (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Through Volume</td>
<td></td>
</tr>
<tr>
<td>4-lane Condition</td>
<td>Left-turn/U-turn Volume</td>
<td>Signal U-turn</td>
</tr>
<tr>
<td>0 - 50</td>
<td>1000 - 2000</td>
<td>77</td>
</tr>
<tr>
<td></td>
<td>2000 - 3000</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td>3000 - 4000</td>
<td>83</td>
</tr>
<tr>
<td>&gt;= 50</td>
<td>1000 - 2000</td>
<td>76</td>
</tr>
<tr>
<td></td>
<td>2000 - 3000</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td>3000 - 4000</td>
<td>83</td>
</tr>
<tr>
<td>6 to 8-lane Condition</td>
<td>0 - 50</td>
<td>2000 - 3000</td>
</tr>
<tr>
<td></td>
<td>3000 - 4000</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td>4000 - 5000</td>
<td>92</td>
</tr>
<tr>
<td>50-100</td>
<td>2000 - 3000</td>
<td>79</td>
</tr>
<tr>
<td></td>
<td>3000 - 4000</td>
<td>97</td>
</tr>
<tr>
<td></td>
<td>4000 - 5000</td>
<td>103</td>
</tr>
<tr>
<td>&gt;=100</td>
<td>2000 - 3000</td>
<td>N/A(^a)</td>
</tr>
<tr>
<td></td>
<td>3000 - 4000</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>4000 - 5000</td>
<td>N/A</td>
</tr>
</tbody>
</table>

\(^a\)No data points in the particular category.

Vehicles making U-turns at a signalized intersection need to wait until the signal turns green to perform the U-turn maneuver. Delay for vehicles making RTUTs at a signalized intersection depends not only on the conflicting volumes, but also on some other parameters such as the signal timing and left-turn traffic demand at the signalized intersection as well. Vehicles making U-turns at a signalized intersection with a large traffic demand could experience a long delay. As shown in Table 5-2, vehicles making RTUTs at a signalized intersection result in 44 to 59 s more delay than with those making DLTs at a
driveway, and around 49 to 64 s more delay as compared with those making RTUTs at a median opening.

5.3 Travel Time Comparison

5.3.1 Travel Time Drivers Spend at Weaving Sections While Making RTUTs

The travel time drivers spent at weaving sections while making RTUTs consists of the following two parts: (1) the elapsed time from the time when a vehicle leaves the driveway until the time when it stops at the exclusive left-turn bay of downstream U-turn location; and (2) the elapsed time from the time when a vehicle starts making a U-turn until the time when it finishes traversing the separation distance from U-turn bay to subject driveway at the speed of major-street through-traffic.

In this study, a multiple linear regression model was developed to predict the travel time drivers spent at different weaving sections while making RTUTs. The stepwise regression method was applied to determine the independent variables that should be included into the regression model. A pre-selected $F_{\text{OUT}}$ critical value of 0.1 was selected as the criteria for selecting independent variables. The selected independent variables include the number of lanes of the major-street, the traffic control type at U-turn locations, the major-street speed limit, and the separation distance between a driveway and the downstream U-turn location.

Data collected from 29 sites were used to build the travel time model. Among the selected sites, there are 13 sites located on 4-lane divided roadways with 2 lanes in each
direction, while 16 sites are located on 6 to 8-lane divided roadways with at least 3 lanes in each direction. At 14 selected sites, U-turns are provided at a median opening in advance of a signalized intersection. At 15 selected sites, U-turns are provided from an exclusive left-turn lane at a signalized intersection. Two dummy variables are defined in this study to distinguish these different situations. The dummy variable “Lanes” was defined to distinguish between the sites on 4-lane roadways and the sites on 6 to 8-lane roadways. The dummy variable “Location” was defined to distinguish between the situation where U-turns are provided at a median opening and the situation where U-turns are provided at a signalized intersection.

The dependent variable of the model is the average travel time drivers spend at the weaving section at a particular site while making RTUTs. The range of the average travel time at the selected sites is from 14 s to 41 s. The descriptive statistics for dependent variables and independent variables are given in Table 5-3. The final regression results are given in Table 5-4.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>N</th>
<th>Min</th>
<th>Max</th>
<th>Mean</th>
<th>Std. Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Travel Time (s)</td>
<td>29</td>
<td>13</td>
<td>41</td>
<td>28.02</td>
<td>7.11</td>
</tr>
<tr>
<td>L (ft)</td>
<td>29</td>
<td>285</td>
<td>1150</td>
<td>608.97</td>
<td>216.66</td>
</tr>
<tr>
<td>Speed (mph)</td>
<td>29</td>
<td>40</td>
<td>55</td>
<td>46.55</td>
<td>3.56</td>
</tr>
</tbody>
</table>

The developed travel time model has fairly high $R^2$ value (0.912) and adjusted $R^2$ value (0.901). The t-statistics show that the selected explanatory variables are all statistically
significant at a 95% level of confidence. In Figure 5-1, the unstandardized residuals were plotted against the fitted travel time value. It was found that the residuals were randomly distributed around the $y=0$ axis, indicating the fact that the model was correctly specified and the homogeneous assumption about the error term was not violated.

Table 5-4. Regression Results for Travel Time Model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Coefficients</th>
<th>Std. Error</th>
<th>t</th>
<th>Sig.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>22.01</td>
<td>6.953</td>
<td>3.170</td>
<td>0.0042</td>
</tr>
<tr>
<td>L</td>
<td>0.032</td>
<td>0.002</td>
<td>16.330</td>
<td>0.0000</td>
</tr>
<tr>
<td>Speed</td>
<td>-0.296</td>
<td>0.133</td>
<td>-2.220</td>
<td>0.0363</td>
</tr>
<tr>
<td>Lanes</td>
<td>-3.701</td>
<td>0.861</td>
<td>-4.300</td>
<td>0.0002</td>
</tr>
<tr>
<td>Location</td>
<td>2.838</td>
<td>0.901</td>
<td>3.150</td>
<td>0.0043</td>
</tr>
</tbody>
</table>

$R^2 = 0.927$, $R^2_{adj} = 0.914$

The correlation matrix and the variance inflation factors (VIF) were used to evaluate the extent of the multicollinearity problem between selected independent variables. The correlation matrix and the variance inflation factors are given in Table 5-5. The variance inflation factor for each independent variable varies from 1.205 to 1.458. From Table 5-5, it is clear that there is little or no collinearity problem in the proposed travel time model. The equation of the travel time model was given as follows:

$$T = 22.01 + 0.032L - 3.701Lanes + 2.838Location - 0.296Speed \quad (5-1)$$

where, $T =$ Travel time drivers spent at weaving sections while making RTUTs (s),

$L =$ the separation distance between driveways and U-turn locations (ft),

$Lanes =$ Dummy variable ($=1$ on 4-lane roadways; $=0$ on 6 or more-lane roadways),
Location = Dummy variable (= 1 if U-turns are provided at signalized intersections, 
= 0 if U-turns are provided at median openings), and

Speed = major-street speed limit (mph).

Figure 5-1. Plot of Unstandardized Residuals versus the Fitted Travel Time Data

From Equation 5-1, it is clear that the separation distance between a driveway and the 
downstream U-turn location significantly impacts the travel time for vehicles making 
RTUTs, and the travel time increases with the separation distance and decreases with the 
major-street speed limit. The coefficient of the dummy variable “Lanes” is -3.701, implying 
that vehicles making RTUTs on 6 or 8-lane streets may spend around 4 s additional travel 
time at the weaving sections as compared with those on 4-lane streets. The coefficient of the 
dummy variable “Location” is 2.838, implying that vehicles making RTUTs at signalized
intersections will have around 3 s more travel time at the weaving sections than those making U-turns at median openings.

Table 5-5. Collinearity Diagnostics for Travel Time Model

<table>
<thead>
<tr>
<th>Independent Variables</th>
<th>VIF</th>
</tr>
</thead>
<tbody>
<tr>
<td>L</td>
<td>1.197</td>
</tr>
<tr>
<td>Lanes</td>
<td>1.205</td>
</tr>
<tr>
<td>Location</td>
<td>1.345</td>
</tr>
<tr>
<td>Speed</td>
<td>1.458</td>
</tr>
</tbody>
</table>

\[
X^* X^* = \begin{bmatrix}
1.000 & 0.223 & 0.333 & 0.444 \\
0.223 & 1.000 & -0.158 & 0.332 \\
0.333 & -0.158 & 1.000 & 0.049 \\
0.444 & 0.332 & 0.049 & 1.000
\end{bmatrix}
\]

5.3.2 Travel Time Comparison

As mentioned in chapter 3, the travel time for a complete direct left-turn maneuver is equal to the total delay plus the running time from when a vehicle leaves the driveway until it stops at the median opening. Based on the field observation, a vehicle making a DLT requires around 3 s of travel time to cross 2 lanes in the major-street on 4-lane divided roadways; and around 5 s of travel time to cross 3 to 4 lanes in the major-street on 6 to 8-lane divided roadways.

The total travel time for a complete right-turn followed by a U-turn procedure is equal to the delay to right-turning vehicles at the driveway and the delay to U-turning vehicles at the U-turn bay plus the travel time drivers spend at the weaving section. The
travel time drivers spend at the weaving section while making RTUTs can be estimated using the travel time model developed in the previous section.

In practice, when a full median opening is closed or replaced with a directional median opening, drivers often oppose being directed to make a right-turn followed by a U-turn due to the perception that it will result in much longer travel time as compared with the direct left-turn movement at the driveway. The delay comparison results and the travel time model provide a tool to compare the total travel time for different driveway left-turn alternatives under different levels of conflicting traffic volumes and different roadway geometric conditions. For example, assuming that the major-road through-traffic volume is 4500 veh/hr, the driveway left-turn traffic demand is 75 veh/hr, and the major-street speed limit is 45 mph, the total travel time for different driveway left-turn alternatives on a 6-lane divided roadway are shown in Figure 5-2.

