Abstract

CARTER, DANIEL LANCE. Operational and Safety Impacts of U-Turns at Signalized Intersections. (Under the direction of Dr. Joseph E. Hummer.)

With rapidly growing urban areas and construction of new developments, efficient access to the roadway network becomes a relevant issue. In the effort to balance safety, mobility, and access, many transportation officials are in favor of designs that employ raised medians on the main road. However, this decision draws much controversy from those opposed to the lack of direct access that comes with raised median designs. One of the issues in this controversy is the effect of increased U-turns at adjacent intersections. The purpose of this research is to determine the operational and safety effects of U-turns at signalized intersections.

The operational analysis involved measurements of vehicle headways in exclusive left turn lanes at 14 intersections. By regression analysis, I obtained an equation to estimate saturation flow reduction based on intersection characteristics. This equation indicates a 1.8% saturation flow rate loss in the left turn lane for every 10% increase in U-turn percentage and an additional 1.5% loss for every 10% U-turns if the U-turning movement is opposed by protected right turn overlap from the cross street.

The safety study involved a set of 78 intersections. Fifty-four sites were chosen randomly, and twenty-four sites were selected based on their reputation as U-turn "problem sites". Although the group of study sites was purposely biased toward sites with high U-turn percentages, the study found that 65 of the 78 sites did not have any collisions involving U-turns in the three-year study period, and the U-turn collisions at the remaining 13 sites ranged from 0.33 to 3.0 collisions per year. Sites with double left

turn lanes, protected right turn overlap, or high left turn and conflicting right turn traffic volumes were found to have a significantly greater number of U-turn collisions.

Operational and Safety Impacts of U-Turns at Signalized Intersections

by

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Chair of Advisory Committee

Biography

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The author was born and raised in Charlotte, North Carolina. He graduated with his Bachelor of Science degree in Civil Engineering from N.C. State University in 2001. After a year's stint as a volunteer English teacher in Romania, he returned to N.C. State to pursue a Master of Science degree in Civil Engineering with a transportation engineering concentration. He and his wife Alexa live in Cary, North Carolina.

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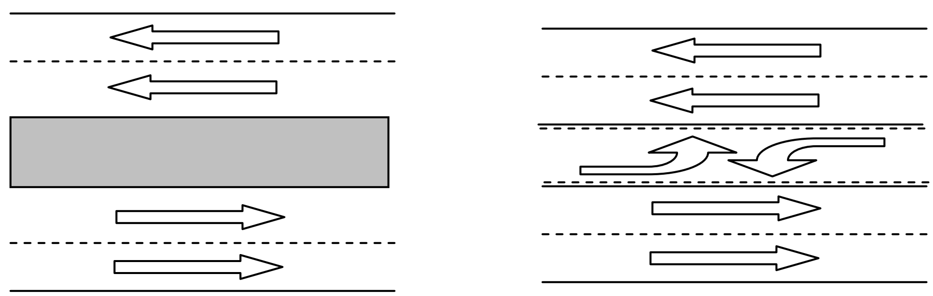
1. Introduction

1.1. Problem Definition

The traffic demand on urban highways consists of a dynamic and diverse group of drivers, including commuters, delivery vehicles, business traffic, and recreational drivers. With growing urban areas and the construction of new developments, efficient access to the roadway network becomes a relevant issue. Many departments of transportation have access management sections that seek to balance access to adjacent land parcels with safety and efficient traffic flow on the highway. One of the tools used in access management is a raised median, which reduces the number of conflict points on a roadway by decreasing the number of crossing movements a driver can make. However, access to businesses is not as direct with a raised median, and this issue draws heavy public involvement.

An example can be cited from the US-70 widening project in Salisbury, North Carolina in 2001. Before widening, US-70 had had three lanes, with the middle lane serving as a TWLTL lane. While transportation officials supported a median-divided design, the public took strong opposition to a median installation [1]. The dispute was carried in the media, eventually eliciting an editorial response from the state Secretary of Transportation. This example is only one of many such debates around the state. Many times the debate centers on the question: median or no median?

On the one hand, a median creates a divided cross-section which seems to be safer based on collision data from existing facilities [2]. Four-lane divided cross-sections are also more aesthetic and provide better midblock levels of service than undivided designs. On the other hand, an undivided cross-section with a center two-way-left turn-lane (TWLTL) allows direct access from both directions into adjacent properties, and is generally favored by landowners, developers, and many local government officials. These five-lane cross-sections are also narrower, which may save on construction costs.



Four-lane divided highway (raised median)   Five-lane undivided highway (TWLTL)

Figure 1.1. Cross-section Illustration

From an engineering perspective, the concern over this issue pertains to the performance of two parts of the roadway – the midblock segments and the intersections. A partner project to this research, entitled "Empirical Collision Model for Four-Lane Median Divided and Five-Lane with TWLTL Segments", compares the safety performance of the two cross-sections on midblock segments in North Carolina [3]. With a different but complementary scope, this project focuses on the effect of median installation on intersections.

The major effect on intersections is expected to be produced from U-turning vehicles. Drivers turning left from a minor driveway without a median opening would have to turn right and then make a U-turn at the nearest median opening. Drivers desiring to turn left from the main highway at a location without a median opening would have to proceed to the next available median opening, then U-turn and turn right at the intended driveway (Figure 1.2).

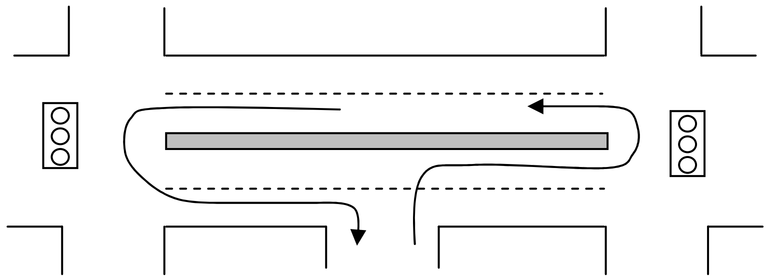


Figure 1.2. Flow of Traffic with Median on Main Highway

In this manner, a divided facility is expected to bring about an increased number of U-turns at intersections. Often these intersections are signalized and serve a large number of left-turning vehicles already. As can be seen in the banning of U-turns in urban areas across the United States, current opinion assumes that U-turns would decrease capacity and cause safety hazards. It is evident that the operational and safety effects of U-turns could be a major factor in the design decision. However, engineers have mostly been presented with speculation on this topic as past research has not conclusively addressed this issue. This project seeks to provide solid research to allow engineers and officials to make informed decisions on this hotly debated topic.

1.2. Research Objectives

1. To analyze the impacts of U-turns on left turn saturation flow rate at signalized intersections on median-divided facilities, and
2. To evaluate the safety impacts of U-turns at signalized intersections on median-divided facilities.

1.3. Scope

The scope of this project is limited to signalized intersections in the state of North Carolina. All sites have raised medians at the intersection, but no restriction is placed on the median length or width. Study sites are located on either four- or six-lane facilities. The operational research focuses only on the performance of passenger vehicles, thereby excluding capacity effects of heavy vehicles. This project only studies the impacts of U-turns on divided highways as it pertains to operational and safety impacts; other effects such as economic impact, pedestrian safety, and public perception are not included.

Another possible measure of performance for a median-divided highway would be the effect of a median on the average travel time of vehicles using the facility. While a raised median may cause an increase in travel time, this effect is not included in the scope of this research.

2. Literature Review

2.1. Operational Impacts of U-turns

The majority of the research on operational effects of U-turns has been conducted for unsignalized intersections. The current literature has little to offer concerning operational effects at signalized intersections. A few studies have been done to estimate the effect of U-turns on saturation flow rate, but the studies were hindered by small sample sizes.

The Highway Capacity Manual (HCM) method for capacity analysis of signalized intersections contains various factors such as opposing flow and proportion of left turns that reduce saturation flow for lane groups containing left turns [4]. However, there is no factor for the effect of U-turns on saturation flow. Also, these factors do not apply to exclusive left turn lanes with protected phasing, for which the HCM recommends a flat 0.95 adjustment factor. The need for a U-turn adjustment factor may increase with the growing popularity of nonconventional designs such as median U-turns and superstreets that integrate U-turns into their designs [5,6].

Adams studied U-turns at signalized intersections to determine whether a U-turn factor should be included in HCM capacity analyses [7]. His methodology involved measuring saturation flow for every left turn queue and noting the number and position of U-turning vehicles in the queue. He studied four signalized intersections during midday peak.

His results showed no correlation between saturation flow and percentage of U-turns for intersections with a maximum U-turn percentage less than 50. The analysis was inconclusive between 50 and 65 percent U-turns because of the small samples in the study. For sites having U-turn percentages greater than 65, the analysis showed that a saturation flow reduction factor would be statistically valid. Adams recommended tentative reduction factors of 0.9 for U-turn percentages between 65 and 85 and 0.8 for U-turn percentages exceeding 85. The study suggests further study of intersections with high percentages of U-turns. This project was subject to criticism due to small sample size. Also, his methodology used a queue average to obtain the saturation flow measurement. A measurement of individual vehicle headways would have shown more clearly the effect of U-turning vehicles.

Thakkar et al. produced a methodology to evaluate the impacts of prohibiting median opening movements [8]. Although her approach covered many factors (i.e., operations, safety, motorist's convenience, etc.), one aspect she studied was the effect of U-turns on the saturation flow of the left turn lane of the downstream signal. She analyzed operational performance using a TRANSYT-7F simulation. Given the lack of models to evaluate the operational effect of U-turns, Thakkar used linear regression analysis to produce her own model based on data from field observation. Her resulting model has the following form:

$$SF = 1803 - 4.323 * UTURN - 0.484 * UTURN * RTOA$$

where:
SF = saturation flow rate of mixed-use left turn/U-turn lane in veh/hr/lane,
RTOA = conflicting right turn volume from the cross street during the U-turn phase in veh/min, and
UTURN = U-turn percentage in the mixed-use lane.

Figure 2.1 shows her model in graphical form. Her analysis shows large effects when high U-turn percentage and high right turn volumes coincide.

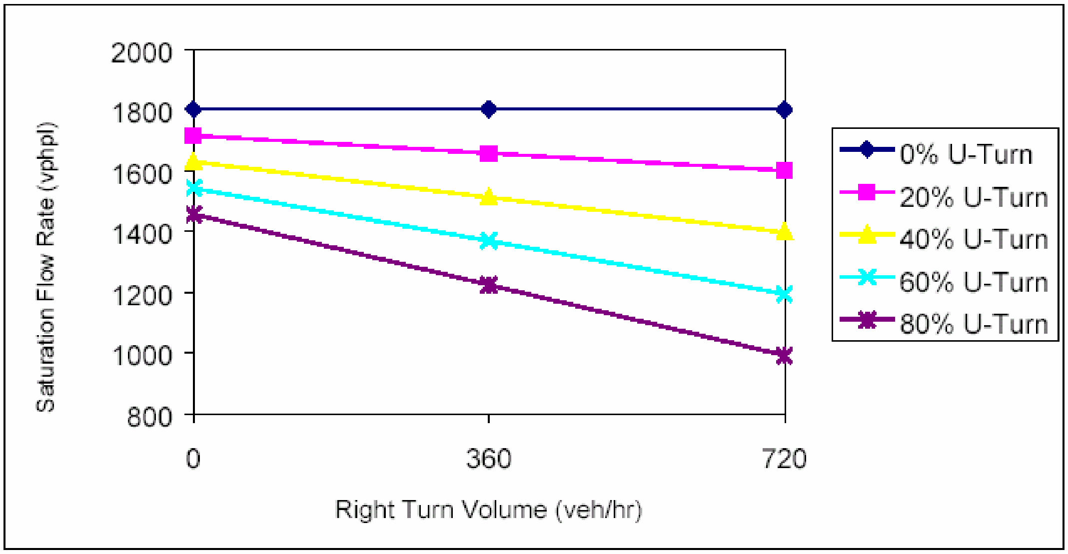


Figure 2.1. The Effect of Right Turn/U-Turn Movement Conflicts on the Saturation Flow Rate of U-turn/Left Turn Lane [8]

Her model is practical on a volume basis since it allows for varying degrees of U-turn percentage and right turn volume. However, the RTOA factor would have little impact on all but the highest volume intersections. The reviewed literature did not comment on the sample size or the goodness-of-fit at the study intersection. While this regression analysis is a good basis for U-turn analysis, it is specific to only one intersection. A calibration on more intersections would lead to greater confidence and wider applicability of the results.

There has been some research on U-turn capacity at unsignalized intersections. Al-Masaeid conducted a study on the capacity of U-turns at median openings [9]. His study included seven median openings in different cities in Jordan, all of which operated at-capacity. His analysis compared the capacity of the median opening to the amount of conflicting traffic flow. The result was an empirical linear regression model for U-turn capacity that appears as follows:

$$C = 799 - 0.31q_c$$

where:
C = capacity of U-turn movement (PCU/h); and
q_c = conflicting traffic flow (PCU/h)

This equation for U-turn capacity is heavily influenced by the amount of conflicting traffic. This is a logical result considering the large difference between the low speeds of U-turning vehicles and the high speeds of main highway traffic.

2.2. Safety Impacts of U-Turns

The safety impact of U-turning movements has been the subject of extensive research. Current research, however, has been devoted mostly to estimating the safety of U-turns at unsignalized intersections, such as median openings. A thorough search of research literature did not reveal any studies focused on the safety of U-turns at standard signalized intersections.

A study by Xu examined unsignalized intersections on divided highways where a minor street accessed the highway at a median opening [10]. She measured the collision reduction due to eliminating direct left turns from the minor streets by forcing drivers to turn right and make a U-turn. The collision data were collected over a sample of 258 sites with a total of 3,913 collisions over a three-year period. Her results showed that implementing this measure decreased the total crash rate by 26% and the injury/fatality crash rate by 32% for six-lane arterials. She did not consider U-turns at signalized intersections due to the fact that Florida DOT discouraged this practice. She states that U-turns at signalized intersections on major arterials degrade level of service and may cause serious conflicts with right-turning vehicles.

Dissanayake et al. conducted a similar study that looked at the safety performance of direct left turns as compared to right turns followed by U-turns at unsignalized intersections on major arterials [11]. Her study examined conflict rates at each type of site. The conflict sample size consisted of 300 hours of observation collected at seven sites, resulting in 1,654 conflicts. Her results show that total conflicts were significantly lower at sites with right turns followed by U-turns. While this is indicative of the overall safety performance of a design that incorporates U-turns, her scope did not include a study of conflicts or collisions directly resulting from or involving U-turns. The results of this study cannot be conclusively applied to signalized intersections considering that all sites studied by Dissanayake were unsignalized median openings.

These two studies show that designs that incorporate U-turns as a necessary movement are safer than designs that allow direct left turns. However, these findings are based on research at unsignalized intersections, and do not focus specifically on collisions involving U-turns. U-turns at signalized intersections have the potential to create a very different safety situation. This unknown effect provides the impetus for the safety aspect of this project.

3. Methodology

3.1. Operational Impacts of U-Turns

The operational effect of U-turns on left turn lanes has typically been a qualitative estimate. In an effort to quantitatively analyze this effect, I studied queues in exclusive left turn lanes with protected phasing at 14 sites. These studies measured vehicle headways, average delay, and turning movements. Conflict studies were also conducted to supplement findings in the safety analysis. It should be understood that the term “site” refers to one approach at an intersection, not the intersection as a whole.

3.1.1. Selection of Operational Sites

Sites for the operational part of this project had to have several characteristics to meet our study demands. The project team had originally set the following criteria for site selection:

1. *Two lanes receiving U-turns*. The motive for this project dictated that we prioritize sites with two lanes receiving the U-turns (i.e., four-lane divided facility). This geometry gives the best information about the effect of a median installation in the widening of a two-lane road.

2. *Sufficient left turn queue length*. A traditional saturation flow study requires a minimum queue length of seven vehicles. I searched for sites with an average queue length of seven vehicles or more.

3. *Sufficient percentage of U-turns*. In order to get the maximum amount of data per unit of time studied, I wanted some sites with an average U-turn percentage of 50% in the left

turn queue. Adams and Hummer, who conducted a similar study, concluded that queues with U-turn percentages lower than 50% had little effect on saturation flow [7].

4. *Local site*. To minimize travel costs, I looked for sites within a one-hour travel radius of Raleigh.

After searching the Raleigh area for sites, these criteria were found to be too strict to attain an appropriate number of sites. I revised the procedure and relaxed some selection criteria to come up with the following criteria:

1. *Two or three lanes receiving U-turns*. Although the research focus was directed toward four-lane divided facilities, the scope was expanded to sites with three lanes receiving U-turns. These sites would still provide useful data, and the data collection at these sites would be many times more efficient than at the next-best sites with two receiving lanes.

2. *Sufficient left turn queue length*. The planned process for measuring saturation headway was changed from a measurement of the average headway in a queue to a measurement of each individual headway using precise timing equipment. Excluding vehicles in queue positions one through four due to the effect of start-up lost time, this procedure would still allow the team to gather data from queues as short as five vehicles.

3. *Sufficient percentage of U-turns*. Since sites with 50% U-turns in the left turn queue were few and far between, I lowered the criterion to a level of 20%. This usually meant an average of 1 or 2 U-turns per cycle, and still provided some sites with 50% or more U-turns.

4. *Located in nearby major cities*. The Raleigh area did not yield a sufficient number of eligible sites, so the search radius was expanded to the cities of Winston-Salem, Charlotte, and Wilmington.

After all selection areas were searched for eligible sites, I had 14 sites that were appropriate for these operational studies (see Table 3.1). These sites were selected from 106 sites that I visited. Appendix D contains a list of all sites considered for selection. Appendix E contains the list of selected sites as well as all pertinent characteristics. The selected sites for this project are located in the metropolitan areas of Raleigh, Charlotte, and Winston-Salem.

Table 3.1. Sites Selected for Operational Studies

Site No.	Main Rd	Dir	Cross St	Left Turn Signal Type	Conflicting Right Turn	No. Left Turn Lanes	Median Width (ft)	No. Lns Rcvg
202	Cary Pkwy	NB	Kildaire Farm	prot	perm	1	16	2
203	US 64	WB	Edinburgh	prot	prot	2	20	2
204	US 15-501	NB	Ephesus Church	prot	perm	1	12	2
205	Harris Blvd	WB	N Tryon	prot	prot	2	15	2
206	I-277 ramp	NB	4th St	prot	none	2	4	2
207	N Tryon	NB	Harris Blvd	prot	prot	2	7	2
210	New Bern	WB	Sunnybrook	prot	prot	2	13	2
211	Silas Creek	WB	Miller	prot	prot	1	3	2
212	Capital	SB	Calvary	prot	perm	1	19	3
213	Capital	SB	Millbrook	prot	perm	2	6	3
215	US 64	EB	Trawick	prot	prot	2	14	3
216	US 70	EB	Pleas. Valley Prom.	prot	perm	1	15	3
217	Western	WB	Kent	prot	perm	1	7	3
218	Creedmoor	NB	Lynn	prot	prot	2	3	2

To locate sites outside the Raleigh area, I relied on the guidance I received from transportation engineers and personnel in the various cities. The lists of sites that they recommended saved a good amount of time and yielded several sites that were appropriate for the operational study. Many of the sites they recommended were also used in the recommended group of the safety study sites (see section 3.2.1).

The best study sites were usually located in urban areas on streets that border a high level of business development (i.e., restaurants, gas stations, and shopping centers). However some intersections turned out to have a sufficient number of U-turns though they were in unlikely places. Usually this was caused by the design of the highway that caused regular commuters to make U-turns as a part of their route.

Figure 3.1. I-277 Ramp at Fourth Street

The intersection of the I-277 ramp and Fourth Street near downtown Charlotte is a good example of U-turns caused by regular commuters. Fourth Street (only inbound) forms a one-way pair with Third Street (only outbound). The ramp from I-277 only intersects with Fourth Street. Vehicles wishing to travel away from downtown had to make a U-turn to a small road parallel with the ramp in order to get to Third Street, as shown in Figure 3.1. This was an unusual situation, but the high percentage of U-turns provided unique and valuable data.

3.1.2. Field Studies

The purpose of the operational field studies was to gather data to determine the effect of U-turns on intersection operation. The assumption was that U-turns would impact saturation flow only in the exclusive left turn lane. The team conducted operational studies measuring saturation headways, stopped delay, and volumes as well as a conflict study to compare with collision data.

The project team studied each site for six to nine hours, depending on the quality of the data and how many usable queues were observed. This study period consisted of consecutive hours spanning most of a day, usually from around 10:30 AM to 6:30 PM. The team consisted of three observers performing four tasks:

Observer 1: saturation headway measurements

Observer 2: conflict study and volume counts

Observer 3: stopped delay study

Due to the fact that U-turn conflicts were so infrequent, the tasks of conflict study and volume count were assigned to one person. This combination of tasks was manageable for one person and proved to work well. All observers used Jamar TDC-8 electronic counters. The use of these counters facilitated the collection and compilation of study information.

The study included sites with single left turn lanes as well as double left turn lanes. In the case of the double left turn lanes, only the inside turn lane was studied, since that was the lane affected by U-turns. A video camera recorded the entire study for later reference.

3.1.2.1. Saturation Flow Study

The observer measured the headway of each vehicle individually using a Jamar TDC-8 electronic counter. This counter records headways to a 15.6-millisecond precision. Of course, the fact that this counter was being operated manually means that human reaction time error was introduced. Each headway measurement cannot be considered accurate to the millisecond level. However, the same observer conducted the saturation headway study for all intersections and the effect of the human error should have balanced itself out. The importance of this amount of precision is that the headway measurements were not placed in bins or rounded to the nearest second.