The curves in Figure 5-2 show that vehicles making RTUTs at a median opening in advance of a signalized intersection result in comparable total travel time as compared with those making DLTs at a driveway. Vehicles making RTUTs at a median opening do have relatively longer travel time when the separation distance between the driveway and the downstream median opening is great. As shown in Figure 5-2, vehicles making RTUTs at a median opening could have up to 28 s longer travel time than those making DLTs when the separation distance is 1200 ft. However, a separation distance of 1200 ft is in fact very difficult to be found in the real world. The research team has measured the separation distance at 179 roadway segments in central Florida. Out of the 179 roadway segments, the largest separation distance is found to be 1150 ft, and more than 85% of the sites have a
separation distance between 150 ft and 750 ft. Within this distance range, vehicles making right-turns followed by U-turns at a downstream median opening are not found to result in much longer travel time than those making direct left-turns at a driveway.

Figure 5-2. Travel Time Comparison for Different Driveway Left-turn Alternatives

As shown in Figure 5-2, vehicles making RTUTs at a downstream signalized intersection could have up to 93 s additional total travel time as compared with those making direct left-turns at a driveway. If the separation distance is between 200 ft and 800 ft, vehicles making RTUTs at a downstream signalized intersection could result in 61 to 80 s more travel time than those making direct left-turns at a driveway. Previous studies have demonstrated that using right-turns followed by U-turns at a signalized intersection as an alternative to direct left-turns at a driveway could improve the safety performance of the
roadway (Lu et al., 2004, Potts et al., 2004, and Carter et al., 2005). Considering this fact, the treatment is not un-acceptable when the left-turn traffic demand at the driveway is not so high. However, if the left-turn traffic demand at a driveway is relatively high, for example, greater than 150 veh/hr, closing the existing median opening or replacing the full median opening with a directional median opening will relocate the left-turning vehicles to downstream signalized intersections to make U-turns. Under this condition, vehicle delay at the signalized intersection could constitute an operational concern for vehicles at the driveway. In addition, the increased number of U-turning vehicles at a signalized intersection may also have some adverse impacts on the capacity of the signalized intersection, as demonstrated by several previous studies (Carter et al. 2005, Adams and Hummer 1993, Tsao and Chu 1996, Liu et al. 2005). Therefore, when the left-turn demand at driveway is high, consideration should be given to providing enough U-turn opportunities in advance of the downstream signalized intersection.

5.4 Summary

This chapter presents the results for delay and travel time data analysis. The following findings are made on the basis of the delay and travel time analysis:

(1) Vehicles making right-turns followed by U-turns at a downstream median opening before a signalized intersection have less delay as compared with those making direct left-turns at a driveway. However, when U-turns are provided at a downstream
signalized intersection, vehicles making right-turns followed by U-turns result in longer delay than those making direct left-turns at a driveway.

(2) The separation distance between a driveway and the downstream U-turn location significantly impacts the travel time drivers spend at the weaving section while making right-turns followed by U-turns. the travel time increases with the separation distance and decreases with the major-street speed limit.

(3) Vehicles making right-turns followed by U-turns at a downstream median opening before a signalized intersection have comparable total travel time as compared with those making direct left-turns at a driveway.

(4) When the left-turn traffic demand at a driveway is high, consideration should be given to providing enough U-turn opportunities in advance of the downstream signalized intersection.

The findings obtained from the delay and travel time analysis can help traffic engineers and designers make decisions about the design and selection of median treatments and various driveway left-turn alternatives on multilane highways. The delay and travel time comparison results show that the access management technique - providing right-turns followed by U-turns at a downstream median opening as an alternative to direct left-turns at a driveway does no result in longer delay and travel time. This conclusion is particular helpful in addressing the public concerns with regard to the delay and travel time for indirect left-turn treatments.
It is important to note that, delay and travel time analysis in this study was conducted based on the data collected from urban or suburban multilane highways, where traffic signals could significantly impact the headway distribution in the major-street through-traffic. The delay and travel time comparison results cannot be directly applied to a rural traffic environment, where the major-street through-traffic is generally in a free flow state.
CHAPTER 6
SELECTION OF U-TURN LOCATIONS

6.1 Introduction

One of the objectives of this study is to evaluate how the separation distances between driveway exits and downstream U-turn locations impact the safety and operational performance of vehicles making right-turns followed by U-turns. With such results, the optimal location of a U-turn bay can be determined so that drivers have better access to make right-turns followed by U-turns. To achieve this research objective, crash data was investigated at 179 roadway segments. Four different types of crashes are selected for crash data analysis. These crashes include angle crash, right-turn crash, sideswipe crash and rear-end crash. This chapter presents the crash data analysis results. On the basis of the crash data analysis results, recommendations are given about the minimum and optimal separation distances under different roadway conditions to facilitate driver use of RTUT.

6.2 Crash Rate at Weaving Sections

The separation distance between a driveway and the downstream U-turn bay is an important consideration for a driver deciding whether to make a RTUT or a direct left-turn. If the separation distance is too short, vehicles making RTUTs do not have enough space to
make comfortable lane changes; this situation may cause safety problems at the weaving section. On the other hand, a separation distance that is too great may result in a longer travel time and, thus, discourage drivers from making RTUTs. Currently, there are no regulations or guidelines for determining the minimum and optimal separation distance to facilitate driver use of RTUT.

In this study, crash data is investigated at 179 selected roadway segments. Out of the 179 sites investigated, 39 sites do not have any crashes occurred during 3 years time period. The crash frequency at selected roadway segments varies from 0 to 18 with an average of 2.9 within 3 years. A total of 557 crashes were used for crash data analysis. Out of these crashes, about 49% crashes are rear-end crashes; about 29% crashes are angle crashes (including right turn crashes); and about 22% crashes are sideswipe crashes.

Crash rate was calculated at each selected roadway segment. The definition of crash rate is explained in chapter 3. The crash rate at the selected roadway segments varies from 0 to 2.27 crashes/MVM with an average of 0.38 crashes/MVM. The observed crash rate data were fitted to an exponential distribution. The parameters of the exponential distribution were estimated using linear regression method. Base on the regression results, the distribution fitting equation for the exponential distribution is given as follows:

\[ f(x) = \lambda e^{-\lambda(x-\beta)} \]  \hspace{2cm} (6-1)

where, \( \lambda = 2.923 \)
\( \beta = 0.05 \)
Figure 6-1 presents the frequency distribution of crash rates at selected roadway segments and the curve for the fitted exponential distribution. The fitted curve is in good fit to the observed data in terms of the high $R^2$ value (.99). The Chi-square test and K-S test were performed to test the hypothesis that the crash rates are exponentially distributed. The results show that there is no evidence that the hypothesis about the exponential distribution can be rejected.

With the fitted exponential distribution, the percentile values for crash rates can be determined. As shown in Figure 6-2, the 50th and 85th percentile values of crash rates are .287 and .700 crashes/MVM respectively. The 50th percentile is the median value of the distribution, and the 85th percentile value represents the point where 85% of all the selected
roadway segments have crash rates no larger than this point’s X-coordinate value. These two percentiles are the most commonly used threshold values in engineering analysis.

![Diagram showing cumulative percentage and fitted line for crash rates.]

Figure 6-2. The 50th and 85th Percentile Values of Crash Rates

6.3 Crash Rate Model

In this study, the crash rates on 4-lane divided roadways and on 6 to 8-lane divided roadways were analyzed separately. The reason lies in the fact that on 4-lane streets, vehicles making RTUTs need to make one lane change before they stop at the U-turn bay; on 6 to 8-lane streets, however, vehicles making RTUTs need to make at least two lane changes before they can stop at the U-turn bay. Thus, theoretically, vehicles making RTUTs on 6 to 8-lane
divided roadways are more likely to be involved in an accident as compared with the situation on 4-lane divided roadways.

The linear regression method was used to identify the factors that significantly impact the crash rate at the selected roadway segments. The dependent variable of the model is the crash rate at a selected roadway segment. A dummy variable “Location” was defined to distinguish between the situation where the U-turn bay is located at a median opening and the situation where U-turn bay is located at a signalized intersection. The candidate independent variables include the separation distance, the location of the U-turn bay, the through-traffic volume and the major street speed limit.

The stepwise regression method was applied to determine which variables will be incorporated into the crash rate model. It was found that the logarithm of the separation distance and the location of the U-turn bay significantly impact the crash rate at selected roadway segments. The major-street through-traffic volume and the major-street speed limit were not found to be significant at a 90% confidence level; and therefore, were not included into the crash rate model.

Descriptive statistics for the dependent variable and the independent variables are shown in Table 6-1. The range of the separation distance at selected roadway segments is from 73 ft to 1150 ft. The frequency distribution and cumulative curve for the separation distances at selected roadway segments are shown in Figure 6-3. It is found that more than 85% of separation distances are between 150 ft and 750 ft.
Table 6-1. Descriptive Statistics for Collected Data

<table>
<thead>
<tr>
<th>Parameters</th>
<th>N</th>
<th>Min.</th>
<th>Max.</th>
<th>Mean</th>
<th>Std. Deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crash Rate</td>
<td>179</td>
<td>0</td>
<td>2.27</td>
<td>0.379</td>
<td>0.370</td>
</tr>
<tr>
<td>Separation Distance (ft)</td>
<td>179</td>
<td>73</td>
<td>1150</td>
<td>429.156</td>
<td>202.920</td>
</tr>
<tr>
<td>Location</td>
<td>179</td>
<td>0</td>
<td>1</td>
<td>0.363</td>
<td>0.482</td>
</tr>
</tbody>
</table>

Figure 6-3. Frequency Distribution and Cumulative Curve for Separation Distances

The regression results are presented in Table 6-2 and Table 6-3. The $R^2$ values for these two crash rate models are .34 and .30 respectively. The crash rate models for 4-lane divided roadways and for 6 to 8-lane divided roadways are given in Equation 6-2 and Equation 6-3, respectively.
\[ CR_1 = 2.235 + 0.133 \text{Location} - 0.334 \ln(L) \]  
\[ CR_2 = 2.516 + 0.192 \text{Location} - 0.364 \ln(L) \]

where, \( CR_1 \) = Crash rate at weaving sections on 4-lane divided roadways (crashes/MVM);

\( CR_2 \) = Crash rate at weaving sections on 6 to 8-lane divided roadways (crashes/MVM);

\( Location \) = Dummy variable (= 1 if U-turn bays are located at signalized intersections, = 0 if U-turn bays are located at median openings); and

\( \ln(L) \) = the logarithm of the separation distance between a driveway and a U-turn bay (ft).

Table 6-2. Regression Results of Crash Rate Model (4-lane Roadways)

<table>
<thead>
<tr>
<th>Independent Variables</th>
<th>Coefficient</th>
<th>t</th>
<th>Sig.</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \ln(L) )</td>
<td>-0.334</td>
<td>-4.722</td>
<td>0.000</td>
</tr>
<tr>
<td>Location</td>
<td>-0.133</td>
<td>1.874</td>
<td>0.066</td>
</tr>
<tr>
<td>Intercept</td>
<td>2.235</td>
<td>5.389</td>
<td>0.000</td>
</tr>
</tbody>
</table>

\( R^2 = 0.34, R_{adj}^2 = 0.31 \)

Table 6-3. Regression Results of Crash Rate Model (6 to 8-lane Roadways)

<table>
<thead>
<tr>
<th>Independent Variables</th>
<th>Coefficient</th>
<th>t</th>
<th>Sig.</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \ln(L) )</td>
<td>-0.364</td>
<td>-5.761</td>
<td>0.000</td>
</tr>
<tr>
<td>Location</td>
<td>0.192</td>
<td>2.955</td>
<td>0.004</td>
</tr>
<tr>
<td>Intercept</td>
<td>2.516</td>
<td>6.514</td>
<td>0.000</td>
</tr>
</tbody>
</table>

\( R^2 = 0.30, R_{adj}^2 = 0.28 \)
Based on the T-statistics, the selected independent variables are all statistically significant at a 90% confidence level. From the crash rate models, it is clear that the separation distance between a driveway exit and the downstream U-turn bay significantly impacts the crash rate at the weaving section, and the crash rate decreases with the increases of the separation distance. The coefficient for the dummy variable “Location” is positive, implying the fact that vehicles making right-turns followed by U-turns at a signalized intersection will have more chance to be involved in a crash at the weaving section as compared with those making U-turns at a median opening. This is in part due to the more complex driving situation close to the signalized intersection. The residuals of two crash rates model were plotted against the fitted crash rate data in Figure 6-4 and Figure 6-5, respectively. It was found that the residuals were randomly distributed around the \(y=0\) axis, indicating the fact that the model was correctly specified and the homogeneous assumption about the error term was not violated.

### 6.4 Determination of the Minimum Separation Distance

In this study, the 50\textsuperscript{th} percentile value of crash rate was used as the threshold to determine the critical value of separation distance. The 50\textsuperscript{th} percentile value of crash rate was found to be .287 crashes per million vehicles per mile. The critical separation distance for vehicles making RTUTs under different roadway conditions were then determined by applying the 50\textsuperscript{th} percentile value of crash rate into the regression models developed in the previous section. The thinking behind this methodology is that the roadway segment with a separation distance less than the critical value will, theoretically, have a crash rate greater
than the median level. The procedures to obtain the critical values of separation distance under different roadway conditions were presented in Figures 6-6 and Figure 6-7.

![Figure 6-4. Plot of Unstandardized Residuals versus the Fitted Crash Rates on 4-lane Divided Roadways](image)

The critical separation distances under different roadway conditions are given in Table 6-4. Based on the critical separation distances, recommendations were given for the minimum separation distances under different roadway conditions. On 4-lane divided roadways with 2 lanes in each direction, if U-turn bay is located at a median opening, the minimum separation distance between the driveway exit and the downstream median opening is found to be 350 ft. If U-turn bay is located at a signalized intersection, the minimum separation distance is found to be 500 ft. On 6 or 8-lane divided roadways, if U-
turn bay is located at a median opening, the minimum separation distance between the driveway exit and the downstream median opening is found to be 450 ft. If U-turn bay is located at a signalized intersection, the minimum separation distance is found to be 750 ft.

Figure 6-5. Plot of Unstandardized Residuals versus the Fitted Crash Rates on 6 to 8-lane Divided Roadways

6.5 Travel Time at Different Weaving Sections

As mentioned previously, the separation distance between a driveway and the downstream U-turn bay could significantly impacts the travel time drivers spend at the weaving section while making right-turns followed by U-turns. If the separation distance is too great, drivers may not want to make a right-turn followed by a U-turn because of the
increased travel time and gap consumption. In chapter 5, a travel time model was developed to predict the travel time drivers spent at different weaving sections while making RTUTs. Assuming that the major-street speed limit is 50 mph, the expected travel time a driver spends at the weaving section with recommended minimum separation distance under different roadway conditions are shown in Table 6-5.

As shown in Table 6-5, vehicles making RTUTs could have up to 34 s travel time at the weaving section if the U-turn bay is located at 750 ft away from the driveway. An additional travel time of 34 s, sometimes, does discourage some drivers from making RTUTs. In fact, when making RTUTs, drivers are often in face of a tradeoff between the...
increased travel time and a safer driving environment. Vehicles making direct left-turns at a driveway may have less travel time as compared with those making RTUTs at a signalized intersection. However, they are also more likely to be involved in a collision with the major-street vehicles, as demonstrated by some previous studies (Kach, 1992; Levinson et al., 2000; Maki, 1996; Cluck et al., 1999; Lu et al., 2001; Potts et al., 2004; Carter et al., 2005). It is true that vehicle delay and travel time are important Measures of Effectiveness of traffic operations, however, they should not be considered as the major criteria for urban traffic design. When making design decisions on urban or suburban multilane arterials, safety should always be considered with the highest priority. Due to this reason, it was recommended by the author that the travel time drivers spent at weaving sections while making RTUTs should not be considered the major criteria for determining the separation distance.

6.6 Summary

The following findings are made based on the crash data analysis results presented in this chapter:

(1) The separation distance between driveway exits and downstream U-turn bays significantly impacts the safety and operational performance of vehicles making right-turns followed by U-turns. The crash rate at weaving sections decreases with the increases of the separation distance, and the travel time drivers spend at weaving sections increases with the separation distance.
(2) On 4-lane divided roadways with 2 lanes in each direction, if U-turn bay is located at a median opening, the minimum separation distance between the driveway exit and the downstream median opening is found to be 350 ft. If U-turn bay is located at a signalized intersection, the minimum separation distance is found to be 500 ft.

(3) On 6 or 8-lane divided roadways with at least 3 lanes in each direction, if U-turn bay is located at a median opening, the minimum separation distance between the driveway exit and the downstream median opening is found to be 450 ft. If U-turn bay is located at a signalized intersection, the minimum separation distance is found to be 750 ft.

Figure 6-7. Procedure for Determining the Critical Separation Distances on 6 to 8-lane Divided Roadways
Table 6-4. Recommended Minimum Separation Distances

<table>
<thead>
<tr>
<th>Number of Lanes</th>
<th>Location of U-turn Bay</th>
<th>Critical Separation Distance (ft)</th>
<th>L\textsuperscript{a}(ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-lane</td>
<td>Median Opening</td>
<td>341</td>
<td>350</td>
</tr>
<tr>
<td>4-lane</td>
<td>Signalized Intersection</td>
<td>508</td>
<td>500</td>
</tr>
<tr>
<td>6 to 8-lane</td>
<td>Median Opening</td>
<td>457</td>
<td>450</td>
</tr>
<tr>
<td>6 to 8-lane</td>
<td>Signalized Intersection</td>
<td>774</td>
<td>750</td>
</tr>
</tbody>
</table>

\textsuperscript{a}Recommended minimum separation distances

Table 6-5. Travel Time at Weaving Sections with Recommended Minimum Separation Distances

<table>
<thead>
<tr>
<th>Number of Lanes</th>
<th>Location of U-turn Bay</th>
<th>L(ft)</th>
<th>Travel Time (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>Median Opening</td>
<td>350</td>
<td>14.7</td>
</tr>
<tr>
<td>4</td>
<td>Signalized Intersection</td>
<td>500</td>
<td>22.3</td>
</tr>
<tr>
<td>6 or more</td>
<td>Median Opening</td>
<td>450</td>
<td>21.6</td>
</tr>
<tr>
<td>6 or more</td>
<td>Signalized Intersection</td>
<td>750</td>
<td>34.0</td>
</tr>
</tbody>
</table>

It is important to note that, the separation distance defined in this study is the distance between a driveway exit and the downstream U-turn bay, which also includes the transition length and the exclusively left-turn bay. This study not only examined crash data occurred at weaving sections, but also the crash data at the transition lengths and the storage lengths. This methodology follows the fact that drivers could sometimes use the transition length and the storage length to perform the weaving maneuver, as observed in the field. From safety perspective, it is not desirable to perform a weaving maneuver at the transition length and the storage length. Thus, it was recommended by the author that a transition length and a storage
length be added to the minimum separation distance. The optimal separation distance for RTUT should include the minimum separation distance recommended by this study, plus the transition length and the length for a left-turn storage bay.