The headways were measured for all vehicles in the queue, but only headways for vehicles in the fifth position or greater were used in saturation flow analysis to eliminate any effect of start-up lost time on the saturation flow estimates. The observer only measured headways of vehicles that were stopped in the queue when the light turned green. As the front axle of each vehicle crossed the stop bar, the observer pushed a button which assigned a timestamp to that vehicle. On a sheet, the observer marked which vehicles in that queue made U-turns. Appendix B contains a sample data collection form. Headways were only recorded for vehicles that were stopped in the queue when the light turned green.

The more traditional method of saturation flow measurement suggests that a maximum of ten vehicles be used. The reason behind this is that if only one person is conducting the study, it is unlikely that they would be able to accurately count over ten queued vehicles when the light turned green. For this project, there was no such maximum observed due to the good communication between observers. The observer conducting the delay study would indicate at each cycle how many vehicles were stopped in the queue when the light turned green.

3.1.2.3. Volume Count

Volume data were collected for the left turn lane of interest and the conflicting right turn (RTOR/RTOA) movement. U-turns were counted as left turns in the volume data. Conflicting right turns were only counted when the left turn movement had the green. This gave the indication of what volume of right-turning vehicles are normally competing with U-turns. The observer also counted heavy vehicles and pedestrians to ensure any particular study site was not abnormally saturated with either count compared to the rest of the sites.

3.1.2.4. Delay Study

The team conducted a stopped delay study for the left turn lane of interest. This study was conducted using the delay function of the Jamar TDC-8 counter set to a 15-second interval. Typical delay intervals range from 10 to 20 seconds. Some engineers prefer to use intervals that are not evenly divisible into the signal cycle length to avoid biased delay estimates. However, any error this may introduce is negligible and current practice is to use any convenient interval [12].

3.2. Safety Impacts of U-Turns

U-turns have been thought to be a safety concern due to their movement, which can be difficult to anticipate. They could cause conflicts with vehicles turning right from the cross street as well as conflicts with vehicles in the main road left turn queue. Through a study of collision history, I examined the safety impact of U-turns on an intersection. This process involved the selection of appropriate study sites and the compilation of data on physical characteristics, traffic volume, and collision history. It should be understood that the term “site” refers to one approach at an intersection, not the intersection as a whole.

3.2.1. Site Selection

The set of intersections used for the safety study was a compilation of two groups of sites. The first group contained sites that were randomly chosen. The second group contained U-turn “problem sites” that were recommended based on high volumes of U-turns or a history of U-turn collisions. These two groups provided a list of sites that were intentionally biased to predict higher U-turn problems than would be predicted with a completely random set of sites. This gave a very conservative estimate of the safety impact of U-turns at signalized intersections.

Eligibility Criteria

To be eligible as a study site, each intersection had to meet the following criteria:

1. *Signalized Intersection*. The scope of this project included only signalized intersections. Permitted and protected left turn signal types were included in the study.

2. *Presence of Median*. Even though U-turns may occur at intersections that have no median, I only looked at sites with medians at the intersections. However, no restriction was placed on the length or width of the median.

3. *Two Lanes Receiving*. I only included sites that had two lanes receiving the U-turns. This reason stems from the contracted project's goal of comparing four-lane divided highways to five-lane undivided highways. This criterion excluded sites that had three through lanes or a third lane for buses or exclusive right turns, but did not exclude sites with U-turn "bulb-outs" or wide shoulders.

No sites were chosen that had a signed prohibition of U-turns at the approach. I wanted a safety analysis that would examine U-turn collisions under normal conditions. U-turns made illegally cannot be expected by other drivers. The impact of such U-turns would be difficult to predict. See Appendix J for a list of selected safety study sites. Table 3.2 provides a summary of the number of sites selected for the safety study.

Random Sites

The group of random sites was selected with the help of a partner project that focused on comparing cross-sections on midblock segments [3]. The data collection for this partner project involved the random selection of highway segments from the NCDOT inventory. Any signalized intersection bordering a selected segment was examined for eligibility. The 54 eligible intersections bounding these segments became the randomly selected sites for the safety study.

Recommended Sites

To select sites with high U-turn volumes or a history of U-turn collisions, I contacted 120 city and state transportation engineers across North Carolina. I asked each person to give me a list of signalized intersections in their area that had high percentages of U-turns. Twenty-three people responded giving me a list of 65 recommended sites. After all sites were visited to determine eligibility, 41 sites were disqualified, leaving 24 eligible sites. The most common reasons for disqualification were an improper number of lanes receiving U-turns (three lanes receiving being the most common) and the intersection being unsignalized. Four of the sites recommended for the safety study were also eligible to be used in the operational study.

Table 3.2. Sample Size for Safety Study

	Number of Sites
Random	54
Recommended	24
Total	**78**

3.2.2. Collection of Physical Data

In order to assemble factors for the safety study, it was necessary to collect data on the physical characteristics of each intersection and surrounding area. Figure 3.2 shows the form used to collect data for both the intersection geometry and the roadway segment leading to the intersection approach of interest. This segment was defined as beginning at the last median break and ending at the intersection. Drivers wishing to make a U-turn would be proceeding down this segment before making a U-turn at the intersection. The following data were collected for each site:

1. *Main street and cross street names.* This includes not only the local name of the street but also any state or U.S. route numbers that applied. This information was used later to locate the intersection for collision data collection.

2. *Intersection characteristics.* I collected data on signal phasing, lane widths, number of left turn lanes, median width, and number of lanes receiving U-turns.

3. *Segment information.* I collected data on segment length, speed limit, number of access points, and approximate land use percentages.

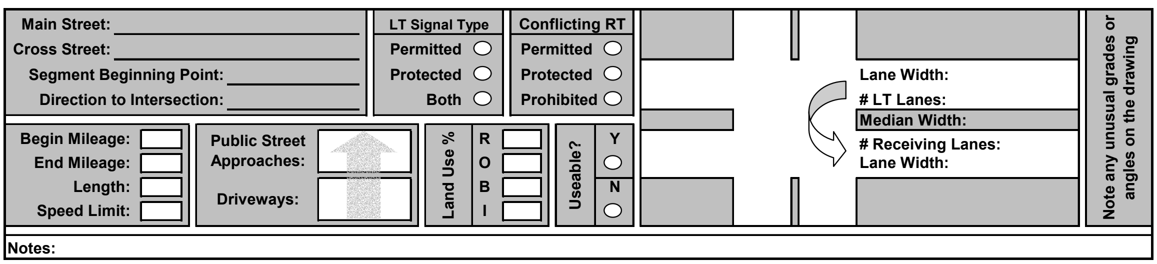


Figure 3.2. Physical data collection form

For each site in the group of recommended sites, I collected all applicable data during a site visit. The data for the randomly selected sites were collected during the data collection trips of the previously-mentioned partner project [3]. This collaboration resulted in efficient use of resources and sped up the data collection process for this project.

3.2.3. Collection of Collision Data

Collision data used in this project were taken from records of police-reported collisions. These data were procured from the NCDOT collision database using the procedure detailed below.

Time Period of Collision Data

Collision data were collected from October 1, 1999 to October 1, 2002. The project team determined that this recent 3-year period was short enough to avoid the effects of development and geometry changes on the data and long enough to provide a reliable amount of collision data.

Collection of Collision Data

The listing of all collisions at a particular intersection was procured using the Traffic Engineering Accident Analysis System (TEAAS) software from the Traffic Safety Systems Management Unit (TSSMU) at the NCDOT. The TEAAS software requires combinations of two road names to produce a listing of collisions. It produces a list of all collisions at the intersection during the specified time period, including information such as collision date, time, and ID number.

Once a list of collisions for a site was assembled, the ID numbers for each collision were entered into the NC DMV Crash Reporting System webpage to obtain a graphic file of each of the official crash reports. In order to determine the number of U-turn collisions at each site, it was necessary to visually inspect every crash report for the time period chosen. The current North Carolina collision report form (DMV 349) does not include a checkbox or code to denote if the collision involved a U-turn movement. The only method available was to inspect the collision diagram and police officer narrative to determine if a U-turn was involved. Figure 3.3 shows a diagram and narrative indicating that the collision involved a U-turning vehicle and a right-turning vehicle.

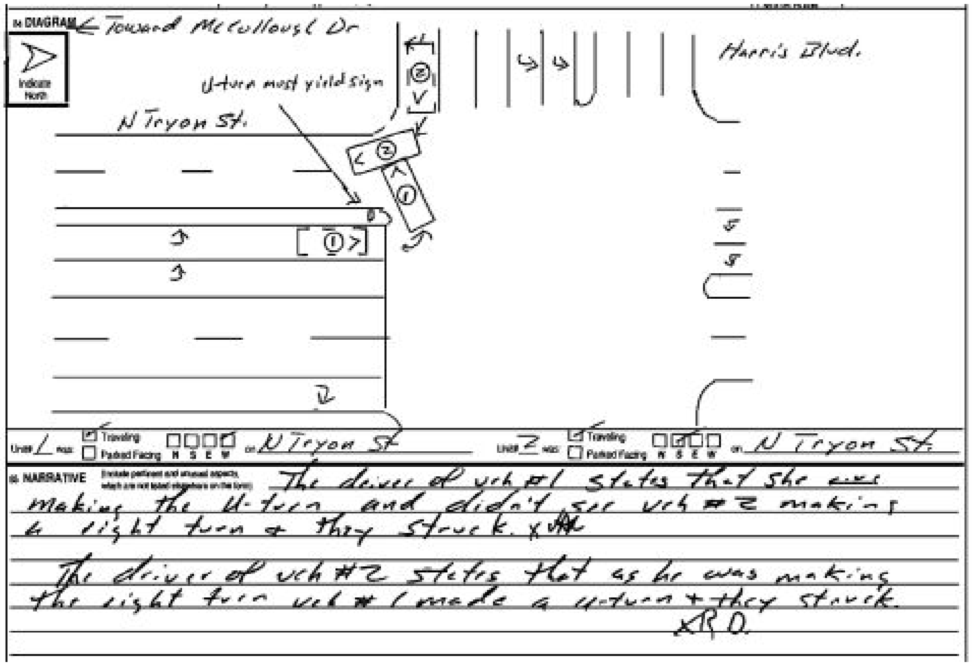


Figure 3.3. Sample Collision Report with U-Turn Collision

3.2.4. Collection of Traffic Volume Data

For each site in the study, I obtained information on main road Average Daily Traffic (ADT). These data were available from the Geographic Information Systems webpage of the NCDOT. Although these volume numbers were indicative of the level of traffic at the intersection, I desired more specific information on the turning movements. I was able to obtain turning movement counts for 29 of the 77 sites. These counts were only available for sites in the cities of Raleigh, Charlotte, and Wilmington due to the fact that these counts are not regularly performed outside of large urban areas.

3.1.2.2. Conflict Study

In order to supplement the intersection safety data concerning U-turns, the team conducted a conflict study simultaneously with the operational field studies at the 14 operational study sites. Conflicts of interest were as follows:

- Left turn same direction conflict (rear-end) between U-turning vehicle and left-turning vehicle
- Conflict between U-turning vehicle and right-turning vehicle moving either under protected right turn or permitted RTOR
- Any other conflicts that were observed to involve a U-turning vehicle

To maintain consistency, all conflict studies were conducted by the same observer. Appendix C contains a sample data collection form. Also, having the study on tape allowed for the opportunity to reexamine possible conflicts. The observer used the description detailed in the ITE *Manual of Transportation Engineering Studies* to determine whether a conflict had occurred.

> "…traffic conflicts are interactions between two or more vehicles or road users when one or more vehicles or road users take evasive action, such as braking or weaving, to avoid a collision….Observers use brake lights, squealing tires, or vehicle front ends that dip or dive as indications that braking occurred and a conflict was possible. A collision or near miss during which no evasive actions were observed also counts as a traffic conflict." [12]

4. Results

4.1. Operational Impacts of U-turns

Fourteen sites were used in the operational study (see Table 3.1). This group of sites was composed of signalized intersections with exclusive left turn lanes and protected left turn phasing. Each site was studied an average of 7.5 hours with an average of 400 eligible queues observed per site. The average U-turn percentages at the study sites covered a wide array, ranging from 6 to 81 percent. A list of these sites and pertinent data on their characteristics is available in Appendix E.

The data provided by these sites proved sufficient for the purpose of determining operational impacts of U-turns. The data were of the quality desired, but a few modifications had to be made in order to use the full set of data. The most notable problem occurred at two study sites. These sites had such a constant stream of U-turns that only a few queues containing no U-turns were observed. Queues containing no U-turns were important because they provided a value for saturation flow rate that was unaffected by U-turns. The modifications to the data from these sites are described in section 4.1.1.

I calculated average U-turn percentage in the left turn queue and the saturation flow reduction due to U-turns for each site. These values were later used in multivariate regression analysis to predict an adjustment factor due to U-turns. Section 4.1.1 details the process I used to calculate these two values used in the regression. Table 4.1 shows a summary of the values for each site.

One note should be made about queue eligibility in these calculations. Although I observed queues of many different lengths, I only considered a queue eligible for calculations if it contained five or more vehicles. Since I wanted to measure only the effect of U-turns on saturation flow rate, I did not use headway data from vehicles in the first through fourth positions. This was to avoid the influence of start-up lost time on the calculations. This five-vehicle minimum is the only requirement for the eligibility referred to in the following paragraphs.

Table 4.1. Summary of U-Turn Percentages and Reduction Factors by Site

Site	Comparison Sat Flow (vph)	Average Observed Sat Flow (vph)	Saturation Flow Adjustment Factor	Average Percentage U-turns	Conflicting Right Turn Overlap?
202	1759	1740	0.99	16	no
203	1791	1762	0.98	6	yes
204	1597	1613	1.01	14	no
205	2070	1731	0.84	41	yes
206*	1650	1370	0.83	81	no
207	1859	1654	0.89	32	yes
210	1653	1551	0.94	15	yes
211	1665	1558	0.94	28	yes
212	1843	1764	0.96	27	no
213	1739	1624	0.93	34	no
215	1722	1498	0.87	52	yes
216	1821	1727	0.95	32	no
217**	1604	1552	0.97	50	no
218	1763	1669	0.95	13	yes

* Comparison sat flow is averaged from three similar sites because no queues without U-turns were observed.

** Comparison sat flow is calculated from vehicles with no U-turns within four positions.

4.1.1. Calculation of Regression Variables

Average U-turn Percentage

The U-turn percentage for each site was calculated by averaging the U-turn percentages of all observed eligible left turn queues. The U-turn percentage for a particular queue was measured by dividing the number of U-turning vehicles in the queue by the total number of vehicles, thereby calculating percentage over the whole queue. This differs from the saturation flow measurements which only use vehicles in position five or greater. This point is discussed below and in section 4.1.5. The U-turn percentages in Table 4.1 were calculated by averaging the U-turn percentages by queue for each site.

Saturation Flow Reduction Factor

The saturation flow reduction factor due to U-turns was calculated for each site by dividing the average saturation flow rate of all observed vehicles at the site by the comparison saturation flow rate. The average observed saturation flow rate is calculated using headways of all observed eligible vehicles, both those affected by U-turns and those unaffected by U-turns. The comparison saturation flow rate is the average rate of all eligible vehicles that had no U-turning vehicles preceding them in the queue. This comparison rate is understood to be already affected by all other adjustment factors (i.e., lane width, grade, intersection angle). Since the only difference in these two saturation flow rates is the presence of U-turning vehicles, all other influencing variables such as lane width, grade, and intersection angle are factored out. This produces an adjustment factor that specifically shows the effect of U-turns on saturation flow in exclusive left turn lanes.

All 14 study sites were used in the saturation flow reduction analysis, but some modifications were made to accommodate two sites. The comparison saturation flow rate for site 206 was not able to be measured since there were no queues without U-turns. The comparison rate for this site was instead taken from an average of three other sites in the study that had similar characteristics. While this is not the preferred method to estimate saturation flow reduction, the average of 81% U-turns per queue at site 206 provided valuable insight to the operational effect of very high U-turn percentages. The comparison saturation flow rate for site 217 was calculated using headways of vehicles with no U-turns within four positions instead of vehicles with no U-turns preceding. This was due to a small sample size of vehicles with no U-turns preceding them. This method of determining comparison saturation flow rate is valid under the assumption that U-turning vehicles do not significantly affect vehicles that come four queue positions later.

The observed saturation flow was calculated using the headway data of all vehicles after the fourth position. I averaged the headways and converted the value to saturation flow rate in vehicles per hour. An example calculation is presented below.

Example Calculation

The following calculation is a demonstration of the process I conducted to obtain data points for the regression analysis. The queue in Table 4.2 is similar to queues obtained during field headway measurements.

Table 4.2. Example Queue for Saturation Flow Calculation

Position	Status	Headway from Preceding Vehicle (sec)
1	Left turn	-
2	Left turn	2.25
3	Left turn	2.06
4	Left turn	2.14
5	U-turn	2.36
6	U-turn	2.35
7	Left turn	2.25
8	Left turn	2.07

Given the vehicle movement and headway data in Table 4.2 for a queue, the U-turn percentage would be calculated as such:

U-turn Percentage = (2 U-turns) / (8 total vehicles) = **25% U-turns**

The average headway would be calculated using only vehicles five through 8:

Average Observed Headway = average(2.36, 2.35, 2.25, 2.07) = **2.25 sec/veh**

The saturation flow is determined by the average headway:

$$\text{Observed Saturation Flow} = \frac{3600 \text{ sec}/hr}{2.25 \text{ sec}/veh} = \mathbf{1600\ veh/hr}$$

It should be noted that the U-turn percentage for a queue was calculated over the whole queue, whereas saturation flow was measured starting with the vehicle in the fifth position. The reason that U-turn percentage was not limited to the fifth position minimum is for model usability purposes. Users of this model will not be able to estimate the percentage of U-turning vehicles that will be above the fifth queue position, but rather they will have an estimate of the percentage of U-turning vehicles they expect at the site in general. I desired that this model should reflect that input. One objection to the inequality in the criteria for measuring U-turn percentage and saturation flow can be seen in the following scenario.

Suppose the following left turn queue is observed:

Position	1	2	3	4	5	6	7	8	9	10
Status	U-turn	U-turn	U-turn	U-turn	Left	Left	Left	U-turn	Left	Left

According to the above procedure, the U-turn percentage would be calculated as 50%, using all vehicles in the queue. However, the saturation flow would be calculated using only positions 5 through 10, which contain only one out of six, or 17% U-turns. In this case, the reported saturation flow would be calculated with 17% U-turns, but reported as having been calculated for 50% U-turns. This issue is addressed in the section below on hypothetical queues.

4.1.2. Factors Affecting Saturation Flow Reduction

Although the saturation flow adjustment factors in Table 4.1 seem to vary between sites based mainly on U-turn percentage, I wanted to know if any intersection characteristics such as median width or conflicting right turn type had a significant role in saturation flow reduction in conjunction with U-turn percentage. To narrow it down to a particular characteristic, I compared only those queues from each site with an equal amount of U-turn percentage. Comparing queues in this manner factored out the effect of U-turn percentage to let me examine the effect of other intersection characteristics. In Table 4.3, I examined two levels of U-turn percentage: 20% and 50%. These two levels of U-turn percentage give a good indication of the effect of site characteristics at low and moderately high percentages of U-turns. There were not enough data to evaluate these effects on queues with very high U-turn percentages.

Table 4.3. Significance of Site Characteristics on Saturation Flow Reduction

Characteristic	Effect on Queues with 20% U-turns		Effect on Queues with 50% U-turns		Statistical Test
	*Significant?**	*Description*	*Significant?**	*Description*	
Median Width	NO	Regression line has insignificant slope	NO	Regression line has insignificant slope	Regression Analysis
Total Receiving Width**	NO	Regression line has insignificant slope	NO	Regression line has insignificant slope	Regression Analysis
Average Conflicting Right Turn Volume	NO	Regression line has insignificant slope	NO	Regression line has insignificant slope	Regression Analysis
Presence of Protected Right Turn Overlap	**YES**	Sites with overlap have lower capacity than site w/o overlap	**YES**	Sites with overlap have lower capacity than site w/o overlap	T-test
Number of Receiving Lanes	NO	No significant difference in group means	NO	No significant difference in group means	T-test
Number of Left turn Lanes	**YES**	Sites with 2 LT lanes have lower capacity than single LT lane sites	**YES**	Sites with 2 LT lanes have lower capacity than single LT lane sites	T-test

* All statistical tests in Table 4.3 were performed at 90% confidence.
**Total receiving width at first appeared significant due to one extreme value. When the value was removed, remaining data had no cohesiveness. The extreme value was a site with wide receiving width due to an extra-flared right turn.

Appendices G and H display the plots and statistical analyses of each data set. From the analysis of the data, it appears that the only site characteristics that affect saturation flow are the presence of protected right turn overlap and the number of left turn lanes.

Protected right turn overlap conflicting with the U-turn movement affected queues with both low and moderately high degrees of U-turn percentage. The analysis showed that sites with overlap had a significantly lower saturation flow than sites without protected right turn overlap.

The other significant factor was the number of left turn lanes. Sites with a double left turn lane experienced reduced saturation flow when compared to single left turn lanes, for both low and moderately high U-turn percentages. This could be due to the fact that many intersections with double left turn lanes also have a protected right turn overlap, which showed to be significant in Table 4.3. Six of the eight sites with double left turn lanes had protected right turn overlap. Only two of the seven sites with single left turn lanes had right turn overlap. This is an indication of possible correlation between these two factors, but there were not sufficient data in this study to clearly separate these effects. Due to the possibility of correlation, I did not include number of left turn lanes as a factor in the multivariate regression.

The conflicting right turn volume was not significant in this analysis. This may be confusing since the type of conflicting right turn was significant. This volume insignificance is not due to a limited range of volume, since the volumes ranged from 4 to 149 vehicles per hour. The analysis suggests that the real effect comes from the type of conflicting right turn. It is possible that conflicting right turns that have a protected overlap could have a strong influence even if there is low turning volume.