This study has not focused on the safety and operational performance of heavy vehicles. It can be estimated that the separation distance required by a heavy vehicle to perform a RTUT maneuver should be greater than that required by a normal passenger car. Future study could focus on this issue.
CHAPTER 7
CAPACITY ANALYSIS RESULTS

7.1 General

This chapter presents the capacity analysis results for U-turn movement. The following two different conditions were considered for analyzing the capacity of U-turn movement: (1) the condition where U-turns are provided at a signalized intersection; and (2) the condition where U-turns are provided at an unsignalized intersection. Capacity of U-turn movement at a signalized intersection depends on the capacity of the exclusive left-turn lane from which U-turns are provided. Adjustment factors were developed to quantify the effects of various percentages of U-turning vehicles on the capacity of the exclusive left-turn lane. The critical gap and follow-up time for U-turn movement was estimated. With the estimated critical gap and follow-up time, the Harders model was used to determine the capacity of U-turn movement at an unsignalized intersection. The operational performance of U-turn movement on 4-lane divided roadways with narrow medians was also evaluated. Recommendations are given about the minimum roadway width and median width to facilitate vehicles performs U-turn maneuvers at median openings.
7.2 Capacity of U-turn Movement at Signalized Intersections

At a signalized intersection, U-turning vehicles are mixed with left-turning vehicles in the exclusive left-turn lane. The capacity of U-turn movement at a signalized intersection depends on the capacity of the exclusive left-turn lane from which U-turning vehicles are provided. In HCM, a methodology is developed for estimating the capacity for a lane or a lane group at a signalized intersection. However, the HCM methodology does not consider the influence of U-turn movement. In HCM, U-turns are treated as left-turns for the estimation of saturation flow rate. However, the operational effects of U-turns and left-turns are different.

A pilot survey conducted at the early stage of this study has demonstrated that U-turning vehicles adversely impact the capacity of the exclusive left-turn lane; and the effects increase with the increase in the percentage of U-turning vehicles. When the capacity of an exclusive left-turn lane is estimated at a signalized intersection, it is essential to account for the influence of U-turning vehicles.

In this study, the effects of U-turns on the capacity of an exclusive left-turn lane were quantified by analyzing the relationship between the percentage of U-turning vehicles in the left-turn lane and the average queue discharge time for each turning vehicle. On the basis of the analysis results, adjustment factors for various percentages of U-turning vehicles were developed to quantitatively evaluate the capacity reduction due to the presence of U-turning vehicles in the exclusive left-turn lane. In order to determine the U-turn adjustment factors, the following variables need to be considered:
(1) Whether U-turns are made from exclusive left-turn lanes or shared lanes;
(2) The type of phasing (protected, permitted, or protected-plus-permitted); and
(3) The proportion of U-turning vehicles in the left-turn lane.

In this study, only the condition in which U-turns being accommodated at an exclusive-left turn lane with protected signal phasing was considered. The capacity of U-turn movement at a signalized intersection with permitted left-turn phase is beyond the research scope of this study. As mentioned before, vehicles making U-turns have lower turning speeds than those making left-turns. Therefore, U-turning vehicles may cause the following left-turning vehicles to slow down because of the difference in speeds between these two movements. When U-turning vehicles are mixed with left-turning vehicles in a left-turn traffic stream, the discharging queue will consume more green time than those queues with only left-turning vehicles. Theoretically, the difference increases with the increase in the percentage of U-turning vehicles in the queue. In this study, a regression model was developed to estimate the relationship between the various percentages of U-turning vehicles in the left-turn lane and the average queue discharge time for each turning vehicle. The average queue discharge time for each turning vehicle was defined as the queue discharge time divided by the number of turning vehicles in the queue, as shown in Equation 7-1:

\[ h = \frac{T}{N_u + N_l} \]  

(7-1)

where, \( h \) = average queue discharge time for each turning vehicle (s),
\( T \) = queue discharge time (s),
The data collected were plotted with the average queue discharge time for each turning vehicle as the dependent variable and the percentage of U-turning vehicles as the independent variable, as shown in Figure 7-1. Several regression models were considered, and the regression results were compared. It was found that three different kinds of regression models were appropriate in describing the relationship, including a simple linear regression model, a linear regression model with an exponential form, and a linear regression model with a quadratic form (second-degree-polynomial regression model). It was found that the linear regression model with a quadratic form had the best goodness-of-fit to field data. The regression results are shown in Table 7-1. The equation of the model is given as:

$$ h = 0.000033 P_U^2 + 0.0033 P_U + 2.1399 $$  

(7-2)

where $h$ is the average queue discharge time for each turning vehicle (s), and $P_U$ is the percentage of U-turning vehicles in the left-turn lane. $P_U$ can be calculated as:

$$ P_U = \frac{N_u}{N_u + N_l} $$  

(7-3)

On the basis of the regression results, the model was statistically significant and the independent variables were also statistically significant. The adjusted $R^2$ value was .506. The unstandardized residuals were plotted against each independent variable in Figure 7-2 and Figure 7-3. As shown in Figure 7-2 and Figure 7-3, the residual plot for each independent
variable was randomly distributed about the x-axis line, which indicated that the model was correctly specified and the basic assumption about the homogeneous variance was not violated. By considering the intercept, which represents the average queue discharge time under “ideal” conditions if it is assumed that no U-turning vehicles were in the left-turn traffic stream, this model provided a reasonable value of 2.14 s.

\[ h = 3E-05x^2 + 0.0033P_u + 2.1399 \]

\[ R^2 = 0.51 \]

![Figure 7-1. Plot of Average Queue Discharge Time versus Various Percentages of U-turning Vehicles](image)

**Figure 7-1.** Plot of Average Queue Discharge Time versus Various Percentages of U-turning Vehicles

**Table 7-1. Regression Results for Queue Discharge Time Model**

<table>
<thead>
<tr>
<th>Independent Variables</th>
<th>Coefficient</th>
<th>Std. Error</th>
<th>t</th>
<th>Sig.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant</td>
<td>2.14</td>
<td>0.021</td>
<td>100.32</td>
<td>&lt; 0.001</td>
</tr>
<tr>
<td>( P_u^2 )</td>
<td>3.34E-05</td>
<td>0</td>
<td>2.48</td>
<td>0.014</td>
</tr>
<tr>
<td>( P_u )</td>
<td>0.0033</td>
<td>0.001</td>
<td>2.564</td>
<td>0.011</td>
</tr>
</tbody>
</table>

\[ R^2=0.51, \ R^2_{adj}=0.506 \]
Figure 7-2. Plot of Unstandardized Residuals versus the Independent Variable ($P_{UT}^2$)

Figure 7-3. Plot of Unstandardized Residuals versus the Independent Variable ($P_{u}$)
On the basis of the definition of the adjustment factors for turning movements, adjustment factor for U-turns on the left-turn saturation flow rate can be estimated by using the following equation:

\[
    f_{UT} = \frac{3600}{h} = \frac{h_0}{2.1399} = \frac{2.1399}{0.000033P_U^2 + 0.0033P_U + 2.1399}
\]  

(7-4)

where, \( f_{UT} \) = adjustment factor for U-turn movement,

\( h \) = average queue discharge time for U-turn and left-turn mix flow (s),

\( h_0 \) = base average queue discharge time for left-turn only flow (s), and

\( P_U \) = percentage of U-turning vehicles from inside left-turn lane (%).

With Equation 7-4, the adjustment factors for various percentages of U-turning vehicles were calculated and listed in Table 7-2. The data in Table 7-2 shows that U-turning vehicles have a considerable effect on the left-turn saturation flow rate, and the effect increases with the percentage of U-turning vehicles in the left-turn lane. For example, the U-turn adjustment factor for the queue with 40% of U-turning vehicles is 0.92, which implies an 8% capacity reduction in the left-turn lane.

<table>
<thead>
<tr>
<th>( P_U ) (%)</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{UT} )</td>
<td>0.99</td>
<td>0.98</td>
<td>0.96</td>
<td>0.94</td>
<td>0.92</td>
<td>0.9</td>
<td>0.87</td>
<td>0.84</td>
<td>0.82</td>
<td>0.79</td>
</tr>
</tbody>
</table>
The adjustment factors developed in this study can be directly used to estimate the capacity reduction in a left-turn lane due to the presence of U-turning vehicles if the signalized intersection has only one left-turn lane in the subject approach. If the signalized intersection has dual left-turn lanes, the adjustment factors can be applied only to adjusting the capacity of the inside left-turn lane, considering the fact that U-turns are usually accommodated from the inside left-turn lane.

The adjustment factors developed in this study were compared with the results of the previous two studies cited in the literature review. As shown in Figure 7-4, the curve of the proposed model generally conforms to but is somewhat lower than that in Adams and Hummer’s study. Among those adjustment factors, Tsao and Chu’s study predicts more severe effects than other two studies. This finding is not a surprise, because their study was conducted in Taiwan and the study results may not reflect the behaviors of motor vehicle drivers in the United States.

7.3 Capacity of U-turn Movement at Unsignalized Intersections

The use of restrictive medians and directional median openings has produced an increased number of U-turning vehicles at unsignalized intersections. Currently, however, there is no widely accepted procedure for estimating the capacity and level of service of U-turn movement at unsignalized intersections. U-turn movement has a smaller turning radius than other turning movements. In the current edition of the Highway Capacity Manual, a procedure was developed for estimating the capacity and level of service of different minor
traffic streams at unsignalized intersections. However, the HCM methodology does not contain the procedure for estimating the capacity and level of service of U-turn movement.

As explained in chapter 3, U-turn movement is usually provided from an exclusive left-turn lane at an unsignalized intersection. Vehicles making U-turns at an unsignalized intersection should yield to the major-street through-traffic stream. The potential capacity of U-turn movement at an unsignalized intersection generally depends on the following variables:

(1) The major-street through-traffic volume in the direction that is in conflict with U-turning vehicles;

Figure 7-4. Comparison of Research Results from Present Study with Those from Previous Studies
(2) The critical gap for U-turn movement; and

(3) The follow-up time for U-turn movement.

Besides, the width of median noses may also affect the potential capacity of U-turn movement at unsignalized intersections. As mentioned before, for a satisfactory design for U-turns, the width of the roadway should be wide enough to permit the design vehicle to turn from an exclusive left-turn lane in the median into the lane next to the outside shoulder. Vehicles at such locations should be able to perform U-turn maneuvers without any impedance. If the median width is narrow, however, vehicles might have limited physical space to perform the U-turn maneuver. Vehicles under this condition may make “tight” U-turns which have slower turning speeds and may require more turning time to perform the U-turn maneuver. Due to this fact vehicles making U-turns at unsignalized intersections with wide medians should, theoretically, have larger potential capacity than those making U-turns at unsignalized intersections with narrow medians.

The width of median nose at selected sites varies from 3 ft to 45 ft, as shown in Table 4-4. At site 1, site 2, and site 3, there are more than 70% of passenger cars need to encroach on to the shoulder to perform the U-turn maneuver. At site 4, site 5, and site 6, there are more than 65% of passenger cars make U-turns from the left-turn storage lane into the lane next to the shoulder. Less than 15% of passenger cars at those sites need to encroach on to the shoulder to perform the U-turn maneuver. The selected sites were then divided into two groups, including the “narrow median group” (site 1, site 2, and site 3) and the “wide median group” (Site 4, site 5, and site 6). The critical gap and follow-up time for U-turn movement...
under these two different conditions were analyzed separately. The following sections present the procedures for estimating the critical gaps and follow-up time for U-turn movement.

7.3.1 Critical Gap

In this study, the maximum likelihood method proposed by Miller and Pretty in 1968 was used to estimate the critical gap for U-turns. Detailed procedures for the maximum likelihood method were explained in chapter 3. The observed largest rejected gap varies from 0.6 s to 10.1 s with an average of 4.7 s. The accepted gap varies from 4.0 s to 29.6 s with an average of 10.1 s. Among the 387 observations, there are 6 vehicles having the largest rejected gap being greater than the accepted gap. These data were withdrawn from analysis because they violated the basic assumption of the maximum likelihood method.

The maximum likelihood method assumes that each individual driver has a constant critical gap. The critical gaps for the whole driver population are log-normally distributed with a mean of $\mu$ and a standard deviation of $\sigma$. These two parameters can be solved iteratively by some computer programs. In this study, the statistical software Gauss 4.0 was used. Gap acceptance data in the “wide median” group and the “narrow median” group were analyzed separately. The maximum-likelihood estimators converged very well when the starting point was close enough to the solution. The Gauss output forms are shown in Figure 7-5 and Figure 7-6.
With the estimated $\mu$ and $\sigma$ value, the mean critical gap $t_c$ and the variance of critical gaps $s^2$ for a minor movement can then be computed by Equation 3-12 and Equation 3-13, respectively. The estimated critical gaps are shown in Table 7-3. If U-turns are accommodated at unsignalized intersections with wide medians, the critical gap is found to
be about 6.4 s. If U-turns are accommodated at unsignalized intersections with narrow medians, the critical gap is found to be about 6.9 s. Based on the estimated mean and standard deviation values, the gap acceptance curves were developed. As shown in Figure 7-7 and Figure 7-8, the cumulative curves of critical gaps are situated between the distribution curves of the largest rejected gaps and the distribution curves of the accepted gaps. This is a result of the basic assumption made by the maximum likelihood method that a particular driver’s critical gap is always smaller than his accepted gap and greater than his largest rejected gap.

### Table 7-3. Analysis Results for Critical Gaps

<table>
<thead>
<tr>
<th>Median Type</th>
<th>Base Critical Gap, $t_{c,base}$</th>
<th>Mean (s)</th>
<th>Std.</th>
<th>Obs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Narrow Median</td>
<td></td>
<td>6.9</td>
<td>1.00</td>
<td>167</td>
</tr>
<tr>
<td>Wide Median</td>
<td></td>
<td>6.4</td>
<td>1.33</td>
<td>214</td>
</tr>
</tbody>
</table>

#### 7.3.2 Follow-up Time

A queue discharge time model was developed for estimating the follow-up time for U-turn movement. The dependent variable of this model is the queue discharge time for a queue of U-turning vehicles using a same gap. The independent variable is the number of U-turning vehicles in the queue. The full model can be represented as follows:

$$t_q = \alpha + \beta n + \varepsilon \quad (7-5)$$

where, $t_q = \text{queue discharge time for a queue of U-turning vehicles using a same gap (s)}$, ...
\[ \alpha = \text{start-up lost time associated with the first several queued U-turning vehicles plus time required by the first vehicle to perform the U-turn maneuver (s),} \]
\[ \beta = \text{saturation headway for U-turning vehicles (follow-up time) (s), and} \]
\[ \varepsilon = \text{error term.} \]

Figure 7-7. Gap Acceptance Curves for U-turns at Wide Medians

The queue discharge time method has been widely used for estimating the saturation headway for a particular movement at signalized intersections. The coefficient \( \beta \) estimated by this method is, in fact the saturation headway for U-turning vehicles, which based on the definition in HCM, is equal to the follow-up time.
The queue discharge time was measured from the time when the first vehicle started making a U-turn until the time when the last vehicle in the queue finished the U-turn maneuver. Only those vehicles that had come to a complete stop at the median openings were included in the follow-up time analysis. The follow-up time for wide median U-turns and narrow median U-turns were analyzed separately. The model equations and field data were illustrated in Figure 7-9. Both models are reasonably in good fit with the observed data in terms of the high $R^2$ values (0.89 and 0.95). For U-turns at unsignalized intersections with wide medians, the follow-up time is found to be about 2.5 s. For U-turns at unsignalized intersections with narrow medians, the follow-up time is found to be about 3.1 s. The results are also shown in Table 7-4.
t_q = 3.11n + 2.65 (R^2 = 0.95)  
Narrow Median

\[ t_q = 2.48n + 2.26 \, (R^2 = 0.89) \]  
Wide Median

Figure 7-9. Queue Discharge Time Model for U-turn Movement

Table 7-4. Analysis Results for Critical Gap and Follow-up Time

<table>
<thead>
<tr>
<th>Median Type</th>
<th>Base Follow-up Time, ( t_{f,\text{base}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean (s)</td>
</tr>
<tr>
<td>Narrow Median</td>
<td>3.1</td>
</tr>
<tr>
<td>Wide Median</td>
<td>2.5</td>
</tr>
</tbody>
</table>

7.3.3 Capacity Model for U-turn Movement

With the estimated critical gaps and follow-up time, Harders model can be used for estimating the potential capacity of U-turn movement at unsignalized intersections. The potential capacity of U-turn movement can then be calculated using the following equations:
\[ c_{p,w} = V_c \frac{e^{-0.00178V_c}}{1 - e^{-0.00064V_c}} \]  (7-6)

\[ c_{p,n} = V_c \frac{e^{-0.00192V_c}}{1 - e^{-0.00086V_c}} \]  (7-7)

where, \( c_{p,w} \) = potential capacity of U-turns at unsignalized intersections with wide medians (veh/hr), 
\( c_{p,n} \) = potential capacity of U-turns at unsignalized intersections with narrow medians (veh/hr), and 
\( V_c \) = major-street through-traffic volume in the direction that is in conflict with U-turns (veh/hr).

If the width of median nose at a particular unsignalized intersection is wide enough to accommodate U-turns, Equation 7-6 can be used to calculate the potential capacity of U-turn movement. If the width of median nose at a particular unsignalized intersection is narrow and passenger cars need to encroach onto the shoulder to perform the U-turn maneuver, the potential capacity of U-turn movement should be estimated using Equation 7-7. The potential capacities of U-turn movement at different levels of conflicting volumes are given in Figure 7-10. The curves in Figure 7-10 show that the potential capacity of U-turn movement decreases with the increases in the conflicting major-street flow rate, and vehicles making U-turns at unsignalized intersections with wide medians have larger potential capacity than those making U-turns at unsignalized intersections with narrow medians. These results are reasonable and consistent with what we expected.
7.3.4 Capacity of the Exclusive Left-turn Lane

With the potential capacity of U-turn movement, the capacity of an exclusive left-turn lane at an unsignalized intersection can be estimated. In practice, U-turning traffic is usually mixed with major-street left-turning traffic in the exclusive left-turn lane. The HCM has provided a method to compute the shared-lane capacity at TWSC intersections. The capacity of the exclusive left-turn lane can then be estimated by the following equation (cf. TRB, 2000):

\[
 c_{LT} = \frac{V_U + V_l}{\frac{V_U}{c_{p,U}} + \frac{V_l}{c_{p,l}}} \tag{7-8}
\]

Figure 7-10. Potential Capacity for U-turn Movement at Unsignalized Intersections
where, \( c_{LT} \) = capacity of the exclusive left-turn lane (veh/hr),

\[ V_U = \text{flow rate of U-turn movement in the exclusive left-turn lane (veh/hr)}, \]

\[ V_I = \text{flow rate of left-turn movement in the exclusive left-turn lane (veh/hr)}, \]

\[ c_{p,U} = \text{potential capacity of U-turn movement (veh/hr)}, \]

\[ c_{p,I} = \text{potential capacity of major-street left-turn movement (veh/hr)}. \]

### 7.3.5 Capacity Model Test

Data collected from site 2 were used for capacity model testing. Site 2 is a perfect site for capacity model testing in terms of the following characteristics:

(1) This intersection is a specially designed mid-block U-turn median opening. There is no disturbance from other minor movements;

(2) The distance from the median opening to the upstream signalized intersection is greater than 1 mile. Therefore, the upstream signal does not affect the arrival pattern of the major-street traffic; and

(3) The distance from the median opening to the downstream signalized intersection is around 2300 ft. Therefore, traffic from downstream signalized intersections does not back into the subject intersection.