4.1.3. Saturation Flow Adjustment Factor Determination by Regression

The saturation flow adjustment is based on a multivariate linear regression, involving average U-turn percentage and the interaction of U-turn percentage and the presence of protected right turn overlap from the cross street. This adjustment factor should be used for exclusive left turn lanes with protected phasing. The regression equation is as follows:

$$f_{uturn} = 1.0 - 0.0018*UTURN - 0.0015*UTURN*OVERLAP$$

where:

f_{uturn} = saturation flow adjustment factor for an exclusive left turn lane with protected phasing

UTURN = average U-turn percentage in the exclusive left turn lane (or inside turn lane if double left turn lanes)

OVERLAP = yes/no variable, 1 if conflicting right turn has protected overlap, 0 if no protected right turn overlap

The regression line has an R^2 of 0.79 with an adjusted R^2 of 0.75. Both coefficients are significant at a 99% confidence level. See Appendix F for a summary of the regression output. The actual regression intercept was 1.0097 and I did not force this to 1.0. The Excel statistical tools do not allow intercept forcing for multivariate regression and the SAS software package produced unreliable values of R^2 when the intercept was forced to 1.0. For the purpose of this adjustment factor, I determined that the intercept should be listed as 1.0 in the equation under the assumption that 0.0097 would be insignificant in capacity adjustment. An intercept of 1.0 would be more intuitively correct for the situation since a zero U-turn percentage should cause the U-turn adjustment factor to be 1.0 and have no effect on saturation flow.

The regression analysis that produced the above equation used each site as an individual data point. Figure 4.1 shows how the results would appear if plotted by individual queue. Two lines run through the scatter plot (may appear to be one line). One line is the predicted values from the above regression equation. The other line is the linear trend line fitted by Excel to the scatter plot. The fact that the two lines are almost identical shows that the site-based regression equation above would give the same results if based on individual queues.

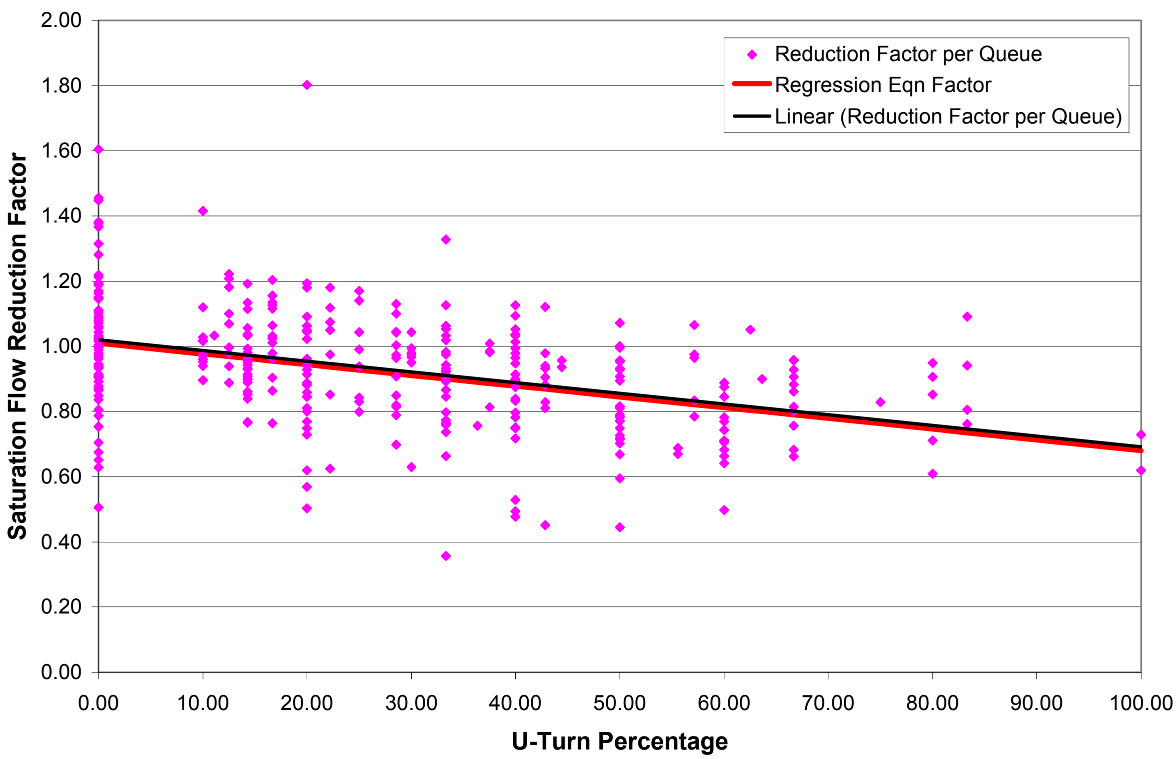


Figure 4.1. Plot of Saturation Flow Reduction Factor by Individual Queue

I initially performed the regression as a single variable regression using only U-turn percentage as the independent variable. While this analysis was reasonably good with an R^2 value of 0.55, I wanted to try a multivariate regression to produce a better fit. The intersection characteristic that proved the most significant in section 4.1.2 was the presence of protected right turn overlap.

Including an overlap factor by itself in the regression, however, would violate the underlying assumption of this analysis. The assumption is that the U-turn adjustment factor should only have an effect when some amount of U-turn percentage is involved. If the overlap factor were included by itself, there would be some value for f_{uturn} less than 1.0, even when there was a zero U-turn percentage. Upon further analysis, the interaction between U-turn percentage and overlap proved to be significant, so I included that interaction in the equation and provided a much better goodness-of-fit as well as a more useful model overall. With the model in this form, a zero U-turn percentage will produce a U-turn adjustment factor of 1.0.

The reduction factor f_{uturn} should be used as an adjustment to saturation flow rate for an exclusive left turn lane. In the case of double left turn lanes, this factor only applies to the inside left turn lane, since that is the only lane affected by U-turns. To analyze the left turn lane group as a whole, the analyst will need to calculate a weighted average adjustment factor using the procedure in section 4.1.7. The f_{uturn} factor is similar to other adjustment factors found in the Highway Capacity Manual, including adjustments for heavy vehicles and lane utilization. Utilization of this U-turn adjustment factor will give a more accurate projection of the operation of a signalized intersection on a divided facility.

The two methods for saturation flow reductions located in current literature included a regression equation from Thakkar and saturation factor recommendations from Adams [7,8]. In Figure 4.2, I compared my regression results to the results given by the other two methods when used on my dataset. The Adams saturation flow reduction factors were a rough estimate based on tiers of U-turn percentage, thus producing a step-like function. The Thakkar equation uses input variables of U-turn percentage and RTOA (right turn-on-arrow, the volume of traffic that turned right during the U-turn phase) to determine saturation flow reduction. I used the average RTOA volume from the 14 sites for the RTOA variable. For my own analysis, I plotted lines of the saturation flow reduction factor with and without protected right turn overlap.

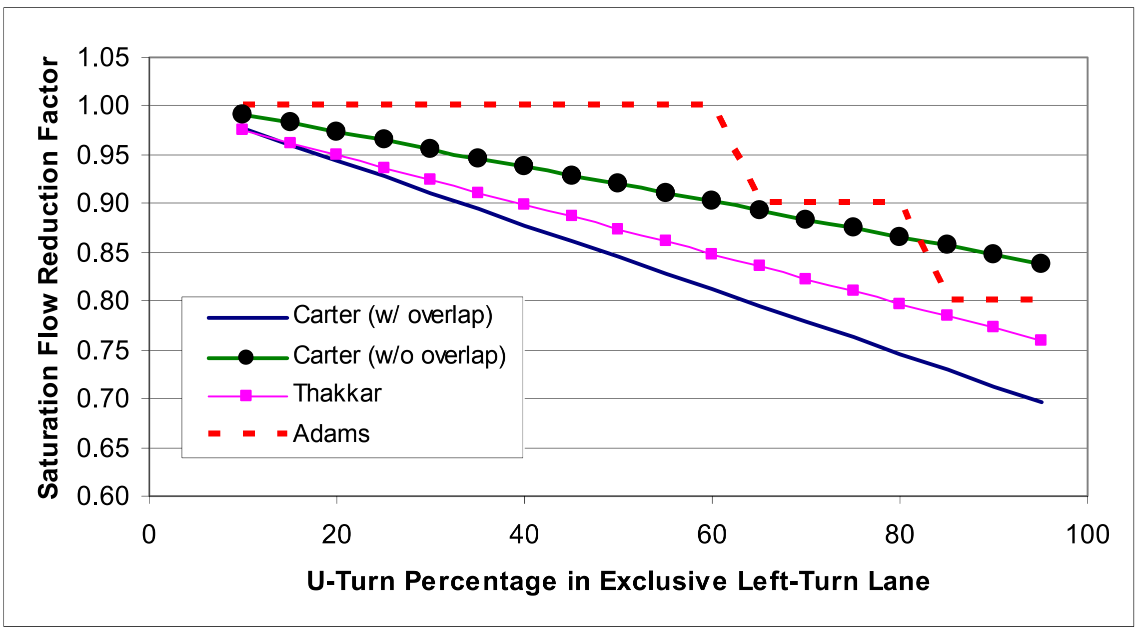


Figure 4.2 Comparison of Saturation Flow Reduction Studies

The overall trends of the three methods are similar, with Thakkar showing the closest results to my own. Her saturation flow reduction equation fell almost directly in between my two lines, though her results were closer to my prediction for sites with protected right turn overlap. This is to be expected since there was protected overlap at the one intersection that Thakkar used. Adams did not note whether his sites had protected right turn overlap from the cross street.

There may be instances when traffic engineers and planners would not have an estimate of U-turn percentage that is more precise than the nearest 10% or would like to know the sensitivity range of this capacity reduction. Table 4.4 shows the predicted saturation flow reduction factors for common U-turn percentages, for intersections with protected right turn overlap and those without.

Table 4.4. Saturation Flow Reduction Factors for U-Turn Percentages

Percentage U-turns	Saturation Flow Reduction Factor with overlap	Saturation Flow Reduction Factor without overlap
10	0.98	0.99
20	0.94	0.97
30	0.91	0.96
40	0.88	0.94
50	0.84	0.92
60	0.81	0.90
70	0.78	0.88
80	0.75	0.87
90	0.71	0.85
100	0.68	0.83

4.1.4. Individual Driver Behavior

The methodology used in this project to collect saturation flow data involved precise measurements of individual vehicle headways (see section 3.1.2.1, "Saturation Flow Study"). In addition to that, each vehicle was recorded as having made a left turn or a U-turn. This level of detail provided the opportunity to measure the behavior of individual vehicles and produced results that would prove useful in micro-simulation scenarios.

During field data collection, I observed that a vehicle's headway was affected not only by the type of turn it executed, but also the movements made by vehicles that preceded it in the queue. For example, a vehicle following three consecutive U-turns was generally slowed much more than a vehicle following a single U-turn. To examine the behavior of individual vehicles under different circumstances, I created 16 "micro-categories" into which all vehicles are classified, based on whether the vehicle made a left turn or a U-turn and the vehicle's proximity to U-turning vehicles. Table 4.5 lists the category descriptions. The right column of Table 4.5 is provided as a quick visualization of how the queue would appear in traffic situations, with the front of the queue being on the left-hand side.

Table 4.5. Description of Vehicle Micro-Categories

Vehicle Category	Vehicle Movement	Proximity to U-turns	Illustration*
L1	Left turn	No U-turn preceding it in queue	ooooooooL
L2	Left turn	Directly behind single U-turn	oooUL
L3	Left turn	Directly behind 2 consecutive U-turns	oooUUL
L4	Left turn	Directly behind 3 consecutive U-turns	oooUUUL
L5	Left turn	2 positions behind any U-turn	oooUoL
L6	Left turn	3 positions behind any U-turn	oooUooL
L7	Left turn	4 positions behind any U-turn	oooUoooL
L8	Left turn	No U-turn within 4 positions	oUooooL
U1	U-turn	No U-turn preceding it in queue	ooooooooU
U2	U-turn	Directly behind single U-turn	ooooUU
U3	U-turn	Directly behind 2 consecutive U-turns	oooUUU
U4	U-turn	Directly behind 3 consecutive U-turns	oooUUUU
U5	U-turn	2 positions behind any U-turn	ooooUoU
U6	U-turn	3 positions behind any U-turn	ooooUooU
U7	U-turn	4 positions behind any U-turn	oooUoooU
U8	U-turn	No U within 4 positions	oUooooU

* o = vehicle in left turn lane; U = U-turning vehicle; L = left-turning vehicle

Table 4.6 presents a list of proportions for each vehicle category at Site 207 (Tryon and Harris). This site had two lanes receiving and a protected right turn overlap. Each headway value represents an average of the headways of all vehicles that fall into that category. The proportion values in the right-hand column of this table compare the headway of a particular category to the "comparison" headway – the headway of a vehicle completely unaffected by U-turns. This value is taken from the category shown in bold in the first row of Table 4.6. At Site 207 for example, all category headways are compared to the comparison headway of 1.94 seconds. The proportion is calculated as follows:

$$\text{Proportion of Comparison Headway} = \frac{\textit{Category Headway}}{\textit{Comparison Headway}}$$

In this method, a proportion greater than 1.0 would indicate that the particular category has a larger headway than a vehicle not affected by U-turns. If the value is 1.13, the category vehicle will take 13% longer than “normal” to complete its passage through the intersection.

Table 4.6. Proportions of Comparison Headway by Vehicle Category for Site 207

Vehicle Category*	Illustration	Headway (sec)	Proportion of Comparison Headway	Sample Size
L1	**ooooooooL**	**1.94**	**1.00**	**78**
L2	oooUL	2.08	1.07	89
L3	oooUUL	2.72	1.40	40
L4	oooUUUL	2.66	1.37	9
L5	oooUoL	2.17	1.12	88
L6	oooUooL	1.92	0.99	58
L7	oooUoooL	1.94	1.00	39
L8	oUooooL	1.92	0.99	109
U1	ooooooooU	2.12	1.09	44
U2	ooooUU	2.23	1.15	60
U3	oooUUU	2.48	1.28	18
U4	oooUUUU	3.43	1.77	8
U5	ooooUoU	2.47	1.27	43
U6	ooooUooU	2.19	1.13	34
U7	oooUoooU	2.13	1.10	13
U8	oUooooU	2.16	1.12	59

* See Table 4.5 for category descriptions

As can be seen clearly in Table 4.6, the proportions increase for categories involving consecutive U-turns. The highest proportion is 1.77 times the comparison headway and is for category U4, which involves four consecutive U-turns. Other trends are not as clear, but the general tendency is for the headway of a vehicle to increase when the vehicle has more involvement with U-turns. I did not analyze the effects of intersection characteristics, such as median width, on the headways of individual vehicles.

Table 4.6 presented findings for one particular site; however, the complete results of this analysis need to involve all sites to be as comprehensive as possible. In order to concisely present the results in Table 4.7, I divided the sites into four categories based on the type of conflicting right turn and the number of lanes receiving.

Table 4.7. Proportions of Comparison Headway by Vehicle and Intersection Category for All Sites

	Intersection Characteristics			
Vehicle Category*	Permitted Conflicting Right Turn, 2 Lanes Receiving	Permitted Conflicting Right Turn, 3 Lanes Receiving	Protected Conflicting Right Turn, 2 Lanes Receiving	Protected Conflicting Right Turn, 3 Lanes Receiving
L1	1.00	1.00	1.00	1.00
L2	0.97	1.00	1.12	1.09
L3	1.09	1.06	1.47	1.29
L4	1.19	1.12	1.33	1.29
L5	0.99	0.99	1.09	1.09
L6	1.00	1.04	1.02	1.02
L7	0.94	0.99	1.03	1.00
L8	0.99	0.98	1.00	0.97
U1	1.09	1.14	1.15	1.32
U2	1.05	1.16	1.19	1.11
U3	1.06	1.19	1.55	1.18
U4	No data	1.20	1.26	1.29
U5	1.15	1.17	1.31	1.16
U6	1.05	1.13	1.16	1.13
U7	1.07	1.06	1.14	1.07
U8	1.11	1.07	1.15	1.14

* See Table 4.5 for category descriptions

The values in Table 4.7 are calculated as the proportions of the average headway of each category to the comparison headway for that category. This follows the same procedure described for Table 4.6.

Pursuant to the methodology described in section 4.1.1, I did not use headway measurements involving the first four vehicles in the queue. While these vehicles do have saturation flow headways, general practice assumes that the first three vehicles are affected by start-up lost time. It is worthwhile to mention that these vehicles' headways can be affected not only by start-up lost time but also by their proximity to U-turning vehicles, the same as vehicles farther back in the queue, such that the headway calculations would be as follows:

Vehicle	Headway Calculation
Vehicle in position 1-4 (affected by lost time)	Comparison headway + Uturn effect + startup lost time
Vehicle in position 5 or greater (not affected by lost time)	Comparison headway + Uturn effect

To complete the headway calculation for lost time vehicles, the analyst would determine the U-turn effect according to Table 4.7 and then decide the amount of startup lost time to assume for each vehicle. For example, given the typical value of two seconds for total start-up lost time, one may assume that 1.2 seconds of that lost time affects the first vehicle, 0.6 seconds affects the second vehicle, and 0.2 seconds affects the third vehicle, since start-up lost time has been observed to have a declining effect after the first vehicle in line.

The data provided in Table 4.7 would be of great use in micro-simulation. Some software packages such as SimTraffic and Vissim already have some U-turn modeling capabilities. The results from this research would enable these programs to replace their current parameters for U-turning vehicles with numbers that are more refined and validated.

4.1.5. Hypothetical Queues

As previously mentioned, the headway data gathered for this project contain a high degree of detail pertaining to individual vehicles. I combined precise measurements of vehicle headways with a description of the turn executed (left turn or U-turn) to create 16 micro-categories (see Table 4.5). This knowledge of the average headway associated with each category gave me the opportunity to create "hypothetical queues".

Hypothetical queues use the micro-categorical data to give an estimate of the average headway for a particular left turn queue given a distribution of U-turns. For example, an analyst may specify a queue to be made up of 10 vehicles making left turns and U-turns, with a U-turn percentage of 30%. Although there are many possible combinations of 3 U-turns and 7 left turns, the queue could be set up as follows:

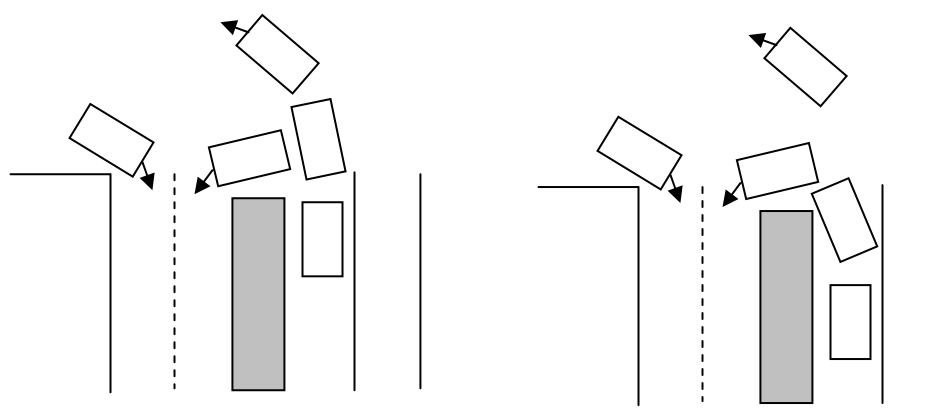
Queue Position	1	2	3	4	5	6	7	8	9	10
Movement (left or U-turn)	L	L	L	**U**	L	L	**U**	**U**	L	L

Consider that this queue occurred at Site 207. We can then use the headway data provided in Table 4.6 to estimate what this hypothetical queue's average headway would be. Each vehicle in the queue falls into one of the 16 categories. Following the process in section 4.1.1, headways would be determined only for vehicles in position 5 or greater. Vehicle 5 falls into the category of a left-turning vehicle directly behind a single U-turning vehicle (category L2) and would be expected to have a headway of 2.08 seconds. Vehicle 6 would be in category L5 with a headway of 2.17 seconds, and so on. When all headways are filled in, we get the following queue:

Queue Position	1	2	3	4	5	6	7	8	9	10	Avg
Movement (left or U-turn)	L	L	L	**U**	L	L	**U**	**U**	L	L	
Headway (seconds)	-	-	-	-	2.08	2.17	2.19	2.23	2.72	2.17	**2.26**

One advantage of this hypothetical queue analysis is the ability to set up a "best" case and "worst" case scenario for a particular U-turn percentage. There are many ways that three U-turning vehicles can be positioned in a 10-vehicle queue. Some arrangements can result in a larger average headway than others.

For example, I found that consecutive U-turns generate high headways because of the compounding effect of delay involved with the maneuver. If a U-turning vehicle stops to yield to a right-turning vehicle, left-turning vehicles may still be able to proceed around the U-turning vehicle and complete their left turn. However, if two consecutive U-turning vehicles are stopped to yield to a right turn, no other vehicles in the left turn queue can pass until the U-turns clear (see Figure 4.3). This delay causes headway measurements to increase, thereby decreasing the saturation flow rate.



Single U-turn paused; left turning vehicles still able to pass

Two consecutive U-turns paused; left turning vehicles unable to pass

Figure 4.3. Illustration of the Effect of Consecutive U-Turns

The "best" case would produce the smallest average headway and generally involves U-turns that are spaced evenly with most U-turns in the first four positions so as to affect only slightly the headway measurements of positions five and greater. The “worst” case would produce the largest average headway and generally involves consecutive U-turns arranged in the middle of the queue.

The objection raised in section 4.1.1 pertained to the possibility of a discrepancy in the measuring of U-turn percentage and the determination of saturation flow. The objection noted that most of the U-turns for a particular queue could fall in the first four positions. Since saturation flow is measured using only vehicles in positions five or greater, the apparent discrepancy is that the queue is reported to have one U-turn percentage while saturation flow is measured for a part of the queue that has a very different U-turn percentage. As defined above in the introduction of hypothetical queues, this situation would be referred to as a “best” case since the U-turns hardly affect the measured saturation flow rate.