An aerial photo of Site 2 is shown in Figure 7-11. The width of median nose at site 2 is 5 ft. At this site, there are more than 70% of passenger cars need to encroach onto the shoulder to perform the U-turn maneuver. Therefore, Equation 6-8 was used for calculating the potential capacity of U-turn movement at this site.
In order to measure the capacity of U-turn movement in the field, researchers usually need to find a site with a continuous queuing for U-turns. That means the U-turn movement at the selected site should be in an oversaturated state. However, it is usually difficult to find a site with an oversaturated U-turning traffic flow. Therefore, instead of directly measuring the capacity of U-turn movement in the field, a special method developed by Kyte in 1992 was used. Kyte et al. developed a method for measuring the capacity of a minor movement at an undersaturated state (Kyte et al. 1992). The method can be described in the following equation:

\[
    c_f = \frac{3600}{t_s + t_{mv}}
\]

(7-9)

where, \(c_f\) = capacity of U-turn movement measured in the field using Kyte’s method (veh/hr),

\(t_s\) = average service delay for each U-turning vehicle (s), and

\(t_{mv}\) = average move-up time for each U-turning vehicle (s).
The service delay for each U-turning vehicle was defined as delay occurs at the first position of the U-turn queue. The move–up time is the amount of time from when the previous U-turning vehicle exits the stop line until the subsequent queued vehicle reaches the stop line (Kyte et al. 1992). Both of these two parameters can be directly measured in the field. The service delay and conflicting major-street flow rate were collected in the field based on 5-minute time interval data.

A validation model was developed to test the gap acceptance model against field data. The dependent variable of this model is the capacity of U-turn movement estimated using the gap acceptance model. The independent variable of this model is the capacity of U-turn movement measured in the field using Kyte’s method. Figure 7-12 shows the validation results. The validation model can be represented as follows:

\[ c_m = 0.8c_f + 144.3 \]  \hspace{1cm} (7-10)

where \( c_m \) is the capacity of U-turn movement which is estimated using Harder’s model (veh/hr), and \( c_f \) is the capacity of U-turn movement which is measured in the field using Kyte’s method (veh/hr). As shown in Figure 7-12, the proposed model slightly overestimates the capacity of U-turn movement under low volume conditions and underestimates the capacity of U-turn movement under high volume conditions. Ideally, the best fit line through the test data would have an intercept of 0 and a slope of 1. The fitted line has an intercept of 144.3 and a slope of 0.8, which is very close to the ideal line. In general, the proposed gap acceptance model provides reasonable capacity estimate for U-turn movement at unsignalized intersections.
7.4 U-turns on 4-lane Roadways with Narrow Medians

One of the key factors that affect the operational performance of U-turns on 4-lane divided roadways is the turning radius accommodated by roadways to negotiate vehicles making U-turns. For a satisfactory design for U-turn maneuvers, the width of the roadway should be wide enough to permit the design vehicle to turn from an exclusive left-turn lane in the median into the lane next to the outside shoulder. The turning radius of the design vehicle could be accommodated by the combination of the median width (without including the width of exclusive left-turn lane) and the receiving lane width. If the roadway width and median width are not wide enough, extra pavement width should be added through use of a
taper, a flare or a loon to facilitate vehicles make U-turn maneuvers. A sketch about the turning radius at a median opening is shown in Figure 7-13.

![Figure 7-13. Combination of the Turning Radius at a Median Opening](image)

It can be estimated that the average turning speed for vehicles making U-turn maneuvers at different median openings should be a relatively constant value if the roadway width and median width are wide enough and drivers can perform continuous U-turn maneuvers without any impedance. However, if a roadway fails to provide a sufficient turning radius, vehicles at such a location may make “tight” U-turns which have slower turning speeds and may have longer turning time to perform the U-turn maneuver. The average turning time for vehicles making U-turn maneuvers at a median opening is a good indicator to judge if a particular median opening has enough turning radius to facilitate
vehicles make a continuous U-turn maneuver without causing operational problems at the 
median opening.

In this study, field measurements were conducted at 16 selected median openings in 
the Tampa Bay area of Florida. The selected sites were located on urban or suburban 4-lane 
divided arterials with 2 lanes receiving U-turns. A video camera was set up in the field to 
record traffic data. The recorded videotapes were later reviewed in the laboratory. The 
information extracted from reviewing videotapes includes the turning time for a vehicle 
making a U-turn, delay for a U-turning vehicle at a median opening and the U-turn traffic 
volume. Major-road through-traffic volume was recorded using a Hi-star portable traffic 
analyzer installed on the pavement. Geometric data such as median width, roadway width 
and extra pavement width were directly measured in the field using a measuring wheel.

Analysis of field data reveals that there exists a relationship between the turning 
radius at a median opening and the average turning time for vehicles making U-turns at the 
median opening. A regression model was developed based on field data to describe this 
relationship. The dependent variable of this model is the average turning time for vehicles 
making U-turns at a median opening. The independent variable is the turning radius at each 
site. The turning time for vehicles making U-turns was defined as the total elapsed time from 
the time when a vehicle starts making a U-turn until the time when it finishes the U-turn 
movement.

The average turning time for U-turning vehicles at selected median openings varies 
from 4.3 s to 7.0 s with an average of 5.1s. The range of the turning radius at each site varies 
from 34 ft to 83 ft with an average of 56.4 ft. Several model formats were tried. It was found
that the linear regression model with an inverse exponential form has the best goodness of fit to field data. The regression results are shown in Table 7-5. Based on the regression results, the following equation is obtained:

\[
t = 3.23e^{\frac{22.55}{r}}
\]  
(7-10)

where, \( t \) = the average turning time for vehicles performing U-turn maneuvers at a median opening (s), and

\( r \) = the turning radius accommodated at a median opening to facilitate vehicles making U-turns (ft)

Table 7-5. Regression Results for the Average Turning Time Model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Coefficients</th>
<th>t</th>
<th>Sig.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>1.17</td>
<td>18.98</td>
<td>&lt; 0.0001</td>
</tr>
<tr>
<td>1/r</td>
<td>22.55</td>
<td>7.39</td>
<td>&lt; 0.0001</td>
</tr>
</tbody>
</table>

\( R^2 = 0.796, \ R^2_{adj} = 0.782 \)

The regression model has a fairly high \( R^2 \) value (.796). The T-statistics indicate that the turning radius at a median opening significantly impacts the turning speed for vehicles making U-turns. In Figure 7-14, the curve for the fitted model is plotted against the observed turning time data. As shown in Figure 7-14, the average turning time for vehicles making U-turns at a median opening decreases with the increases of the turning radius and reaches a relatively stable state after the turning radius reaches around 46-48 ft. A turning radius of 46 ft is sufficient for most types of design vehicles (except heavy vehicles) to perform a U-turn.
maneuver without any impedance. This conclusion can help traffic engineers and designers decide whether U-turn movement should be permitted at a particular median opening. For example, if an exclusive left-turn lane is required for a 4-lane arterial, the minimum full median width (including exclusive left-turn lane) for vehicles making continuous U-turn maneuvers at a median opening can be estimated using the following equation:

\[ M_w = 46 - 2 \times L_w + L_L \]  

(7-11)

where, \( M_w \) = full median width (ft) (including the exclusive left-turn lane width),

\( L_w \) = the lane width of each through-traffic lane (ft), and

\( L_L \) = lane width of each left-turn storage lane.

Figure 7-14. Average Turning Time for Vehicles Making U-turns versus the Turning Radius at a Median Opening
If the roadway width and median width at a median opening fails to meet the requirements about the minimum turning radius, consideration should be given to adding extra pavement width through use of a taper, a flare or a loon to facilitate vehicles making U-turns. If the geometric condition at the particular median opening does not allow vehicles to use extra pavement width to make U-turn movements, U-turns should not be provided at this location.

One of the concerns with regard to the use of extra pavement width to facilitate vehicles make U-turns is that vehicles making U-turns at such locations may experience longer delay at the U-turn bay. In the previous section of this chapter, it has been demonstrated that vehicles making U-turns at unsignalized intersection with narrow medians have smaller potential capacity than those making U-turns at unsignalized intersections with wide medians. In this section, a regression model was developed to quantitatively evaluate the operational effects of using extra pavement width as a supplement to the roadway width and median width to facilitate vehicles make U-turns. Data collected from 11 selected median openings were used to build this model. Out of the 11 selected sites, 6 sites have sufficient roadway width and median width to accommodate U-turns, while the other 5 sites have narrow medians, and as a result, vehicles in these sites need to use extra pavement width to perform the U-turn maneuvers. Figure 7-15 shows that a passenger car uses a flare to perform the U-turn maneuver at a selected median opening.

A total of 237 observations were used to build this model. The major-road through-traffic volume (in the direction that conflicts with U-turning vehicles) and the U-turn volume were selected as the explanatory variables. In addition, a dummy variable was defined to
identify whether vehicles need to use extra pavement width to perform the U-turn maneuver at a particular median opening. The collected average delay for U-turning vehicles varies from 3.6 s to 13.9 s with an average of 7.2 s. The major road through-traffic volume in the direction that is in conflict with U-turning vehicles varies from 1120 veh/hr to 2661 veh/hr with an average of 1699 veh/hr. The U-turn volume varies from 12 veh/hr to 141 veh/hr with an average of 70 veh/hr.