While this “best” case scenario may occur from time to time, there would also be “worst” cases, where all the U-turns are crowded into the latter part of the queue. Indeed, these two scenarios did occur in the dataset, as well as many queues that would classify somewhere in between. However, the large size of the dataset served to average out these cases to an average scenario, the results of which were displayed in Figure 4.2. To serve as a visual representation of this averaging process, Figures 4.4 and 4.5 compare the average observed headway to the “best” and “worst” cases for both low and moderately high U-turn percentages. In all cases but one, the average observed headway fell between the “best” and “worst” cases. For the illustration simplicity, the figures group the 14 study sites into four categories similar to those in Table 4.7. The data behind these graphs can be found in Appendix I.

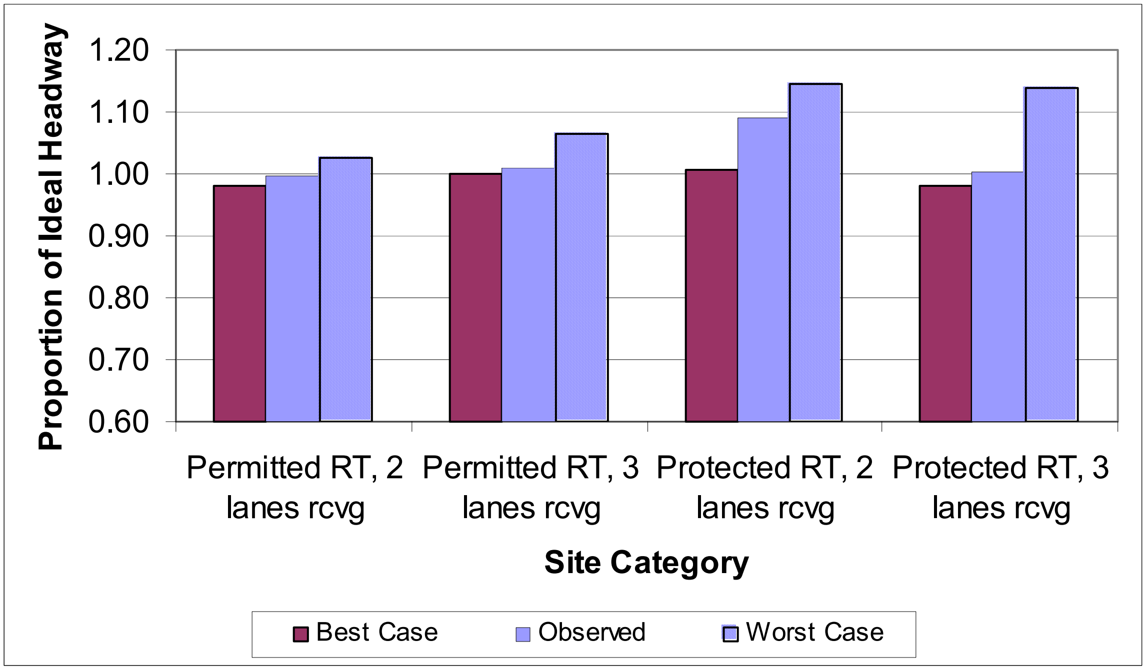


Figure 4.4. Comparison of Average Observed Headway to Best and Worst Case Scenarios for Queues with 20% U-Turns

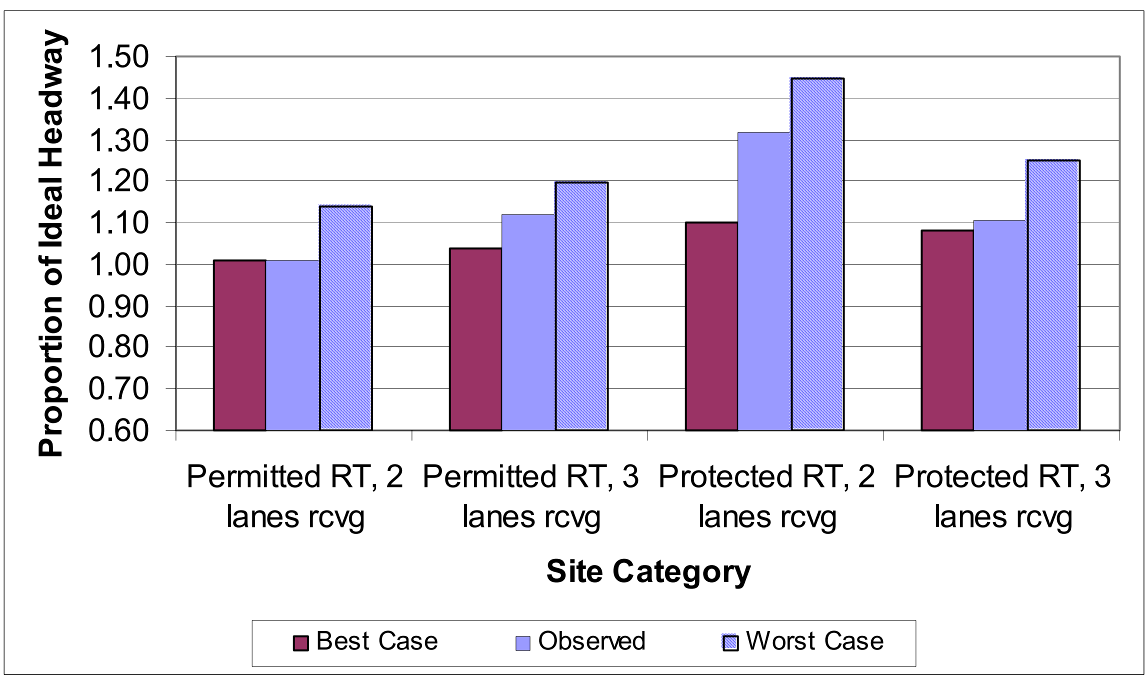


Figure 4.5. Comparison of Average Observed Headway to Best and Worst Case Scenarios for Queues with 50% U-Turns

4.1.6. Delay Data Results

One of the studies conducted on the 14 operational study sites was a stopped delay study. Using a Jamar electronic count board, an observer measured stopped delay in 15-second intervals during the entire data collection period. To determine the effects of U-turns on delay, I compared this observed delay to an estimate of delay calculated with the Highway Capacity Software (HCS2000).

Given the volumes, signal timing, and other intersection characteristics observed in the field, I calculated the estimated delay per vehicle. Table 4.8 compares the average observed delay to the HCS-calculated delay for peak hour traffic.

Table 4.8. Comparison of Observed and Estimated Delay

	Delay in seconds per vehicle					
Site	HCS Calculated	Observed	Difference (Obs-Calc)	Average Percentage U-turns	Conflicting Right Turn	No. Left Turn Lanes
202	78.5	76.9	-1.6	16	permitted	1
203	52.0	74.3	22.3	6	protected	2
204	72.1	71.5	-0.6	14	permitted	1
205	67.1	92.0	24.9	41	protected	2
207	60.0	75.7	15.7	32	protected	2
210	41.8	48.6	6.8	15	protected	2
211	54.5	55.2	0.7	28	protected	1
212	70.4	73.5	3.1	27	permitted	1
213	84.7	82.1	-2.6	34	permitted	2
215	64.5	74.5	10.0	52	protected	2
216	74.1	66.7	-7.4	32	permitted	1
217	76.5	73.1	-3.4	50	permitted	1
218	82.3	74.6	-7.7	13	protected	2
Average	**67.6**	**72.2**	**4.6**			

Since the HCS delay estimation procedure does not account for U-turns, it was thought that a comparison between the HCS estimate of delay and the field-observed delay would give some insight into the effect of U-turns on delay.

The most relevant comparison to make is between the calculated delay and observed delay. I used a t-test to determine if the mean difference between the two delay values were significant. When the sites were examined as one group, the mean difference between calculated and observed delay was 4.6 seconds. This produced a p-value of 0.16, which shows that this is not a significant difference at any commonly used confidence level. When the sites were examined as two groups, those with protected right turn overlap and those without, the mean difference between calculated and observed delay for the overlap group was 12.5 seconds. This produced a p-value of 0.09, which is significant at a confidence level of 90%.

This comparison supports the fact that protected right turn overlap is a significant factor concerning the operational effects of U-turns. Since the HCS procedure did not take U-turns into account with its delay estimation, its calculated delays were shown to significantly lower than the observed delays for sites with U-turns and protected right turn overlap.

4.1.7. Sensitivity Analysis of U-Turn Percentage on Lane Performance

The multivariate regression equation presented in section 4.1.3 gives the estimated saturation flow reduction for each increase in U-turn percentage. Although the effect on saturation flow is clear, one may wonder what effect this has on the bottom line, that is, the lane delay and level of service (LOS). To answer this question, I calculated the U-turn reduction factor f_{uturn} for various levels of U-turn percentage using the regression equation; then calculated the resulting delay with the Highway Capacity Software using the intersection data and the calculated f_{uturn} factor. These calculations gave the delay in seconds per vehicle as well as the comparable LOS.

To conduct a delay analysis of an exclusive left turn lane, an HCS user should use the calculated f_{uturn} factor along with the default 0.95 adjustment factor for exclusive, protected left turn lanes. This 0.95 factor should not be ignored because it accounts for the slower rate at which vehicles will make a left turn movement compared to a through movement. The HCS user can input the f_{uturn} adjustment factor by typing the value in one of the boxes provided for an adjustment factor that is not being used (i.e. displays a value of 1.00).

If the approach of interest has a single left turn lane, the HCS analysis is straightforward and the analyst should use the value for f_{uturn} calculated from the equation in section 4.1.3. However, if there are multiple left turn lanes, the f_{uturn} factor must be modified to account for the fact that U-turns will not have an effect on the saturation flow rate of the outside turn lane(s). Since the adjustment factors must be used for the lane group instead of individual lanes, the analyst must calculate an average value of f_{uturn} for the lane group. To calculate the weighted average value at sites with double left turn lanes, the following equation is recommended:

$$f^*_{uturn} = P_{uturn} * f_{uturn} + (1 - P_{uturn})$$

where:

f^*_{uturn} = weighted adjustment factor for delay calculations for sites with double left turn lanes

f_{uturn} = adjustment factor calculated from equation in section 4.1.3

P_{uturn} = proportion of total left-turning volume that turns from inside turn lane (includes left turns and U-turns)

This weighted factor can be used for left turn approaches that have any number of left turn lanes. However, the analyst must know the lane utilization among the turn lanes since the proportion of total turning volume that uses the inside lane is a required value. To produce the calculated delay for Figure 4.7, I used an even split between the inside and outside turn lanes (P_{uturn} of 0.5). In general, there was a fairly even distribution of turning volume between the two lanes at the study sites.

Figures 4.6 through 4.9 show the effect of increasing U-turn percentage on left turn lane group delay. The dotted lines on each graph demonstrate the HCM-defined cutoffs for each level of service. The effect of high U-turn percentage on lane group delay is not very dramatic in Figures 4.6 and 4.7. In general, the lane group did not experience a drop in LOS until the U-turn percentage reached approximately 70%. On average, each 10% increase in U-turn percentage caused an additional 1.5 seconds of delay to the lane group. However, the U-turn percentage did have a strong effect in Figure 4.9, which shows the one site in my group that had a protected right turn overlap and a single left turn lane. Since there are no other turn lanes at that site with which to average out the effect of the U-turn adjustment factor, the delay is strongly affected. On average, each 10% increase in U-turn percentage caused an additional 4.5 seconds of delay to the lane group.

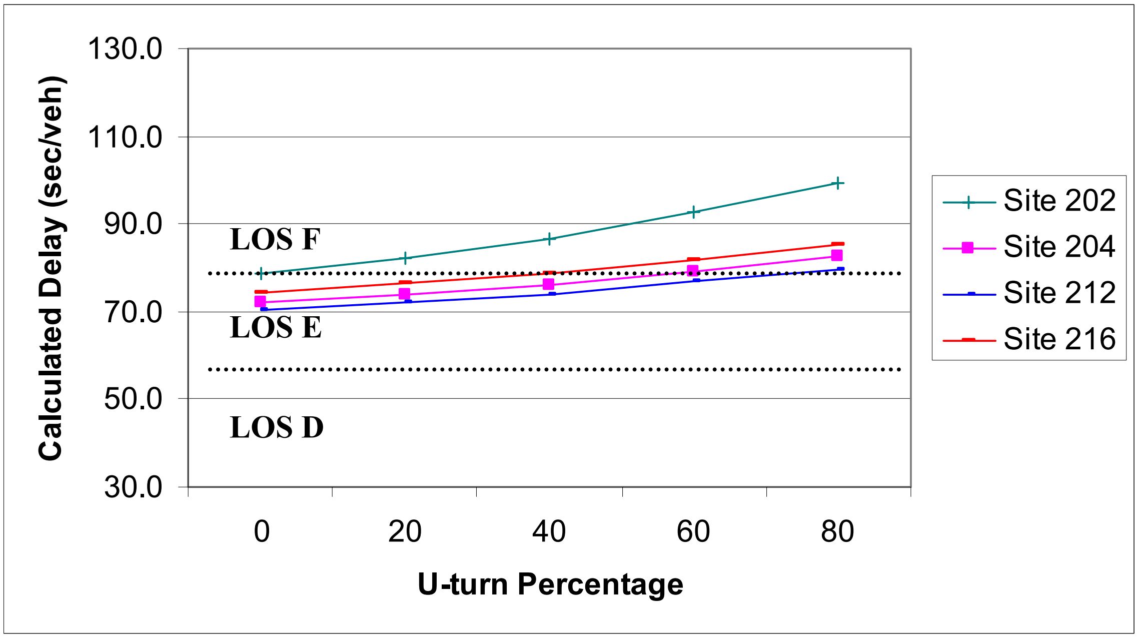


Figure 4.6. Effect of Increased U-Turn Percentage on Delay at Approaches with No Protected Right Turn Overlap and Single Left Turn Lane

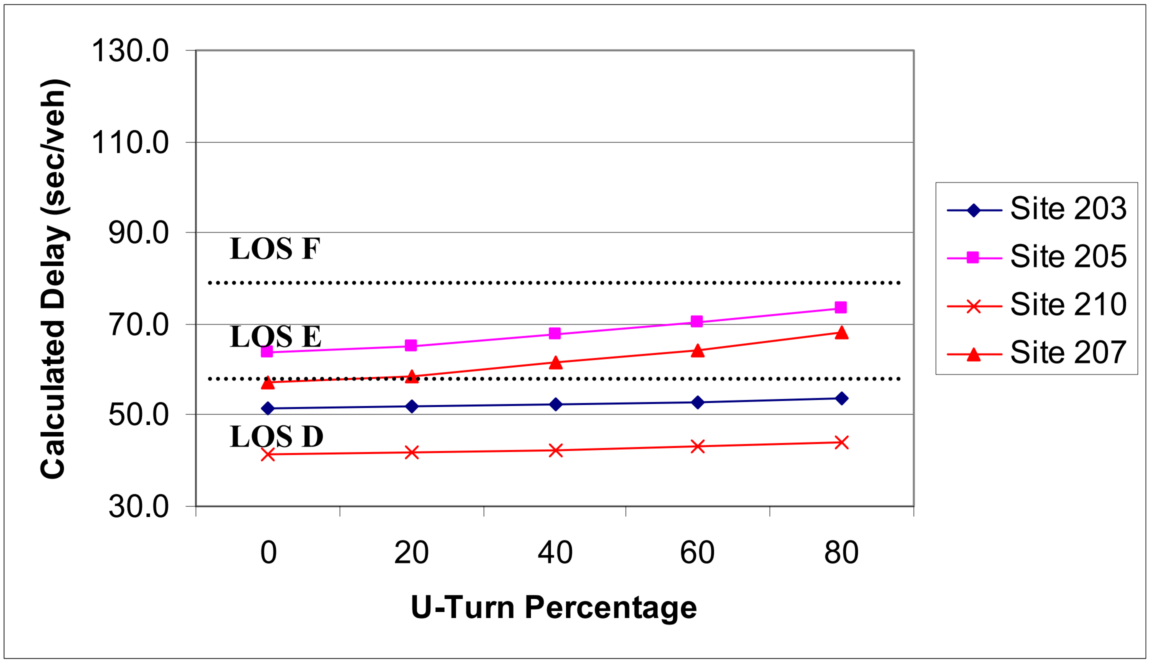


Figure 4.7. Effect of Increased U-Turn Percentage on Delay at Approaches with Protected Right Turn Overlap and Double Left Turn Lanes

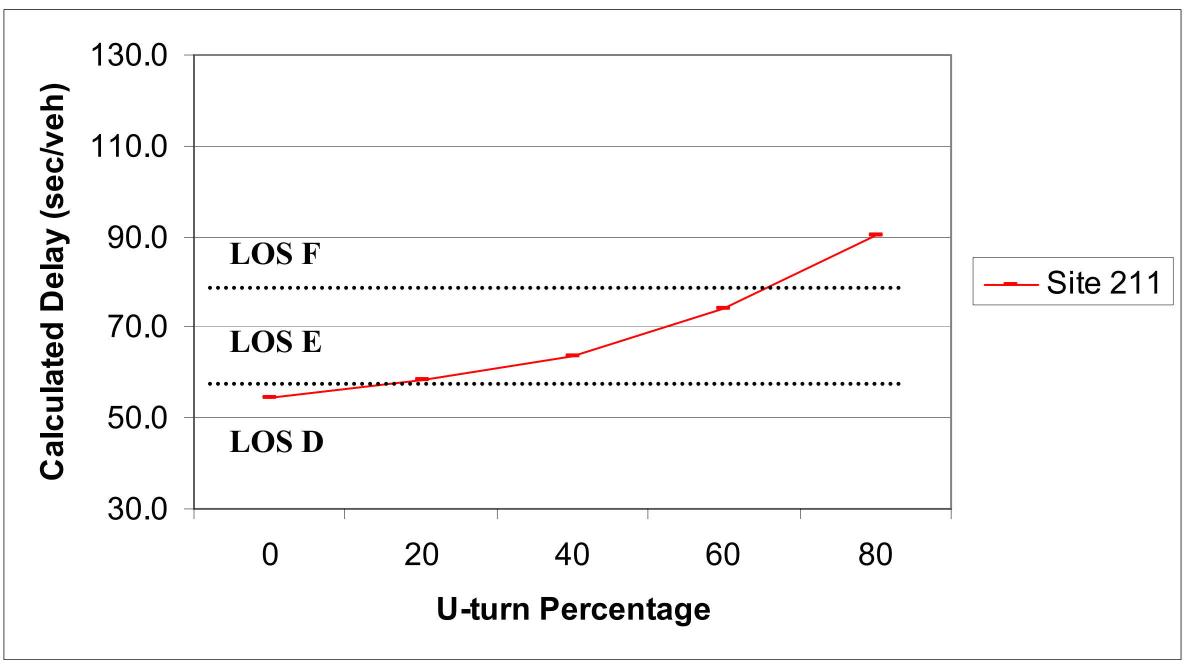


Figure 4.8. Effect of Increased U-Turn Percentage on Delay at an Approach with Protected Right Turn Overlap and Single Left Turn Lane

4.2. Safety Impacts of U-turns

The safety study included 78 sites, consisting of signalized intersections with protected left turns and two lanes receiving the U-turning vehicles. The sites were selected on a combined basis of random intersections and intersections recommended as U-turn “problem sites”. Data collected for these sites include geometry, traffic volumes, and history of collisions involving U-turns. The full database is available in Appendix J. Turning movement counts were obtained for one-third of the sites. The safety study was augmented by a conflict study at the operational study group of 14 sites.

4.2.1. Analysis of U-turn Collisions

One of the most significant findings of this research is seen in the U-turn collision frequency at the study sites. Figure 4.9 illustrates the fact that the majority of the study sites (65 out of 78) did not have any U-turn collisions in the three-year study period. It also shows that the maximum number of U-turn collisions seen on any intersection approach was three collisions per year, and that was observed only at one site. The mean number of collisions is 0.18 collisions per year with a 95% confidence interval of ± 0.11 collisions per year.

The distribution of collisions in Figure 4.9 is similar to a Poisson distribution, which is a typical distribution with collision data. However, the left side peak of this distribution is a bit higher than what a Poisson distribution would predict. Further study of the shape of this distribution may prove useful in developing a U-turn collision prediction model.

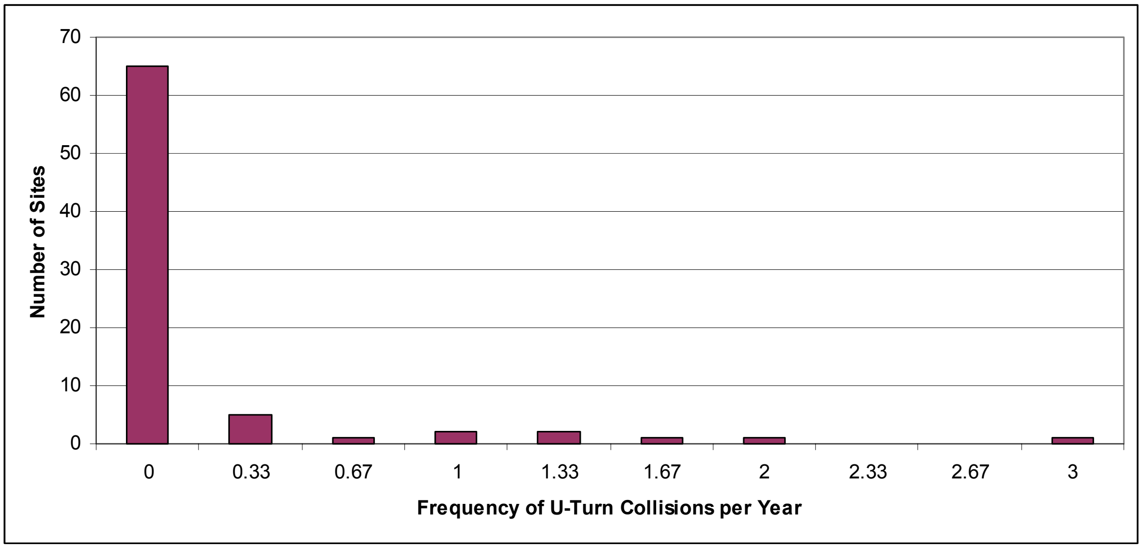


Figure 4.9. Histogram of U-Turn Collision Frequency

This finding is especially significant considering the criteria with which the study sites were selected. Twenty-four of the sites were selected solely for their reputation as U-turn "problem sites", known to have high U-turning volumes or a history of U-turn collisions. The other 54 sites in the group were randomly selected. In all, this makes a group of study sites which are biased to find more than the normal amount of U-turn collisions. However, only 13 sites had any U-turn collisions at all, and those frequencies ranged from 0.33 to 3.0 collisions per year.