![Figure 7-15. A Passenger Car Uses a Flare to Perform the U-turn Maneuver](image)

The developed delay model is a linear regression model with an exponential form. The adjusted $R^2$ value of the delay model is .697. The selected explanatory variables are all statistically significant at a 95% level of confidence. The regression residuals were plotted against the fitted delay value in Figure 7-16. It was found that the residuals were randomly

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distributed around the y=0 axis, indicating the fact that the model was correctly specified and the homogeneous assumption about the error term was not violated. The regression results are shown in Table 7-6. According to these parameter estimates, the final regression equation was shown as follows:

\[
D = 1.22e^{0.0009 TV + 0.002 UV + 0.798 \text{Narrow}}
\]  

(7-12)

where, \(D\) = the average delay for vehicles making U-turns at a median opening (s), 
\(TV\) = the major-road through-traffic volume in the direction that is in conflict with U-turns (veh/hr), 
\(UV\) = U-turn volume at a median opening (veh/hr), and 
\(\text{Narrow}\) = Dummy variable (\(\text{Narrow} = 1\) if vehicles need to use extra pavement width to make U-turns; \(\text{Narrow} = 0\) if vehicles do not need to use extra pavement width to make U-turns).

From Equation 7-12, it is clear that delay for U-turning vehicles at median openings increases with the conflicting major-road through-traffic volume and U-turn volume. The coefficient of the dummy variable is positive, indicating the fact that the vehicles using extra pavement width to perform U-turn maneuvers will experience relatively longer delay at the median opening than those making U-turns at wide medians.

On the basis of the delay model, curves are developed to compare delay for U-turning vehicles at a median opening under given levels of traffic volumes and roadway geometric conditions. Curves in Figure 7-17 and Figure 7-18 are developed assuming that the major-
road through-traffic volume in the direction that is in conflict with U-turning vehicles is 1000 veh/hr and 2000 veh/hr, respectively.

![Figure 7-16. Plot of Unstandardized Residuals versus the Fitted Delay Data](image)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Coefficients</th>
<th>t</th>
<th>Sig.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intercept</td>
<td>-7.436</td>
<td>2.36</td>
<td>0.019</td>
</tr>
<tr>
<td>TV</td>
<td>0.008</td>
<td>17.4</td>
<td>&lt; 0.0001</td>
</tr>
<tr>
<td>UV</td>
<td>0.011</td>
<td>5.27</td>
<td>&lt; 0.0001</td>
</tr>
<tr>
<td>Narrow</td>
<td>6.677</td>
<td>18.1</td>
<td>&lt; 0.0001</td>
</tr>
</tbody>
</table>

$R^2 = 0.700$, $R^2_{adj} = 0.697$
Curves in Figure 7-17 show that vehicles using extra pavement width to make U-turns will result in up to 6.0 s more delay than those making U-turns at a median opening with a wide median if the U-turn volume at the median opening is generally less than 250 veh/hr. Usually, an addition delay of 6.0 s will not be considered an major traffic operations concern. However, if the conflicting major-road through-traffic volume in is set to be 2000 veh/hr, vehicles using extra pavement width to make U-turns will have up to 15 s more delay if the U-turn volume is 250 veh/hr. An additional delay of 15 s could result in a reduced level of service for U-turn movement at an unsignalized intersection from “B” to “D”, reflecting a significant deterioration in traffic operational performance.
7.5 Summary

This chapter presents the capacity analysis results for U-turn movement. On the basis of the capacity analysis results, the following major findings are made:

(1) U-turning vehicles adversely affect the capacity of the exclusive left-turn lane from which U-turns are provided; and the influence increases with the increase in the percentage of U-turning vehicles in the left-turn lane. When the capacity of an exclusive left-turn lane is estimated at a signalized intersection, it is essential to account for the capacity reduction due to the presence of U-turning vehicles,
especially when the percentage of U-tuning vehicles is relatively high (>40%). The effect can be quantified by applying the adjustment factors developed in this study.

(2) The width of median nose significantly affects the capacity of U-turn movement at unsignalized intersections. Vehicles making U-turns at unsignalized intersections with wide medians have larger potential capacity than those making U-turns at unsignalized intersections with narrow medians.

(3) If U-turns are accommodated at unsignalized intersections with wide medians where vehicles can make U-turn movements without any impedance, the critical gap is found to be about 6.4 s. If U-turns are accommodated at unsignalized intersections with narrow medians where vehicles need to encroach onto the shoulder to make U-turns, the critical gap is found to be about 6.9 s.

(4) If U-turns are accommodated at unsignalized intersections with wide medians, the follow-up time is found to be about 2.5 s. If U-turns are accommodated at unsignalized intersections with narrow medians, the follow-up time is found to be about 3.1 s.

(5) With the estimated critical gaps and follow-up time, the potential capacity of U-turn movement at unsignalized intersections can be estimated using Harders model. Field test shows that Harders model provided reasonable capacity estimate for U-turn movement at unsignalized intersections.

(6) The average turning speed of U-turning vehicles increases with the increase of the turning radius accommodated at a median opening and reaches a relatively stable state after the roadway width reaches around 46-48 ft.
(7) A roadway width of 46 ft is generally sufficient for most design vehicles (except heavy vehicles) to perform a continuous U-turn maneuver without impedance. If the roadway width is less than 46 ft, extra pavement width should be added through use of a taper, a flare, or a loon to facilitate vehicles make U-turns.

(8) If the major-road through-traffic volume and U-turn volume is moderate or low, vehicles using extra pavement width to perform U-turn movements do not constitute major traffic operations concern.

(9) U-turn movement should not be provided at a median opening where: (1) roadway width is not wide enough to facilitate vehicles make continuous U-turn movements without any impedance, and (2) the geometric condition at the particular median opening does not allow vehicles to use extra pavement width to make U-turn movements.

Note that the U-turn adjustment factors in this study are developed under some simplified conditions. The simplified conditions include:

(1) Vehicles making left-turns and U-turns from an exclusive left-turn lane;
(2) Vehicles making left-turns and U-turns under protected left-turn phase;
(3) The street segment has enough of a turning radius to accommodate U-turns;
(4) No heavy vehicles are in the left-turn lane; and
(5) There is no significant disturbance from the right-turning vehicles during the U-turn phase in the other approach of the intersection.
The condition where U-turning vehicles are provided at permitted left-turn phase and the condition where there exists significant disturbance from the right-turning vehicles during the U-turn phase are not considered for U-turn capacity analysis.

The potential capacity of U-turn movement at unsignalized intersections has not considered the effects of other minor movements at unsignalized intersections. In practice, drivers making U-turns at unsignalized intersections are in conflict with the minor-street right-turning vehicles. However, the priority of U-turn movement at unsignalized intersections has yet to be clearly defined. If major-street U-turning traffic should yield to the minor-street right-turning traffic at unsignalized intersections, a capacity adjustment factor should be considered to account for the impedance effect of right-turning traffic.

In addition, the delay model developed in this chapter is based on traffic data collected from urban or suburban 4-lane divided roadways. Therefore, the delay model cannot be directly applied to the multilane highway in a rural environment. This study found that a roadway width of 46 ft is generally sufficient for most design vehicles to perform a continuous U-turn maneuver without impedance. This conclusion does not consider the turning radius of U-turning heavy vehicles. Future studies could focus on these issues.
CHAPTER 8
SUMMARIES, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summaries

One of the major principles of access management is to use non-traversable medians and directional median openings to manage left-turn movements on multilane highways. This design prohibits direct left-turn movements on major arterials. Drivers desiring to make a direct left-turn from driveways or side streets onto major arterials would be relocated to a downstream median opening or signalized intersection to make U-turns.

The installation of restrictive medians and directional median openings has resulted in an increased number of U-turning vehicles on multilane highways. Arguments have been advanced by some opponents of median modification projects that the increased numbers of U-turning vehicles may result in safety and operational problems on multilane highways.

There have been a considerable number of studies conducted concerning the safety effects of U-turns. However, relatively fewer studies are available concerning the operational performance of U-turn movement. During 2002 to 2004, a series of research projects concerning the safety and operational effects of U-turns were conducted by the University of South Florida. The USF studies took three basic approaches in evaluating a widely used access management technique – using right-turns followed by U-turns at a downstream U-
turn location as an alternative to direct left-turns from a driveways or a side street, including crash data analysis, conflict data analysis, and traffic operations data analysis. This dissertation presented some key findings of the USF studies. The primary objective of this research is to evaluate the operational performance of U-turn movement.

In order to achieve the research objectives, extensive data collection work were conducted. Two different types of data were collected, including traffic operations data and crash data. In order to collect traffic operations data, extensive field measurements were carried out at 40 selected sites in the Tampa Bay area of Florida. Video cameras were set up in the field to record traffic data. A total of more than 1000 hours of traffic data were recorded. Three years crash data were investigated for 179 selected roadway segments in central Florida. The crash data at selected roadway segments were obtained from the FDOT crash database.

Statistical analysis was conducted based on the collected traffic operations data and crash data to quantitatively evaluate the operational performance of U-turn movement. Delay and travel time data were compared for three different driveway left-turn alternatives that are widely implemented in Florida and nationally. These driveway left-turn alternatives include: (1) direct left-turns at a driveway; (2) right-turns followed by U-turns at a downstream signalized intersection; and (3) right-turns followed by U-turns at a downstream median opening in advance of a signalized intersection.

The average vehicle delay was compared for various driveway left-turn alternatives under different levels of major-street through-traffic volume and driveway volume. A travel time model was developed to predict the travel time drivers spent at different weaving
sections while making right-turns followed by U-turns. With the delay comparison results and the travel time model, the total travel time was compared for various driveway left-turn alternatives under a given traffic volume and roadway geometric condition. It was found that the access management treatment: using right-turns followed by U-turns at a median opening as an alternative to direct left-turns at a driveway, does not result in longer vehicle delay and travel time.