From these 13 sites, a total of 41 U-turn collisions were noted. These collisions fell into one of three categories:

- *Angle* – This collision occurred between a U-turning vehicle and a vehicle making a conflicting right turn from the cross street.
- *Sideswipe* – This collision occurred where there was a double left turn lane and a vehicle attempted to make a U-turn from the outside turn lane.

- *Rear-end* – This collision occurred when a vehicle failed to reduce speed sufficiently to avoid hitting a U-turning vehicle. It was also caused by a right-turning vehicle yielding to a U-turn and being struck from behind – an occurrence that only happened once in the study period.

Table 4.9 displays the frequency of collisions by type. The most common U-turn collision was an angle collision, followed by rear-ends and sideswipes.

Table 4.9. Summary of Collision Types

Site Location			U-Turn Collisions in a 3-year Period			
Main Rd	**Dir**	**Cross St**	**Angle**	**Sideswipe**	**Rear-end**	**Total**
US 29	NB	Harris Blvd	2	3	4*	**9**
Eastway Dr	SB	Shamrock Dr	4	0	2	**6**
New Bern Ave	WB	Sunnybrook Rd	3	1	1	**5**
Glenwood Ave	EB	T.W. Alexander	3	0	1	**4**
Elizabeth Ave	WB	Kings Dr	4	0	0	**4**
US 29	NB	McCullough	1	0	2	**3**
I-277 off-ramp	NB	4th St	0	2	1	**3**
Creedmoor Rd	NB	Lynn Rd	0	2	0	**2**
US 321	SB	Pinewood Rd	1	0	0	**1**
S. College	NB	Holly Tree	1	0	0	**1**
US 29	NB	Dale Earnhardt	1	0	0	**1**
US 29	NB	Minnie	1	0	0	**1**
US 301	NB	Stone Rose	1	0	0	**1**
		TOTAL	**22**	**8**	**11**	**41**

* One of these rear-ends was in the right turn lane of the cross street. An abrupt stop by the right-turning vehicle yielding to a U-turning vehicle caused a rear-end collision on the cross street.

4.2.2. Significant Factors in U-turn Collisions

Collision results show that the average U-turn collision frequency per year per site was relatively low, with a large number of sites having zero collisions. Typically, a collision prediction model for a project such as this would sum up the significant factors and produce an equation for the expected number of U-turn collisions at a particular intersection given certain characteristics. However, the large number of sites with no collisions indicates that a collision prediction model may not be a helpful product of this research. I decided instead to focus on the site characteristics that correlate significantly with U-turn collisions.

I examined factors pertaining to geometry of the intersection, signal type, and traffic volume. Table 4.10 summarizes the factors and their effect on U-turn collisions. Each statistical test used a 90% confidence level. This level of confidence is appropriate for analyzing collision data, given that these data are of a random nature and were few in number. Using a stricter level would give more confident results but would eliminate factors that may have some contribution to the problem.

The statistical tests compared two groups of sites – those sites with one or more U-turn collisions and those sites without U-turn collisions – to see if a particular factor had significance. Appendices J and K contain details on the tests involved. If the factor had continuous data, such as median width in feet, I used a t-test to compare the mean value of the two groups. To verify the t-test results, I also used a Wilcoxon Rank Sum test, which differs from the t-test in that it does not assume any particular distribution of the data. These two tests agreed for all factors.

If the factor could be reduced to a yes/no situation (right turn overlap vs. no right turn overlap), I used a Chi-Square test comparison to determine significant difference. In the event that the expected values in the Chi-Square test were below five, I used the Fisher's Exact test, which gives a more accurate analysis for low expected values.

Table 4.10. Significant Factors in U-Turn Collisions

			Effect on U-Turn Collisions		
No.	**Characteristic**	**Groups to Compare**	**Significant? (90% conf)**	**Description**	**Statistical Test**
1	Median Width	Sites with collisions; sites w/o collisions	NO	-	T-test, Wilcoxon Rank Sum
2	Number of Left Turn Lanes	2 turn lanes; 1 turn lane	**YES**	Double left turn lane sites had more collisions than single left turn lane sites	Fisher's Exact
3	Right Turn Overlap	Overlap; no overlap	**YES**	Sites with protected right turn overlap had more collisions than sites without overlap	Fisher's Exact
4	Left Turn Signal Type	Permitted; protected; protected/permitted	NO	-	Fisher's Exact
5	Number of Access Points	Sites with collisions; sites w/o collisions	NO	-	T-test, Wilcoxon Rank Sum
6	Main Road ADT	Sites with collisions; sites w/o collisions	NO	-	T-test, Wilcoxon Rank Sum
7	AM Left Turn Volume	Sites with collisions; sites w/o collisions	**YES**	Sites with collisions had significantly higher turning volumes	T-test, Wilcoxon Rank Sum
8	AM Conflicting Right Turn Volume	Sites with collisions; sites w/o collisions	**YES**	Sites with collisions had significantly higher turning volumes	T-test, Wilcoxon Rank Sum
9	PM Left Turn Volume	Sites with collisions; sites w/o collisions	**YES**	Sites with collisions had significantly higher turning volumes	T-test, Wilcoxon Rank Sum
10	PM Conflicting Right Turn Volume	Sites with collisions; sites w/o collisions	**YES**	Sites with collisions had significantly higher turning volumes	T-test, Wilcoxon Rank Sum

Discussion of Site Characteristics

1. *Median width.* The width of medians at the study sites ranged from 2 to 48 feet, and all medians were raised. Median width was initially believed to be a significant factor based on the assumption that a wide median provides room for U-turning vehicles that have paused (see Figure 4.3) and allows for an easier, quicker U-turn. However, the analysis showed no significant difference in the mean width of sites with U-turn collisions and sites without U-turn collisions.

2. *Number of left turn lanes.* Analysis showed that sites with double left turn lanes had significantly higher proportion of U-turn collisions than sites with single left turn lanes. This could be caused by the fact that double left turn lanes create the possibility of collisions due to U-turns from the outside lane. All six sideswipe collisions in the study were caused by U-turns from the outside lane. Another possible reason for the significance of this characteristic is that sites with double left turn lanes are often accompanied by a protected right turn overlap, which proved to be a significant factor in U-turn collisions.

3. *Right turn overlap.* Most sites with protected right turn overlap had signs posted indicating that U-turns must yield to right-turning vehicles. In spite of this, the presence of right turn overlap proved to be a significant factor in U-turn collisions.

4. *Left turn signal type.* The types of left turn signals included in this study were protected, permitted, and protected/permitted. Upon comparison, these three groups were not found to have significantly different amounts of U-turn collisions.

5. *Number of access points.* This value is a count of the number of driveways and public streets on the median-divided segment leading to the intersection approach of interest. These

access points are anticipated to be the main generators of U-turns at most intersections, due to exiting drivers who make a right and U-turn instead of a direct left turn. For this reason, access points were counted only on the right-hand side of the road proceeding toward the intersection. No significance was found to this characteristic.

6. *Main road average daily traffic (ADT).* For this characteristic, the main road is defined as the road whose left turn lane is being studied. The main road ADT values ranged from 15,000 to 52,000 vehicles per day, with a median value of 30,000 vpd. These data were collected to investigate a common assumption that more traffic leads to more collisions. The nature of this U-turn collision study, however, proved too specific for a large-scale ADT to be a significant factor.

7-10. *AM and PM peak turning movements.* Because ADT was too broad a measure, I collected turning movement counts wherever available to determine the validity of the assumption that more left-turning and conflicting right-turning traffic results in more U-turn collisions. The left turn volume is the main road left turn count, including U-turns. The conflicting right turn volume is the count of cross street right turns that conflict with U-turning vehicles. The AM peak movement was counted from 7:30am-8:30am and the PM peak was counted from 5:00pm-6:00pm. When the two groups were compared (sites with U-turn collisions and sites without U-turn collisions), the groups with collisions were found to have significantly higher turning movement volumes for all movements studied.

4.2.3. Conflict Study Results

Conflict studies were conducted at each of the 14 operational study sites. Only conflicts involving U-turns were noted during the study. Such conflicts included:

- U-turn and left turn, same direction (near rear-end),
- U-turn and conflicting right turn (near angle),
- U-turn and adjacent vehicle (near sideswipe), and
- U-turn and pedestrian.

Table 4.11 summarizes the number of each type of conflict observed at each site. Since the number of observation hours varied at each site, the last column shows conflicts per hour, which is a more indicative measure of the frequency of conflicts at the intersection. The average observed U-turn conflict rate was 0.9 conflicts per hour for a single approach. In a report on conflict rate statistics, Glauz and Migletz predict a rate of 12 “left turn same direction” conflicts per hour for an intersection as a whole [13]. Even assuming the predicted rate for a single approach would be 3 conflicts per hour, the U-turn conflict rate seems to be much lower than the Glauz and Migletz rate. This would indicate that U-turn conflicts are only a small portion of the total conflicts at a left turn lane. Any more specific comparison with Glauz and Migletz cannot be performed since they did not analyze U-turn conflicts separately.

Table 4.11. Summary of U-Turn Conflict Data

Study No.	U-Turn Conflict Type				Total Conflicts Observed	U-Turn Conflicts per Hour
	Left Turn Same Direction	Right Turn and U-turn	Side-swipe	Ped and U-turn		
202	8	1	0	0	**9**	**0.7**
203	1	1	0	0	**2**	**0.3**
204	1	0	0	0	**1**	**0.1**
205	0	1	0	0	**1**	**0.2**
206	7	0	1	0	**8**	**1.2**
207	18	7	2	0	**27**	**2.9**
210	13	11	0	1	**25**	**2.8**
211	1	1	0	0	**2**	**0.3**
212	0	0	0	0	**0**	**0.0**
213	6	1	1	0	**8**	**1.3**
215	10	1	2	0	**13**	**2.0**
216	0	2	0	0	**2**	**0.3**
217	1	0	0	0	**1**	**0.2**
218	2	0	0	0	**2**	**0.5**
					Average	***0.9***

The sites with the highest U-turn conflict rates were those with the highest number of U-turn collisions. This precedent held true for all categories of conflicts. Table 4.12 shows the top five most hazardous sites according to both methods. See Appendix M for the tabulated comparison of conflict and collision data broken down into conflict categories.

Table 4.12. Comparison of Hazardous Sites Ranking by Conflict and Collision Rate

Rank	Ranked by Conflict Rate	Ranked by Collision Rate
1 (most hazardous)	207	207
2	210	210
3	215	215*
4	213	218*
5	206	206*

* Three-way tie for Rank 3

This confirms that the conflict study conducted to analyze U-turn safety at a group of intersections showed the correct priorities for the most dangerous intersections when compared to collision history.

A point of difference, however, between the conflict and collision results appears in the observed frequency of each type. Table 4.13 shows the summary of conflict and collision frequency according to the category of conflict. The most common conflict observed was the near rear-end (left turn same direction), with the second-most common being the right turn conflict. This differs from the collision listing, for which the most common collision was between a right turn and U-turn.

Table 4.13. Frequency of Conflicts and Collisions

Category	Conflicts*	Collisions**
Left turn same direction	68	10
Right turn and U-turn	26	22
Sideswipe	6	8

* These are the total conflicts observed at the 14 operational study sites.
** These are the total collisions observed at the 78 safety study sites in the three-year study period.

This difference may result from the nature of the rear-end conflicts. Most of these conflicts result from a left-turning vehicle failing to give sufficient room to the U-turn and coming to a quick stop a very short distance behind the U-turning vehicle. However, this type of conflict is conducted at low speeds and U-turns are usually anticipated at intersections where they are common. Conversely, conflicts between U-turns and right turns, while more rarely observed, have a greater potential to become collisions. The movements are usually conducted at higher speeds, especially if the right turn has a protected movement, and the path of the vehicles coming from different directions lends itself to the "came out of nowhere" situation.

5. Conclusions

5.1. Research Results

This project investigated the effects of U-turning vehicles on the operational capacity and safety of exclusive left turn lanes. The operational results indicated that increased U-turns will diminish left turn lane capacity according to the U-turn percentage and treatment of conflicting right turns. The safety study results found that U-turns have some impact on intersection safety by causing additional collisions, but the frequency of these collisions is low and the overall safety effect is minimal compared to the frequencies of other types of collisions at these busy intersections.

The results of the operational study quantified the capacity loss due to U-turn percentage in the left turn queue. The resulting regression equation is as follows:

$$f_{uturn} = 1.0 - 0.0018*UTURN - 0.0015*UTURN*OVERLAP$$

where:

f_{uturn} = saturation flow reduction factor for an exclusive left turn lane with protected phasing

UTURN = average U-turn percentage in the exclusive left turn lane (or inside turn lane if double left turn lanes)

OVERLAP = yes/no variable, 1 if conflicting right turn is protected overlap, 0 if no protected right turn overlap

This equation indicates a 1.8% saturation flow rate loss for every 10% increase in average U-turn percentage and an additional 1.5% loss per 10% U-turns if the U-turning movement is opposed by protected right turn overlap from the cross street. Transportation engineers should use this equation to adjust the expected saturation flow rate for a left turn lane for a more accurate estimate of the impact of increased U-turns.

The safety study examined collision history and conflict data. Although the group of study sites was purposely biased toward sites with high U-turn percentages, the study found that 65 of the 78 sites did not have any collisions involving U-turns in the three-year study period, and the U-turn collisions at the remaining 13 sites ranged from 0.33 to 3.0 collisions per year. Sites with double left turn lanes, protected right turn overlap, or high left turn and conflicting right turn traffic volumes were found to have a significantly greater number of U-turn collisions. Conflict studies of 14 sites agreed with collision data concerning the priority ranking of sites due to hazardous U-turns, but tended to predict a higher number of rear-end hazards than were observed in collision data.

Overall, U-turns do not have the large negative effect at signalized intersections that many have assumed. The safety impact is minimal for all types of intersections, including those with potential conflict by protected right turn overlap. On the operational side, the performance of the left turn lane group at most sites did not see a drop in LOS until U-turn percentage reached 70%. The impact was most noticeable on the one site with right turn overlap and a single left turn lane, where an increase of 35% in U-turn percentage caused a drop in LOS.

5.2. Qualitative Observations

Throughout the course of data collection for this project, I observed U-turns for over 100 hours. Most results of this observation time are captured in tables and figures throughout this paper. However, some qualitative observations may be informative as well as indicative of problems that are difficult to quantify.

I observed that large intersections provide room for left turners to circumvent a paused U-turn (see Figure 4.3). Smaller intersections do not have the space to allow for this bypass maneuver. However, consecutive U-turns are a problem at both small and large intersections. When the first of two or more U-turns is waiting for a right-turning vehicle to clear, the entire left turn queue must stop until all the U-turns have completed their maneuver. If the median is wide enough, it would be beneficial to have a median break with a turn bay specifically for U-turns some distance before the stop bar. This would get U-turning vehicles out of the way of left-turning vehicles and allow for greater capacity of the lane. In effect, this provides for the fact that the exclusive left turn lane is actually a shared left/U-turn lane. This concept is similar to the idea of a flared right turn, which allows right-turning vehicles in a shared through/right lane to make their turn with minimal effect on the through vehicles.

Same-direction conflicts with U-turning vehicles can be difficult to define. Rear-end close calls may happen between left turns and U-turns, but rarely do they become collisions, as shown in the collision history. Many times it seemed that drivers could have stopped farther back from the U-turning vehicle but wished to stop as close as possible for any number of reasons (e.g., show their displeasure at being forced to slow down).

Heavy vehicles would be expected to have more difficulty with U-turns, but I observed that their more experienced driving skills generally allow them to navigate a U-turn fairly well, in fact better than large passenger vehicles in some cases. The main difference is that trucks require more of the intersection in which to make their U-turn (e.g., "swinging out" farther). Since this can be unanticipated by other drivers, this could be a safety concern. However, out of the several observations of truck U-turns, I did not observe any conflicts. My research was not concerned with heavy vehicle U-turns, but if there are sufficient locations with a sizeable percentage of truck U-turns, this may be a topic for future research.

5.3. Recommendations for Future Research

For further research on the operational impacts of U-turns on left turn lanes, I have several suggestions that would allow for more precise data collection and analysis. First, observers should measure headways based on the moment when a vehicle's rear-axle touches the stop bar. I used a front-axle reference point in my research. I observed that the delay caused by conflicts between U-turning vehicles and right-turning vehicles sometimes occurs after the front axle has crossed the stop bar. A rear-axle reference may allow the research to more accurately quantify this delay. Second, conflicting right-turns-on-red or right-turns-on-arrow should be counted per left turn queue, as opposed to a 15-minute increment. This would allow for more detailed analysis of the effect of conflicting right turn volume on queue saturation flow.

Several other issues surrounding the topic of U-turns fell outside the scope of this research but would benefit from further studies. Future research could be dedicated to developing a model that would predict the number of U-turns at an intersection based on driveway density, land usage, and other such characteristics of the preceding roadway segment. A simple breakdown of land use into residential, business, or office may not be sufficient; it may be necessary to involve trip generation data for the various land parcels that have access points on the highway. The analysis should involve access points on both sides of the main road.

Future research could also study the effect of U-turning heavy vehicles on capacity and safety. A median installation may force delivery trucks and other heavy vehicles to make U-turns in order to complete their routes. A study could determine the effects of this situation and make informed suggestions about ways to minimize capacity loss and safety hazards with geometrical improvements.

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Appendices

Appendix A. Data Collection Form for Safety Study Intersections

County: ______________________ **Municipality:** ______________________

Main Street: ______________________

Cross Street: ______________________

Segment Beginning Point: ____________

Direction to Intersection: ____________

LT Signal Type	Conflicting RT
Permitted ○	Permitted ○
Protected ○	Protected ○
Both ○	Prohibited ○

Begin Mileage: [ ]

End Mileage: [ ]

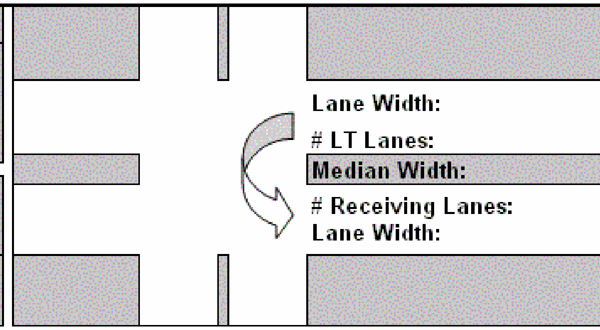
Length: [ ]

Speed Limit: [ ]

Public Street Approaches: [ ]

Driveways: [ ]

Land Use %		Useable?	
R	[ ]	Y	○
O	[ ]	N	○
B	[ ]		
I	[ ]		



Note any unusual grades or angles on the drawing

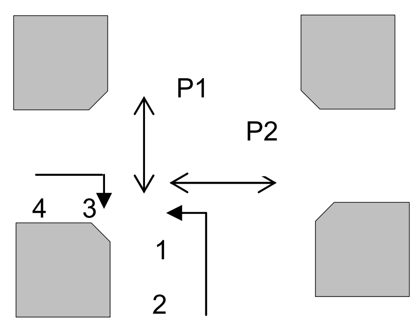
Notes:

Appendix B. Data Collection Form for Saturation Flow Study

Intersection:	
Approach:	
Lane Type:	
Date:	
Time Period:	
Observer:	

Cycle	Time	U-turn positions	0 U-t
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	
	: :	1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20	

Appendix C. Data Collection Form for Conflict Study

Actor Codes	Action Codes	Name:
	HA = Hesitate on green arrow HB = Hesitate on green ball U = U-turn S = Stop R = Ran red A = Accelerate B = Back up	Date: Time Period: Intersection: Direction (leg with actor 1): Weather:

Time	Actor 1	Action	Actor 2	Action	Comments

Appendix D. All Sites Observed for Selection in Operational Study

Main Rd	Dir	Cross St	Loc.	Eligible?	Reason if No	# Lns Rcvg
Harris Blvd	WB	N Tryon	Char	Yes maybe		2
Shipyard	EB	17th	Wilm	Yes maybe		2
US 64	WB	Edinburgh	Cary	Yes maybe		2
US 70	NB	New Rand	Gar	Yes maybe	Low U-turn cnt	2
Cary Pkwy	NB	High House	Cary	Yes		2
Cary Pkwy	NB	Kildaire Farm	Cary	Yes		2
I-277 ramp		4th St	Char	Yes		2
N Tryon	NB	Harris Blvd	Char	Yes		2
New Bern	WB	Sunnybrook	Ral	Yes		2
Creedmoor		Lynn	Ral	Yes		
Silas Creek	WB	Miller	WS	Yes		2
US 15-501	NB	Ephesus Church	CH	Yes		2
US 74	WB	Village Lake Dr	Char	Yes		2
US 70		T.W.Alexander	Ral	NO	Low U-turn cnt	2
US 70		Miami Blvd/Mineral Springs	Dur	NO	Low U-turn cnt	2
Airport	NB	Weaver Dairy	CH	NO	Low U-turn cnt	2
Alexander Dr	WB	Miami	Dur	NO	Low potential	2
Alexander Dr		Page Rd	Dur	NO	Low potential	2
Capital	SB	Calvary	Ral	NO	3 lns rcvg U-ts	3
Capital	SB	Millbrook/New Hope	Ral	NO	3 lns rcvg U-ts	3
Capital	NB	New Hope Church/Buffaloe	Ral	NO	4 lns rcvg U-ts	4
Capital	NB	Spring Forest	Ral	NO	3 lns rcvg U-ts	3
Cary Pkwy	NB	Bebington	Cary	NO	Low U-turn cnt	2
Cary Pkwy	NB	Chapel Hill Rd	Cary	NO	Low U-turn cnt	2
Cary Pkwy	SB	Chapel Hill Rd	Cary	NO	Low U-turn cnt	2
Cary Pkwy	SB	High House	Cary	NO	Low U-turn cnt	2
Cary Pkwy	EB	High Meadow	Cary	NO	Low U-turn cnt	2
Cary Pkwy	WB	High Meadow	Cary	NO	Low U-turn cnt	2
Cary Pkwy	NB	Lake Pine	Cary	NO	Low U-turn cnt	2
Cary Pkwy	NB	MacArthur/Bond Lake	Cary	NO	Low U-turn cnt	2
Cary Towne Blvd	WB	Maynard	Cary	NO	Low U-turn cnt	3
College	NB	Carolina Beach	Wilm	NO	Low U-turn cnt	2
College		New Centre	Wilm	NO	3 lns rcvg U-ts	3
College		Oriole	Wilm	NO	3 lns rcvg U-ts	3
College		Randall	Wilm	NO	3 lns rcvg U-ts	3
Creedmoor	NB	Brennan	Ral	NO	Low U-turn cnt	2
Creedmoor	SB	Howard/Bridgeport	Ral	NO	Low U-turn cnt	2
Creedmoor	NB	Howard/Bridgeport	Ral	NO	3 lns rcvg U-ts	3

Main Rd	Dir	Cross St	Loc.	Eligible?	Reason if No	# Lns Rcvg
Creedmoor	SB	Strickland	Ral	NO	Low U-turn cnt	2
Eastway	SB	Shamrock	Char	NO	Low U-turn cnt	2
Hanes Mall	WB	Lowes/Sams entrance	WS	NO	3 lns rcvg U-ts	3
High House	EB	Cary Pkwy	Cary	NO	Low U-turn cnt	2
High House	WB	Cary Pkwy	Cary	NO	Low U-turn cnt	2
Independence	SB	Oleander	Wilm	NO	Low U-turn cnt	2
Kildaire Fm	NB	New Waverly	Cary	NO	Low U-turn cnt	2
Kildaire Fm	SB	New Waverly	Cary	NO	Low U-turn cnt	2
Kildaire Fm	NB	Tryon	Cary	NO	Low U-turn cnt	2
Maynard		Cary Towne	Cary	NO	Low U-turn cnt	2
Maynard		High House	Cary	NO	Low potential	
Maynard	SB	Walnut	Cary	NO	Low U-turn cnt	2
Millbrook	EB	Creedmoor	Ral	NO	Low U-turn cnt	2
NC 15-501	NB	Sage	CH	NO	Low U-turn cnt	2
NC 42	NB	Lowe's entrance (exit 312 off I-40)	Clay	NO	Low U-turn cnt	2
Oleander		39th	Wilm	NO	3 lns rcvg U-ts	3
Peters Creek	NB	I-40 ramp	WS	NO	3 lns rcvg U-ts	3
Peters Creek		Tradesmart	WS	NO	3 lns rcvg U-ts	3
Peters Creek		Southpark	WS	NO	3 lns rcvg U-ts	3
Peters Creek		Link Rd	WS	NO	3 lns rcvg U-ts	3
Randall Pkwy	WB	Independence	Wilm	NO	Low U-turn cnt	2
S Kings	SB	Elizabeth Ave	Char	NO	Low U-turn cnt	2
S Kings	NB	Elizabeth Ave	Char	NO	Low U-turn cnt	2
Saunders	NB	Carolina Pines	Ral	NO	Low U-turn cnt	
Saunders	SB	Carolina Pines	Ral	NO	3 lns rcvg U-ts	3
Saunders	SB	Maywood	Ral	NO	Low U-turn cnt	
Saunders	NB	Maywood	Ral	NO	3 lns rcvg U-ts	3
Shamrock	WB	Eastway	Char	NO	Low U-turn cnt	2
Silas Creek	EB	Miller	WS	NO	Low U-turn cnt	2
South Blvd	SB	Tyvola	Char	NO	Low U-turn cnt	2
South Blvd	NB	Tyvola	Char	NO	Low U-turn cnt	2
Tryon	WB	Kildaire Farm	Cary	NO	Low U-turn cnt	2
Tryon	WB	Regency Pkwy	Cary	NO	3 lns rcvg U-ts	3
US 15-501		Garrett	Dur	NO	3 lns rcvg U-ts	3
US 15-501		Mt Moriah	Dur	NO	3 lns rcvg U-ts	3
US 15-501	NB	Elliot	CH	NO	Low U-turn cnt	2
US 15-501	SB	Elliot	CH	NO	Low U-turn cnt	2
US 15-501	NB	Manning	CH	NO	Low potential	2
US 15-501		Old Mason Farm	CH	NO	Low potential	2
US 15-501		S.Estes	CH	NO	Low U-turn cnt	2
US 15-501	NB	Willow	CH	NO	Low U-turn cnt	2
US 15-501 (Bus)		Tower Blvd	Dur	NO	3 lns rcvg U-ts	3
US 401	NB	Ten Ten	Gar	NO	Low U-turn cnt	
US 401	SB	Ten Ten	Gar	NO	Low U-turn cnt	

Main Rd	Dir	Cross St	Loc.	Eligible?	Reason if No	# Lns Rcvg
US 64	EB	Corporation	Ral	NO	4 lns rcvg U-ts	4
US 64	EB	New Hope Rd	Ral	NO	Low U-turn cnt	
US 64	EB	Trawick	Ral	NO	3 lns rcvg U-ts	3
US 64	WB	Gregson Dr	Cary	NO	Low U-turn cnt	2
US 64	WB	Lake Pine	Cary	NO	Low U-turn cnt	2
US 64	EB	Laura Duncan	Cary	NO	Low U-turn cnt	2
US 70	WB	Duraleigh/Millbrook	Ral	NO	Short queues	3
US 70	EB	Duraleigh/Millbrook	Ral	NO	Short queues	3
Us 70	EB	Pleasant Valley	Ral	NO	3 lns rcvg U-ts	3
Us 70	EB	Pleasant Valley Promenade	Ral	NO	3 lns rcvg U-ts	3
US 70	WB	Mechanical	Gar	NO	3 lns rcvg U-ts	3
US 70	SB	New Rand	Gar	NO	Low U-turn cnt	2
US 70	WB	Page	Gar	NO	3 lns rcvg U-ts	3
US 70	NB	Yeargan	Gar	NO	Low U-turn cnt	2
US 70	SB	Yeargan	Gar	NO	Low U-turn cnt	2
US 70		Page Rd Ext	Dur	NO	3 lns rcvg U-ts	3
US 70		Pleasant	Dur	NO	No median	
US 74	EB	Village Lake Dr	Char	NO	3 lns rcvg U-ts	2
Walnut	SB	Dillard	Cary	NO	Low U-turn cnt	2
Walnut	WB	Maynard	Cary	NO	Low U-turn cnt	2
Walnut	SB	Meeting St	Cary	NO	Low U-turn cnt	2
Western	EB	Blue Ridge	Ral	NO	Low potential	3
Western	WB	Kent	Ral	NO	3 lns rcvg U-ts	3

Appendix E. Operational Study Site Information

	Site and Approach Location				Intersection Information										
											Width of Receiving Lanes (ft)				
Study No.	Main Rd	Dir	Cross St	Loc.	LT Signal Type	Conflicting RT	No. LT lanes	LT Lane Width	Median Width	No. Lns Rcvg	Inside Lane	Next Lane Over	Next Lane Over	Shoulder Width	Total Rcvg Width
202	Cary Pkwy	NB	Kildaire Farm	Cary	prot	perm	1	13	16	2	15	15	-	0	30
203	US 64	WB	Edinburgh	Cary	prot	prot	2	13	20	2	13	12	-	0	25
204	US 15-501	NB	Ephesus Church	CH	prot	perm	1	12	10	2	13	12	-	1	26
205	Harris Blvd	WB	N Tryon	Char	prot	prot	2	13	15	2	15	15	-	48	78
206	I-277 ramp	NB	4th St	Char	prot	none	2	11	4	2	13	12	-	8	33
207	N Tryon	NB	Harris Blvd	Char	prot	prot	2	10	7	2	12	11	-	6	29
210	New Bern	WB	Sunnybrook	Ral	prot	prot	2	11	13	2	11	13	-	12	36
211	Silas Creek	WB	Miller	WS	prot	prot	1	12	3	2	12	12	-	3	27
212	Capital	SB	Calvary	Ral	prot	perm	1	12	19	3	12	12	12	10	46
213	Capital	SB	Millbrook/New Ho	Ral	prot	perm	2	11	6	3	12	12	12	10	46
215	US 64	EB	Trawick	Ral	prot	prot	2	13	14	3	13	13	13	3	42
216	US 70	EB	Pleasant Valley P	Ral	prot	perm	1	11	15	3	12	12	12	0	36
217	Western	WB	Kent	Ral	prot	perm	1	11	7	3	11	11	12	0	34
218	Creedmoor	NB	Lynn	Ral	prot	prot	2	10	3	2	11	11	-	12	34

	Site and Approach Location				Safety Information											
					Conflicts Involving U-Turns											
Study No.	Main Rd	Dir	Cross St	Loc.	Left Turn Same Direction	RT and U-turn	Near Sideswipe	Ped and U-turn	Total Conflicts	Number Hours Observed	Left Turn Same Direction per hour	RT and U-turn per hour	Near Sideswipe per hour	Total conflicts per Hour	No. U-turn Collisions (3 yrs)	U-turn Collisions per year
202	Cary Pkwy	NB	Kildaire Farm	Cary	8	1	0	0	**9**	13.50	0.6	0.1	0.0	**0.7**	0	0.0
203	US 64	WB	Edinburgh	Cary	1	1	0	0	**2**	6.25	0.2	0.2	0.0	**0.3**	0	0.0
204	US 15-501	NB	Ephesus Church	CH	1	0	0	0	**1**	9.00	0.1	0.0	0.0	**0.1**	0	0.0
205	Harris Blvd	WB	N Tryon	Char	0	1	0	0	**1**	5.50	0.0	0.2	0.0	**0.2**	0	0.0
206	I-277 ramp	NB	4th St	Char	7	0	1	0	**8**	6.50	1.1	0.0	0.2	**1.2**	3	1.0
207	N Tryon	NB	Harris Blvd	Char	18	7	2	0	**27**	9.25	1.9	0.8	0.2	**2.9**	9	3.0
210	New Bern	WB	Sunnybrook	Ral	13	11	0	1	**25**	9.00	1.4	1.2	0.0	**2.8**	5	1.7
211	Silas Creek	WB	Miller	WS	1	1	0	0	**2**	6.25	0.2	0.2	0.0	**0.3**	1	0.3
212	Capital	SB	Calvary	Ral	0	0	0	0	**0**	8.50	0.0	0.0	0.0	**0.0**	0	0.0
213	Capital	SB	Millbrook/New Ho	Ral	6	1	1	0	**8**	6.25	1.0	0.2	0.2	**1.3**	0	0.0
215	US 64	EB	Trawick	Ral	10	1	2	0	**13**	6.50	1.5	0.2	0.3	**2.0**	3	1.0
216	US 70	EB	Pleasant Valley P	Ral	0	2	0	0	**2**	6.25	0.0	0.3	0.0	**0.3**	0	0.0
217	Western	WB	Kent	Ral	1	0	0	0	**1**	6.50	0.2	0.0	0.0	**0.2**	recent constr	
218	Creedmoor	NB	Lynn	Ral	2	0	0	0	**2**	4.00	0.5	0.0	0.0	**0.5**	3	1.0
									Average =	**7.38**						

	Site and Approach Location				Volumes and Turning Movements							
					Turn Movement (12:00pm-1:00pm, vph)			Turn Movement (day avg, vph)				
Study No.	Main Rd	Dir	Cross St	Loc.	Left turn	U-turn	RTOA/RTOR	Left turn	U-turn	RTOA/ RTOR	Main Rd ADT	Main Rd MP
202	Cary Pkwy	NB	Kildaire Farm	Cary	185	28	2	157	15	4	20000	
203	US 64	WB	Edinburgh	Cary	182	5	64	145	8	41	38000	8.46
204	US 15-501	NB	Ephesus Chu	CH	193	25	1	143	18	1	28500	6.92
205	Harris Blvd	WB	N Tryon	Char	136	62	12	112	35	19	54000	
206	I-277 ramp	NB	4th St	Char	287	-	0	376	148	0	can't find	
207	N Tryon	NB	Harris Blvd	Char	240	84	64	196	58	53	25000	18.4
210	New Bern	WB	Sunnybrook	Ral	249	32	176	241	23	149	32000	2.94
211	Silas Creek	WB	Miller	WS	201	61	5	185	44	7	30000	17.3
212	Capital	SB	Calvary	Ral	163	51	12	135	30	5	39000	29.6
213	Capital	SB	Millbrook/Nev	Ral	209	70	38	208	58	33	39000	30.1
215	US 64	EB	Trawick	Ral	330	105	63	288	130	57	61800	25.45
216	US 70	EB	Pleasant Vall	Ral	185	54	30	131	34	17	33000	9.8
217	Western	WB	Kent	Ral	139	69	5	112	48	2	38000	
218	Creedmoor	NB	Lynn	Ral	154	17	19	135	16	20	30600	21.54

* Creedmoor and Lynn only has turning movements for 4:30pm - 6:00pm
The one hour peak value is an average of both days 5:00pm - 6:00pm

	Site and Approach Location				Segment Information												
									Land Use Percentage								
Study No.	Main Rd	Dir	Cross St	Loc.	Segment Beginning	Segment Length (mi)	Public St Approaches	Driveways	Residential	Office	Business	Industrial	Speed Limit (mph)	Reduction Factor for Queues of 20% U-turns	Reduction Factor for Queues of 50% U-turns	Average Queue Length for 50% U-turn Queues	Average Site Reduction Factor
202	Cary Pkwy	NB	Kildaire Farm	Cary	High Meadow	0.1	0	1	0	0	100	0	45	0.97	1.00	5.00	0.99
203	US 64	WB	Edinburgh	Cary	-	0.15	0	0	0	0	0	0	45	0.97		-	0.98
204	US 15-501	NB	Ephesus Chu	CH	-	0.2	1	0	0	0	100	0	45	1.00		5.00	1.01
205	Harris Blvd	WB	N Tryon	Char	median break	1	1	2	0	0	20	0	45	0.84	0.74	5.25	0.84
206	I-277 ramp	NB	4th St	Char	-	-	-	-	-	-	-	-	-		0.89	6.50	0.83
207	N Tryon	NB	Harris Blvd	Char	McCullough	0.4	1	4	0	0	100	0	45	0.95	0.81	7.36	0.89
210	New Bern	WB	Sunnybrook	Ral	Yonkers	0.3	1	0	0	0	0	0	45	0.92		5.00	0.94
211	Silas Creek	WB	Miller	WS										0.95	0.88	6.14	0.94
212	Capital	SB	Calvary	Ral	Millbrook	0.35	0	1	0	0	100	0	45	0.96	0.87	7.29	0.96
213	Capital	SB	Millbrook/Nev	Ral	Spring Forest	0.35	0	4	0	0	100	0	45	0.96	0.86	8.10	0.93
215	US 64	EB	Trawick	Ral	-	0.2	0	2	0	0	100	0	45	0.97	0.90	11.43	0.87
216	US 70	EB	Pleasant Vall	Ral	easant Valley	0.2	0	5	0	0	100	0	45	0.97	0.88	5.00	0.95
217	Western	WB	Kent	Ral	Clanton	0.2	0	10	0	0	100	0	45		0.88	5.77	0.97
218	Creedmoor	NB	Lynn	Ral	-	0.3	0	8	50	0	50	0	45	0.88		-	0.95
															Average =	**6.49**	

Appendix F. Multivariate Regression Summary Output.

This regression analysis was performed with Microsoft Excel 2002.

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.889
R Square	0.791
Adjusted R Square	0.753
Standard Error	0.027
Observations	14

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	2	0.031	0.016	20.789	0.000
Residual	11	0.008	0.001		
Total	13	0.040			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	1.0097	0.0146	69.1318	0.0000	0.9776	1.0419	0.9776	1.0419
Average Percentage U-turns	-0.0018	0.0004	-4.7765	0.0006	-0.0027	-0.0010	-0.0027	-0.0010
Interaction of U-turn percentage and overlap	-0.0015	0.0004	-3.5177	0.0048	-0.0025	-0.0006	-0.0025	-0.0006

Appendix G. Statistical Tests for Saturation Flow Reduction Factors

Tests for difference in reduction factors according to number of receiving lanes

No. Lns Rcvg	Reduction Factor for Queues of 20% U-turns	Reduction Factor for Queues of 50% U-turns
2	0.84	0.74
2	0.88	
2	0.92	
2	0.95	0.81
2	0.95	0.88
2	0.97	1.00
2	0.97	
2	1.00	
2		0.89
3	0.96	0.86
3	0.96	0.87
3	0.97	0.88
3	0.97	0.90
3		0.88
T-test p-value =	0.14661172	0.379066605

Tests for difference in reduction factors according to number of LT lanes

No. LT lanes	Reduction Factor for Queues of 20% U-turns	Reduction Factor for Queues of 50% U-turns
1	0.97	1.00
1	1.00	
1	0.95	0.88
1	0.96	0.87
1	0.97	0.88
1		0.88
2	0.97	
2	0.84	0.74
2		0.89
2	0.95	0.81
2	0.92	
2	0.96	0.86
2	0.97	0.90
2	0.88	
T-test p-value =	0.05	0.07
Difference in means =	0.04	0.06

Tests for difference in reduction factors according to conflicting RT type

Conflicting RT	Reduction Factor for Queues of 20% U-turns	Reduction Factor for Queues of 50% U-turns
perm	0.96	0.87
perm	0.96	0.86
perm	0.97	0.88
perm	0.97	1.00
perm	1.00	
perm		0.88
prot	0.84	0.74
prot	0.88	
prot	0.92	
prot	0.95	0.88
prot	0.95	0.81
prot	0.97	
prot	0.97	0.90
prot		

T-test p-value	0.04	0.09
Difference in group means =	0.05	0.07

Regression analysis of Conflicting RT effect

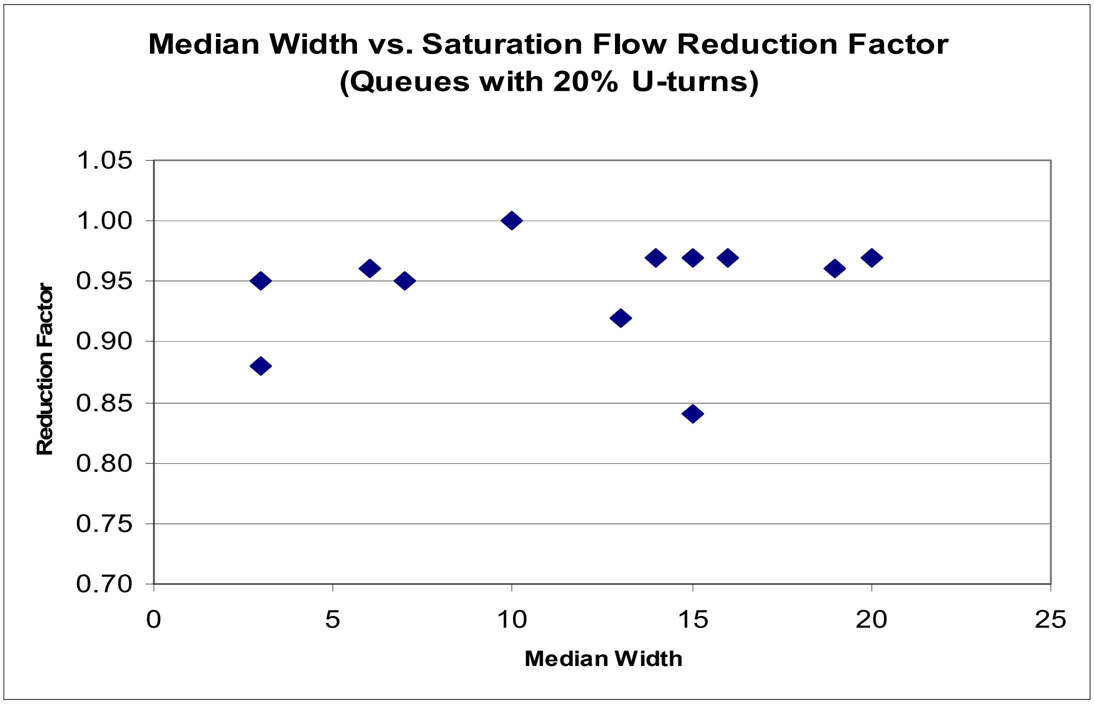
RTOA/RTOR	Reduction Factor for Queues of 20% U-turns
4	0.97
41	0.97
1	1.00
19	0.84
53	0.95
149	0.92
7	0.95
5	0.96
33	0.96
57	0.97
17	0.97
20	0.88