Three years crash data was investigated for 179 selected roadway segments in central Florida. Crash rate models were developed to evaluate how the separation distances between driveway exits and downstream U-turn bays impact the safety performance of vehicles making right-turns followed by U-turns. With the crash data analysis results, the minimum separation distances under different roadway conditions were determined to facilitate driver use of right-turns followed by U-turns.

The capacity of U-turn movement was analyzed under the following two different situations: (1) the condition where U-turns are provided at a signalized intersection; and (2) the condition where U-turns are provided at an unsignalized intersection. Capacity of U-turn movement at a signalized intersection depends on the capacity of the exclusive left-turn lane from which U-turns are provided. Adjustment factors were developed to quantify the effects of various percentages of U-turning vehicles on the capacity of the exclusive left-turn lane. The critical gaps and follow-up time for U-turn movement were estimated. With the estimated critical gaps and follow-up time, the Harders model was used to determine the capacity of U-turn movement at an unsignalized intersection. Field test shows that the
proposed model provided reasonable capacity estimate for U-turn movement at unsignalized intersections.

The operational performance of U-turn movement on 4-lane divided roadways with narrow medians was evaluated based on the traffic operations data collected from 16 selected median openings. A regression model was developed to describe the relationship between the turning radius at a median opening and the average turning time for vehicles making U-turns at a median opening. Based on the turning radius and turning time analysis results, recommendations are made about the minimum roadway width and median width to facilitate vehicles performs U-turn maneuvers at median openings.

8.2 Conclusions

This study evaluated the operational performance of U-turn movement. On the basis of the observed data and analysis, the following conclusions are made:

- The access management treatment: using right-turns followed by U-turns at a median opening as an alternative to direct left-turns at a driveway, does not result in longer vehicle delay and travel time. However, if U-turns are provided at a signalized intersection, vehicles making right-turns followed by U-turns have longer delay and travel time than those making direct left-turns at a driveway.

- If the left-turn traffic demand at a driveway is high, consideration should be given to providing enough U-turn opportunities in advance of the downstream signalized intersection. The optimal location to facilitate U-turning vehicles is a mid-block
median opening in advance of a signalized intersection with an appropriate separation distance from the subject driveway.

- The separation distance between driveway exits and downstream U-turn locations significantly impacts the safety and operational performance of vehicles making right-turns followed by U-turns. The crash rate at weaving sections decreases with the increases of the separation distance, and the travel time drivers spend at weaving sections increases with the separation distance.

- On 4-lane divided roadways with 2 lanes in each direction, if U-turns are provided at a median opening, the minimum separation distance between the driveway exit and the downstream median opening is found to be 350 ft. If U-turns are provided at a signalized intersection, the minimum separation distance is found to 500 ft.

- On 6 or 8-lane divided roadways with at least 3 lanes in each direction, if U-turns are provided at a median opening, the minimum separation distance between the driveway exit and the downstream median opening is found to be 450 ft. If U-turns are provided at a signalized intersection, the minimum separation distance is found to be 750 ft.

- U-turning vehicles adversely affect the capacity of the exclusive left-turn lane from which U-turns are provided; and the influence increases with the increases in the percentage of U-turning vehicles in the left-turn lane. When the capacity of an exclusive left-turn lane is estimated at a signalized intersection, it is essential to account for the capacity reduction due to the presence of U-turning vehicles. The effect can be quantified by applying the adjustment factors developed in this study.
• The width of median nose significantly affects the capacity of U-turn movement at unsignalized intersections. Vehicles making U-turns at unsignalized intersections with wide medians have larger potential capacity than those making U-turns at unsignalized intersections with narrow medians.

• If U-turns are accommodated at unsignalized intersections with wide medians where vehicles can make U-turn movements without any impedance, the critical gap of U-turn movement is found to be about 6.4 s, the follow-up time is found to be about 2.5 s. If U-turns are accommodated at unsignalized intersections with narrow medians where vehicles need to encroach onto the shoulder to make U-turns, the critical gap of U-turn movement is found to be about 6.9 s, the follow-up time is found to be about 3.1 s.

• With the estimated critical gaps and follow-up time, the potential capacity of U-turn movement at unsignalized intersections can be estimated using Harders model. Field test shows that Harders model provided reasonable capacity estimate for U-turn movement at unsignalized intersections.

• The average turning speed of U-turning vehicles increases with the increase of the turning radius accommodated at a median opening and reaches a relatively stable state after the roadway width reaches around 46-48 ft. A roadway width of 46 ft is generally sufficient for most types of design vehicles (except heavy vehicles) to perform a continuous U-turn maneuver without impedance. If the roadway width is less than 46 ft, extra pavement width should be added to facilitate vehicles make U-turns. If the major-road through-traffic volume and U-turn volume is moderate to
low, vehicles using extra pavement width to perform U-turn movements do not have a large negative operational effect.

- U-turn movement should not be provided at a median opening when the following two conditions are met: (1) the roadway width is not wide enough to facilitate vehicles make continuous U-turn movements without any impedance, and (2) the geometric condition at the particular median opening does not allow vehicles to use extra pavement width to make U-turn movements.

### 8.3 Recommendations

The research results of this study can help traffic engineers and designers make decisions about the design and selection of median treatments and various driveway left-turn alternatives on multilane highways. The delay and travel time comparison results show that the access management technique: providing right-turns followed by U-turns at a downstream median opening as an alternative to direct left-turns at a driveway does no result in longer delay and travel time. This conclusion is particular helpful in addressing the public concerns with regard to the delay and travel time for indirect left-turn treatments.

The capacity analysis results provided a tool for analyzing the capacity for U-turn movement at unsignalized intersections and signalized intersections. The adjustment factors developed in this study can be directly used to estimate the capacity reduction in an exclusive left-turn lane due to the presence of U-turning vehicles. The proposed U-turn adjustment factors and U-turn capacity model provide a supplement to the current HCM capacity estimation methodology for signalized intersections and unsignalized intersections.
It is important to note that, delay and travel time analysis in this study was conducted based on the traffic operations data collected from urban or suburban multilane highways, where traffic signals could significantly impact the headway distribution in the major-street through-traffic. The delay and travel time comparison results cannot be directly applied to a rural traffic environment, where the major-street through-traffic is generally in a free flow state.

This study does not make a distinction between the situation on 6-lane divided roadways and on 8-lane divided roadways. It was assumed that the operational effects of U-turns on 6-lane divided roadways and on 8-lane divided roadways are the same. This assumption is not perfect. The reason for making this assumption is that there are very few 8-lane divided roadways in central Florida where the field data collection work was undertaken. Thus, the data collected from 8-lane divided roadways is too few for us to draw some defensible conclusions. If more data is available, the operational performance of U-turns on 8-lane divided roadways should be analyzed separately.

Another limitation of this study is that the operational effects of U-turning heavy vehicles are not evaluated. It is also because of the difficulty in collecting related data. In practice, it is usually difficult to find appropriate sites with large numbers of U-turning heavy vehicles. Until now, the operational effects of U-turning vehicles are still largely unknown.

In addition, this research found that providing the U-turn location at an unsignalized location before the traffic signal has many positive operational impacts. However, finding an appropriate location for this U-turn median opening before a traffic signal in built-out areas,
sometimes, can be difficult due to the tight geometric conditions found there. In this condition, it was recommended by the author that another access management treatment: providing right-turns followed by U-turns at a median opening after a signalized intersection, could be considered. Future study could focus on these issues.
REFERENCES


APPENDICES
## Appendix A: Selected Sites for Traffic Operations Data Collection

### Table A-1. The Selected Sites Where U-turns are Provided at Signalized Intersections

<table>
<thead>
<tr>
<th>Site</th>
<th>City</th>
<th>Location of U-turn Bay</th>
<th>N&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Speed&lt;sup&gt;b&lt;/sup&gt;</th>
<th>Median Type</th>
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<tbody>
<tr>
<td>Bruce B. Downs Blvd.</td>
<td>Tampa</td>
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<td>45</td>
<td>D&lt;sup&gt;c&lt;/sup&gt;</td>
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<td>Maple Dale St.</td>
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</table>

<sup>a</sup> Number of through lanes in the major-street  
<sup>b</sup> Speed Limit in the Major-road  
<sup>c</sup> The Median Opening at the selected driveway is a directional median opening  
<sup>d</sup> The median opening at the selected driveway is a full median opening
### Table A-2. The Selected Sites Where U-turns are Provided at Median Openings

<table>
<thead>
<tr>
<th>Site</th>
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<th>Speed</th>
<th>Median Type</th>
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Appendix B: Pictures Taken in the Field

Figure B-1. A Driver Goes in the Wrong Direction to Avoid Making a U-turn at the Signalized Intersection
Appendix B: (Continued)

Figure B-2. A Truck Making a U-turn
Figure B-3. Undesirable Direct Left-turn Movement at a Directional Median Opening
Appendix B: (Continued)

Figure B-4. A Severe Conflict between a Left-turning Vehicle and the Through Traffic
Figure B-5. Traffic Congestion at a Traditional Full Median Opening
Figure B-6. A Severe Conflict between a “Left-turn-Out” Vehicle and a “Left-turn-in” Vehicle
Figure B-7. A Severe Conflict between a U-turning Vehicle and the Through Traffic
ABOUT THE AUTHOR

Pan Liu was born on September 16, 1979 in a small city in China. At that time, his father was a soldier. His mother was an accountant. By naming him “Pan”, his parents want him to become a person who has a strong personality and will never give up when confronted with challenges.

Pan Liu attended Southeast University in Nanjing, China in 1996. And from there he received a Bachelor’s Degree in 2000 with majors in transportation and civil engineering. Pan Liu joined University of South Florida in January 1997 as a Ph.D. student and a Graduate Research Assistant. While pursuing his Doctoral Degree in transportation engineering, He is actively involved in several research projects funded by state DOT and other agencies. His research interests include: traffic operations, traffic safety and access management.