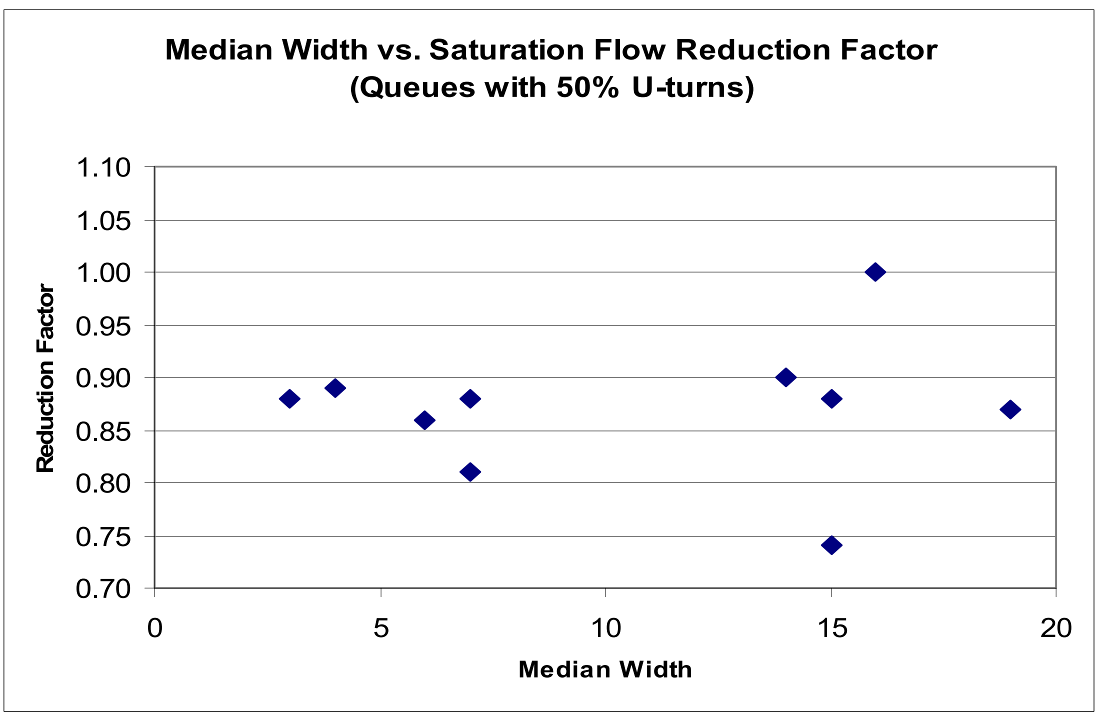
Regression slope p-value = 0.63

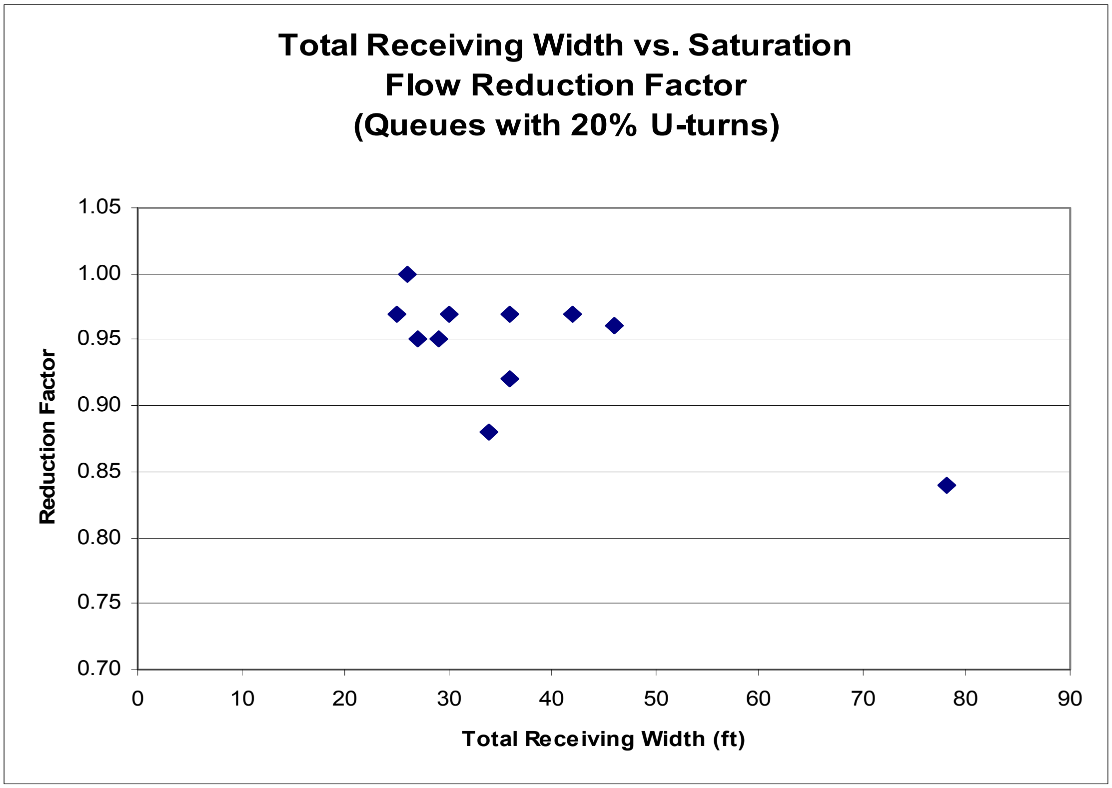
RTOA/RTOR	Reduction Factor for Queues of 50% U-turns
4	1.00
19	0.74
0	0.89
53	0.81
7	0.88
5	0.87
33	0.86
57	0.90
17	0.88
2	0.88

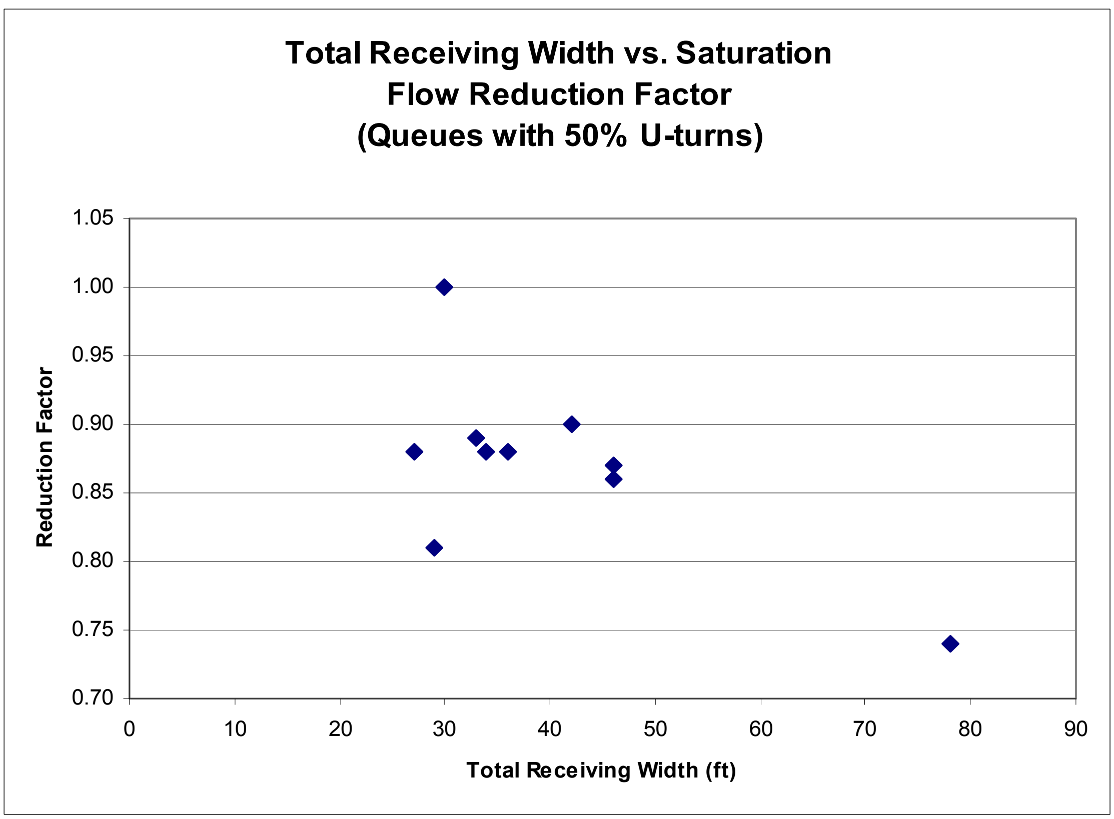
Regression slope p-value = 0.41

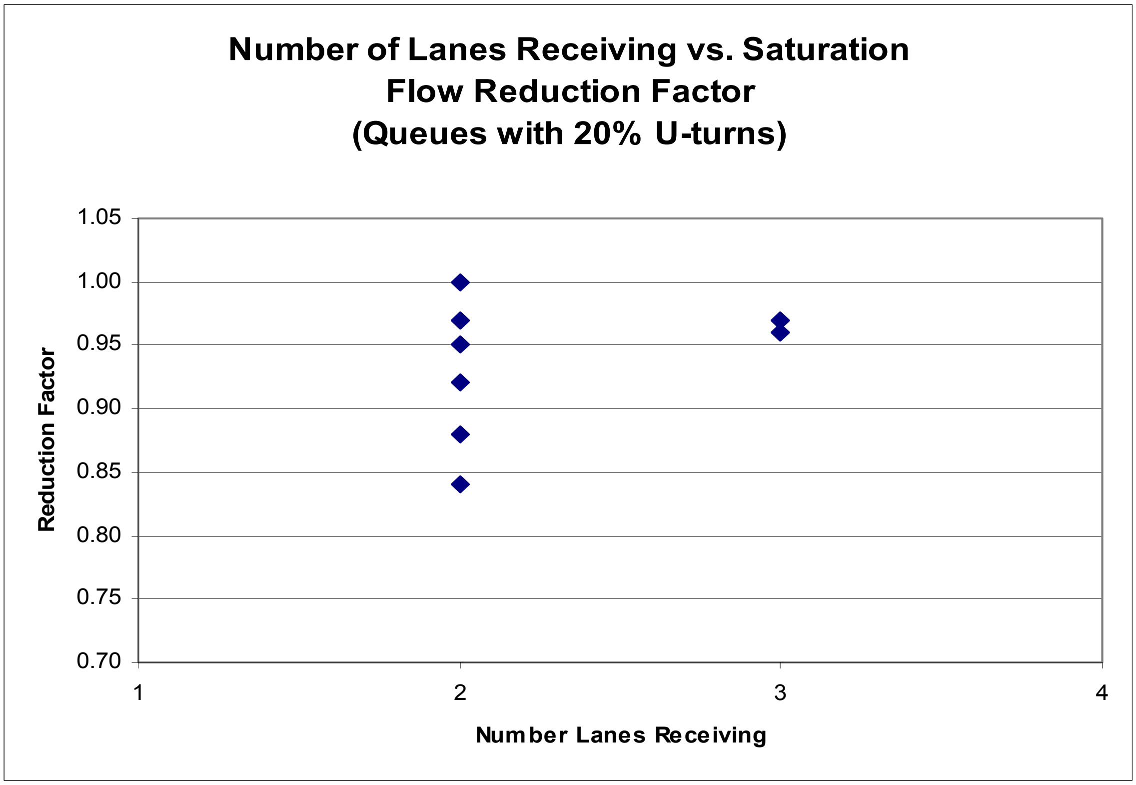
Appendix H. Graphs of Reduction Factor versus Various Site Characteristics

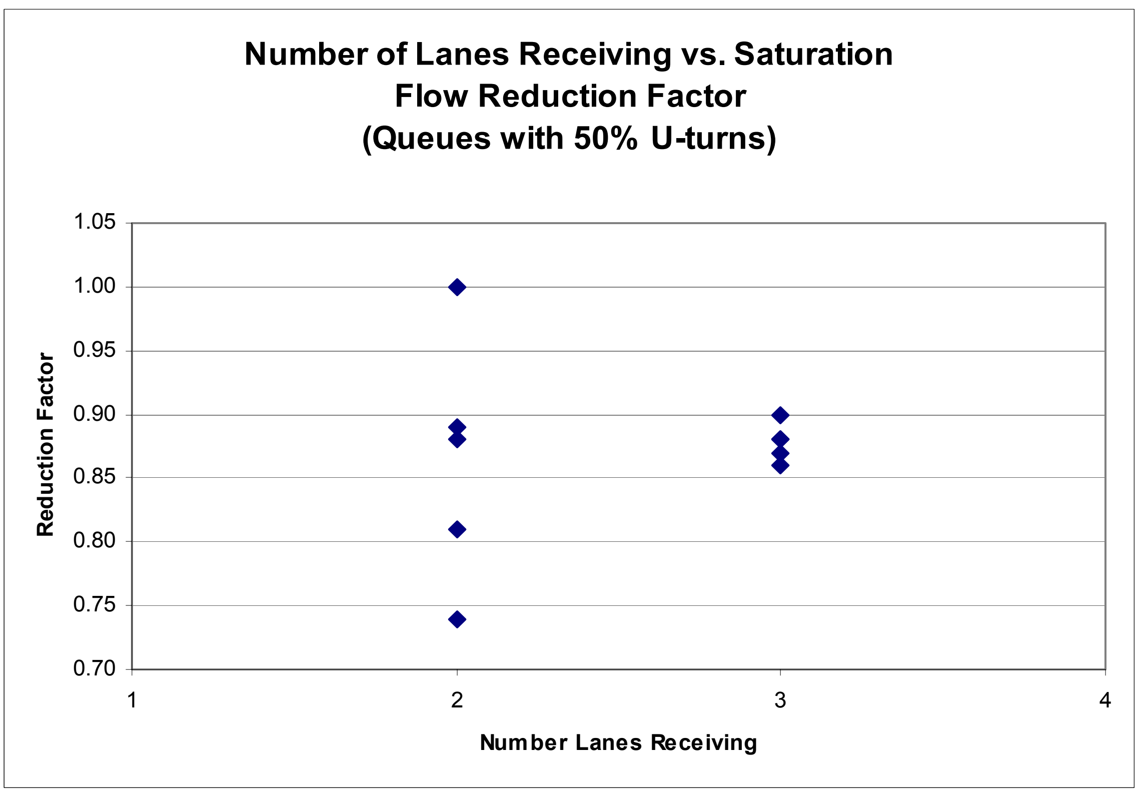


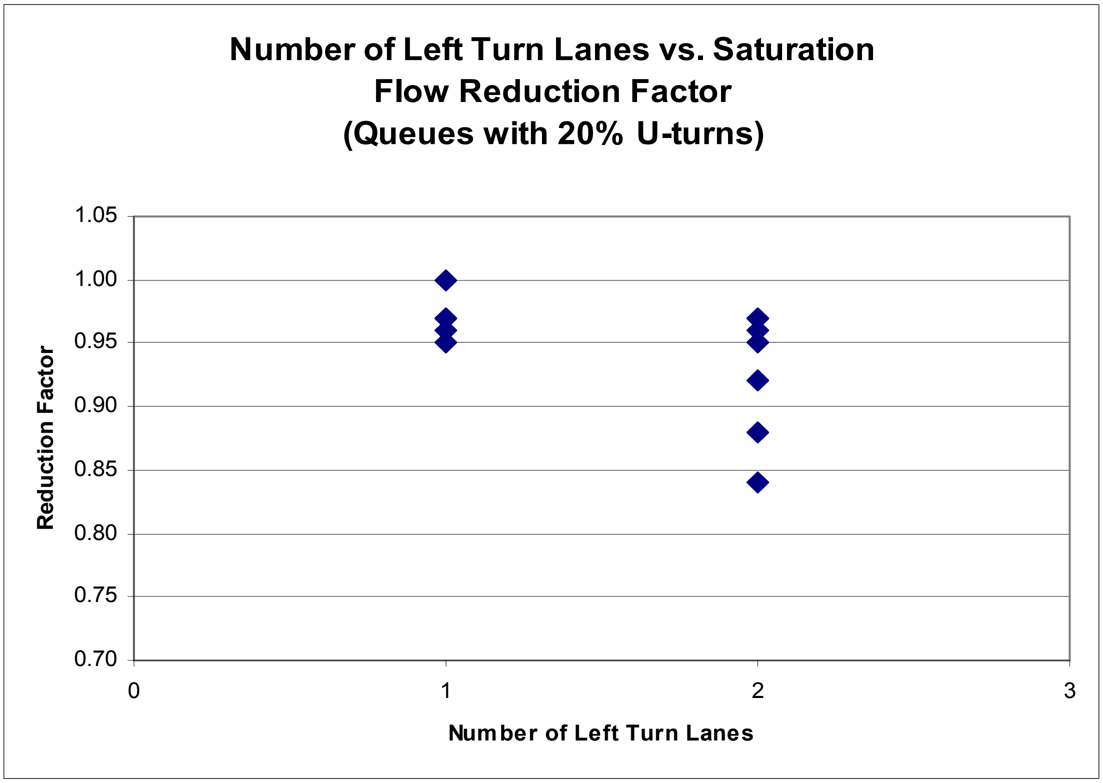


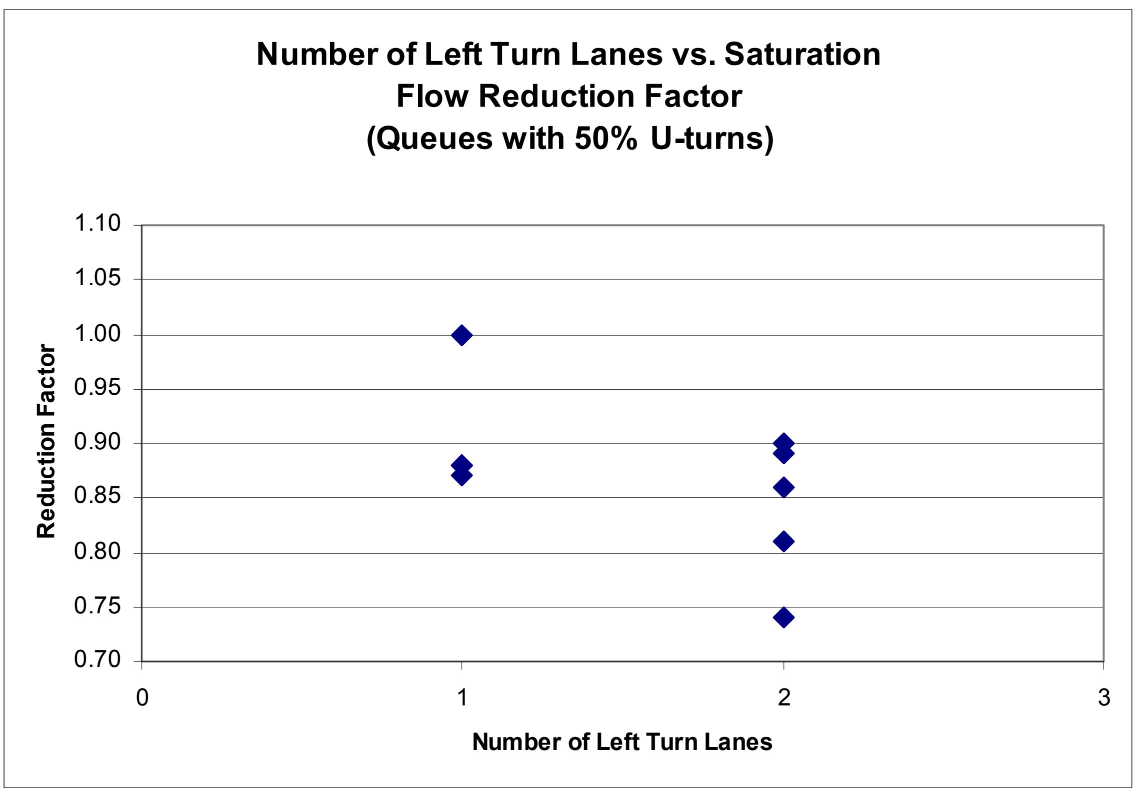

Total Receiving Width vs. Saturation
Flow Reduction Factor
(Queues with 20% U-turns)
Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
10
20
30
40
50
60
70
80
90
Total Receiving Width (ft)

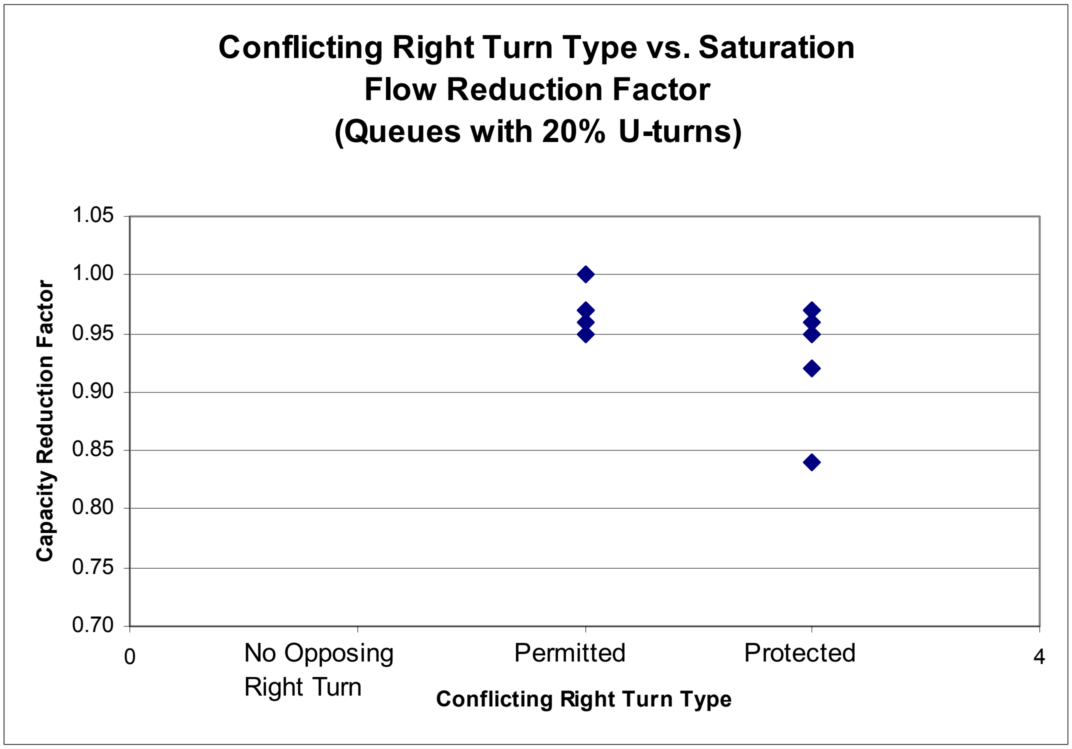

Total Receiving Width vs. Saturation
Flow Reduction Factor
(Queues with 50% U-turns)
Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
10
20
30
40
50
60
70
80
90
Total Receiving Width (ft)

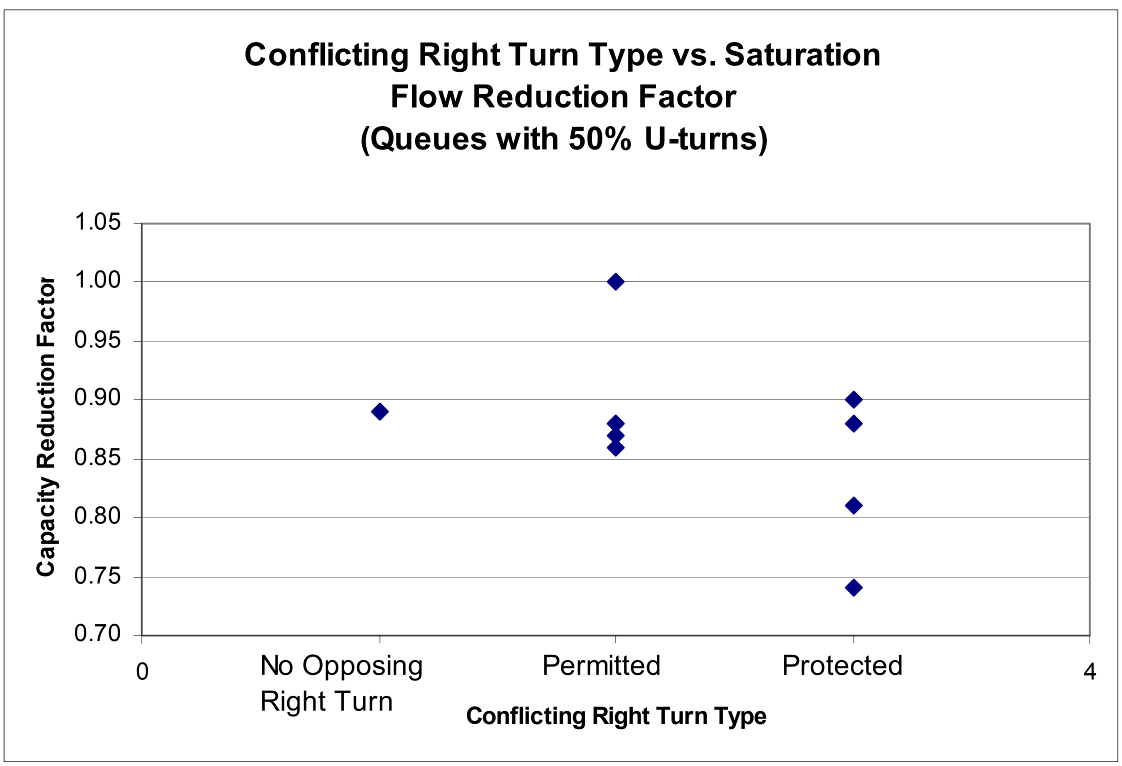

Number of Lanes Receiving vs. Saturation Flow Reduction Factor (Queues with 20% U-turns)
Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
1
2
3
4
Number Lanes Receiving

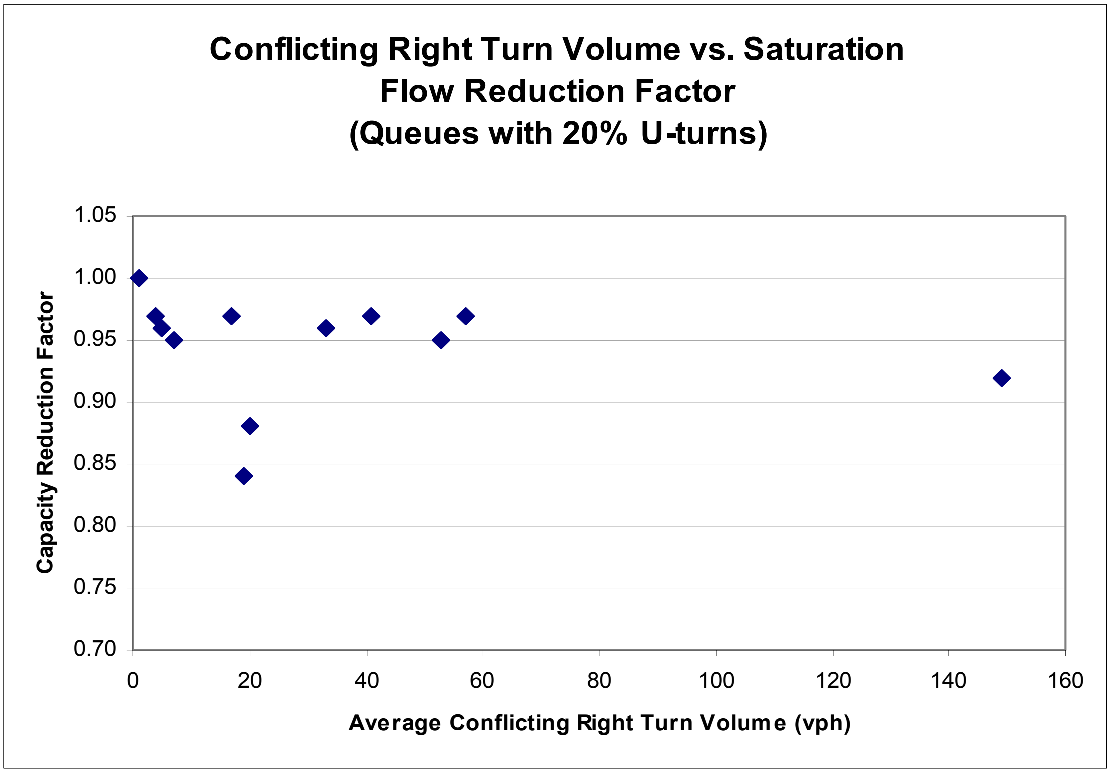

Number of Lanes Receiving vs. Saturation Flow Reduction Factor (Queues with 50% U-turns)
Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
1
2
3
4
Number Lanes Receiving

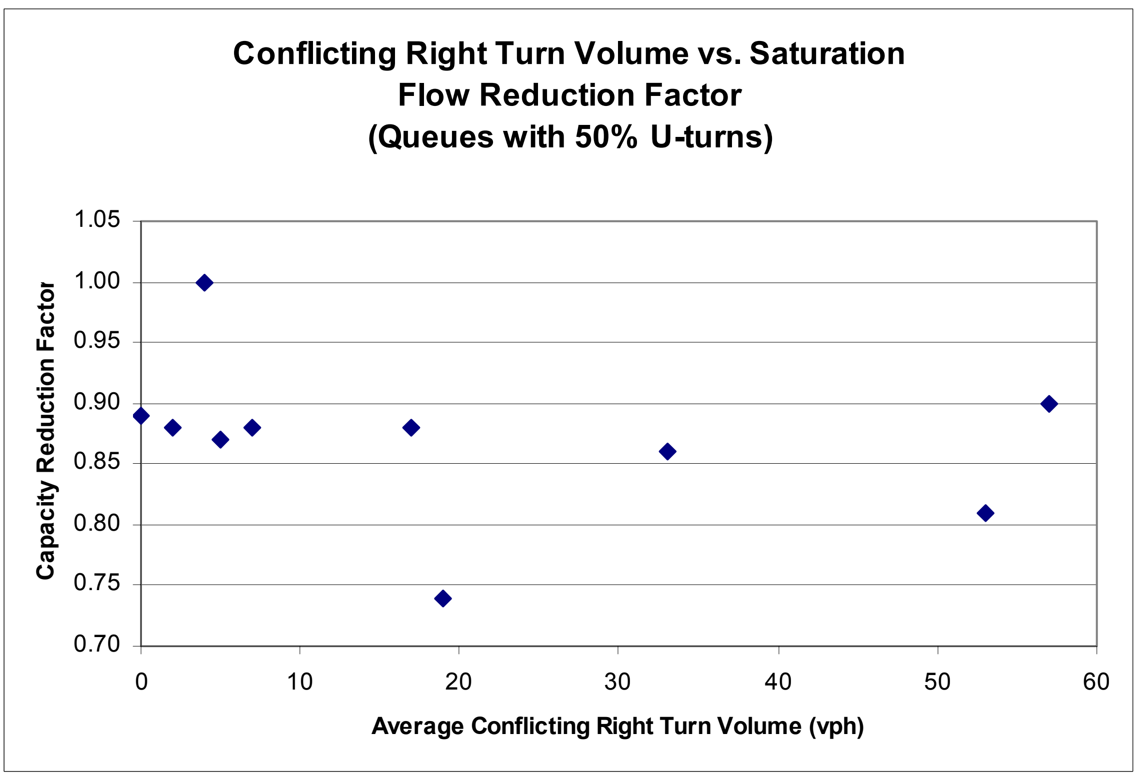

Number of Left Turn Lanes vs. Saturation Flow Reduction Factor (Queues with 20% U-turns)
Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
1
2
3
Number of Left Turn Lanes


Number of Left Turn Lanes vs. Saturation Flow Reduction Factor (Queues with 50% U-turns)
Reduction Factor
1.10
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
1
2
3
Number of Left Turn Lanes


Conflicting Right Turn Type vs. Saturation Flow Reduction Factor (Queues with 20% U-turns)
Capacity Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
No Opposing Right Turn
Permitted
Protected
4
Conflicting Right Turn Type


Conflicting Right Turn Type vs. Saturation Flow Reduction Factor (Queues with 50% U-turns)
Capacity Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
No Opposing Right Turn
Permitted
Protected
4
Conflicting Right Turn Type


Conflicting Right Turn Volume vs. Saturation
Flow Reduction Factor
(Queues with 20% U-turns)
Capacity Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
20
40
60
80
100
120
140
160
Average Conflicting Right Turn Volume (vph)


Conflicting Right Turn Volume vs. Saturation
Flow Reduction Factor
(Queues with 50% U-turns)
Capacity Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
10
20
30
40
50
60
Average Conflicting Right Turn Volume (vph)

Appendix I. Hypothetical Best and Worse Cases for U-Turn Grouping

For Queues of 20% U-Turns								
Site	Avg Measured Headway	"Best" Case Headway	"Worst" Case Headway	Ideal Headway	Proportion of Measured Headway	Proportion of Best	Proportion of Worst	Difference in Best and Worst Proportions
202	2.11	1.99	2.15	2.05	1.03	0.97	1.05	0.08
203	2.07	2.05	2.19	2.01	1.03	1.02	1.09	0.07
204	2.17	2.22	2.25	2.25	0.96	0.99	1.00	0.01
205	2.05	1.80	2.06	1.74	1.18	1.03	1.18	0.15
207	2.03	1.92	2.18	1.94	1.05	0.99	1.13	0.13
210	2.36	2.15	2.69	2.18	1.08	0.99	1.24	0.25
211	2.30	2.15	2.31	2.16	1.06	0.99	1.07	0.07
212	2.09	1.98	2.12	1.95	1.07	1.01	1.09	0.07
213	2.17	2.06	2.27	2.07	1.05	1.00	1.10	0.10
215	2.10	2.05	2.38	2.09	1.00	0.98	1.14	0.16
216	2.09	1.96	2.09	1.98	1.06	0.99	1.06	0.07
217	1.93	2.24	2.29	2.24	0.86	1.00	1.02	0.02
218	2.31	2.05	2.40	2.04	1.13	1.00	1.18	0.17

For Queues of 50% U-Turns								
Site	Avg Measured Headway	"Best" Case Headway	"Worst" Case Headway	Ideal Headway	Proportion of Measured Headway	Proportion of Best	Proportion of Worst	Difference in Best and Worst Proportions
202	2.00	2.11	2.35	2.05	0.98	1.03	1.15	0.12
204	2.34	2.23	2.55	2.25	1.04	0.99	1.13	0.14
205	2.38	2.04	2.12	1.74	1.37	1.17	1.22	0.05
207	2.37	2.11	2.73	1.94	1.22	1.09	1.41	0.32
210	3.38	2.37	4.31	2.18	1.55	1.09	1.98	0.89
211	2.44	2.28	2.57	2.16	1.13	1.05	1.19	0.13
212	2.27	2.07	2.35	1.95	1.16	1.06	1.20	0.14
213	2.27	2.20	2.57	2.07	1.10	1.06	1.24	0.18
215	2.31	2.26	2.61	2.09	1.10	1.08	1.25	0.17
216	2.24	2.05	2.52	1.98	1.13	1.04	1.27	0.24
217	2.44	2.21	2.38	2.24	1.09	0.99	1.06	0.08

Appendix J. Safety Study Site Information

	Site and Approach Location					Collisions				Intersection Information: AM Peak Turn Movement (vph) 07:30-08:30		Intersection Information: PM Peak Turn Movement (vph) 17:00-18:00				
Study No.	Main Rd	Dir	Cross St	County	City	Angle	Sideswipe	Rear-end	No. Uturn Collisions	AM Left Turn	AM Opposing Right Turn	PM Left Turn	PM Opposing Right Turn	Main Rd ADT	LT Signal Type	Conflicting RT
001	Creedmoor Rd	NB	Lynn Rd	Wake	Raleigh	0	2	0	2	150	301	440	115	30600	prot	prot
002	Glenwood Ave	EB	T.W. Alexander	Wake	Raleigh	3	0	1	4					40000	perm	none
003	New Bern Ave	WB	Sunnybrook Rd	Wake	Raleigh	3	1	1	5	781	262	700	913	32000	prot	prot
006	US 74 (Independence Blvd)	WB	Sam Newell Rd	Mecklenburg	Matthews	0	0	0	0					49500	prot	prot
007	US 29 (North Tryon St)	NB	Harris Blvd	Mecklenburg	Charlotte	2	3	4	9	385	255	409	261	25000	prot	prot
009	Harris Blvd	SB	Hickory Grove Rd	Mecklenburg	Charlotte	0	0	0	0	93	120	197	140	40000	prot	prot
010	US 74 (Independence Blvd)	EB	Matthews-Mint Hill Rd	Mecklenburg	Matthews	0	0	0	0					49500	prot	prot
011	I-277		4th St	Mecklenburg	Charlotte	0	2	1	3					-	prot	none
012	Silas Creek Pkwy	NB	Yorkshire	Forsyth	Winston-Salem	0	0	0	0					52100	prot	none (prohib)
014	US 321	SB	SR 1109 (Pinewood Rd)	Caldwell	Granite Falls	1	0	0	1					30200	prot	perm
015	US 321	SB	SR 1108 (Mission Rd)	Caldwell	Hudson	0	0	0	0					30200	prot	perm
016	US 321	SB	Mount Herman Rd	Caldwell	Hudson	0	0	0	0					29000	prot	perm
028	Reynolda Rd	SB	Polo Rd	Forsyth	Winston-Salem	0	0	0	0					22000	prot	prot
029	Silas Creek Pkwy	WB	Miller	Forsyth	Winston-Salem	0	0	0	0					30000	prot	prot
033	Eastway Dr	SB	Frontenac Ave / Shamrock Dr	Mecklenburg	Charlotte	4	0	2	6	237	569	639	343	38100	prot	prot
034	Elizabeth Ave	WB	Kings Dr	Mecklenburg	Charlotte	4	0	0	4	41	184	45	181	15200	perm	perm
038	University City Blvd (NC 49)	SB	Harris ramp / Chancellor Park Dr	Mecklenburg	Charlotte	0	0	0	0	199	129	269	208	34600	prot	none (chan)
039	1/15/501/NC 211 (N Sandhills Blvd)	SB	US 15/501/ NC 211	Moore	Aberdeen	0	0	0	0					24300	prot/perm	none (chan)
040	US 15/501 / NC 211	NB	Johnson St	Moore	Aberdeen	0	0	0	0					15800	perm	perm
042	US 52 Byp / Andy Griffith Pkwy	SB	Snowhill / Worth St	Surry	Mt. Airy	0	0	0	0					20700	prot	perm
046	US 70 (US70A?)	WB	NC 581	Wayne	Goldsborough	0	0	0	0					26700	prot	prot
051	Randall Pkwy	WB	Independence Blvd	New Hanover	Wilmington	0	0	0	0	456	569	567	550	29000	prot	perm
053	Shipyard Blvd	EB	17th St	New Hanover	Wilmington	0	0	0	0	193	112	174	183	24900	prot	none (chan)
054	US 15/501/NC211	SB	US 1/15/501/NC 211 (N Sandhills Blv	Moore	Aberdeen	0	0	0	0					15800	prot	prot
101	US 70	WB	Ebenezer Church	Wake	Raleigh	0	0	0	0	18	35	38	51	43800	prot	perm
102	US 70	EB	Pinecrest	Wake	Raleigh	0	0	0	0	46	127	79	67	43800	prot	perm
103	US 64	EB	MacKenan/Chalon	Wake	Cary	0	0	0	0					28800	prot/perm	perm
104	US 64	EB	Gregson	Wake	Cary	0	0	0	0					28800	perm	none
105	US 64	EB	Lake Pine	Wake	Cary	0	0	0	0					28800	prot	prot
105	US 64	WB	Lake Pine	Wake	Cary	0	0	0	0					28800	prot	perm
106	US 401	NB	Hilltop-Needmore/Air Park	Wake	Garner	0	0	0	0					21900	prot	perm
107	US 15-501		Estes	Orange	Chapel Hill	0	0	0	0					37400	prot	prot
108	US 15-501		Manning	Orange	Chapel Hill	0	0	0	0					49500	prot	perm
109	US 421	NB	George Anderson/Echo Farms	New Hanover	Wilmington	0	0	0	0	110	52	30	35	21500	prot/perm	prot
110	S. College	NB	Pinecliff	New Hanover	Wilmington	0	0	0	0	69	45	10	10	30100	prot/perm	prot
110	S. College	SB	Pinecliff	New Hanover	Wilmington	0	0	0	0	1	8	11	2	30100	prot/perm	perm
111	S. College	NB	Pine Valley	New Hanover	Wilmington	0	0	0	0	20	10	18	15	30100	prot/perm	perm
111	S. College	SB	Pine Valley	New Hanover	Wilmington	0	0	0	0	27	209	113	65	30100	prot/perm	prot
112	S. College	NB	Holly Tree	New Hanover	Wilmington	1	0	0	1	312	102	140	321	30100	prot/perm	prot
113	S. College	SB	Bragg	New Hanover	Wilmington	0	0	0	0	26	49	51	12	30100	prot/perm	perm
113	S. College	NB	Bragg	New Hanover	Wilmington	0	0	0	0	51	42	31	54	30100	prot/perm	perm
114	S. College	SB	17th	New Hanover	Wilmington	0	0	0	0	44	252	187	57	30100	prot	prot
115	SR 4000 (University Parkway)	NB	US 52	Forsyth	Winston-Salem	0	0	0	0					27000	prot	none (chan)
117	NC 67 / Silas Creek Pkwy		Reynolda	Forsyth	Winston-Salem	0	0	0	0					46500	perm	none
118	NC 67 / Silas Creek Pkwy		Lockland	Forsyth	Winston-Salem	0	0	0	0					30000	prot	perm
119	US 70 (Arendell St)	EB	35th	Carteret	Morehead	0	0	0	0					32100	prot	perm
120	US 29/ US 601	NB	Fairview	Cabarrus	Kannapolis	0	0	0	0					21400	prot	prot
121	US 29	NB	Centergrove (Dale Erdt)	Cabarrus	Kannapolis	1	0	0	1					23800	prot	perm
122	US 29/ US 601	NB	Warren C. Coleman	Cabarrus	Concord	0	0	0	0					30000	prot/perm	perm
123	US 29/ US 601	SB	Warren C. Coleman	Cabarrus	Concord	0	0	0	0					30000	prot	perm

										Intersection Information						
	Site and Approach Location					Collisions				AM Peak Turn Movement (vph) 07:30-08:30		PM Peak Turn Movement (vph) 17:00-18:00				
Study No.	Main Rd	Dir	Cross St	County	City	Angle	Sideswipe	Rear-end	No. Uturn Collisions	AM Left Turn	AM Opposing Right Turn	PM Left Turn	PM Opposing Right Turn	Main Rd ADT	LT Signal Type	Conflicting RT
124	US 29	WB	Rock Hill Church	Cabarrus	Concord	0	0	0	0					29700	prot	perm
125	US 29	NB	Minnie	Cabarrus	Concord	1	0	0	1					37000	perm	perm
126	US 29/ US 601	NB	McGill/Poplar Tent	Cabarrus	Concord	0	0	0	0					37000	prot	perm
127	US 29/ US 601	SB	McGill/Poplar Tent	Cabarrus	Concord	0	0	0	0					37000	prot	prot
128	US 29	EB	Cabarrus	Cabarrus	Concord	0	0	0	0					29700	prot/perm	perm
129	US 29	WB	Cabarrus	Cabarrus	Concord	0	0	0	0					29700	prot/perm	perm
130	US 74 Bus	EB	Clemmer	Richmond	Rockingham	0	0	0	0					21700	prot/perm	perm
131	NC 51	EB	Beverly Crest/Hugh Forest	Mecklenburg	Charlotte	0	0	0	0	10	22	23	16	29200	perm	perm
132	US 29	SB	McCullough	Mecklenburg	Charlotte	0	0	0	0	5	10	24	21	35300	prot	prot
133	US 29	NB	McCullough	Mecklenburg	Charlotte	1	0	2	3	452	266	357	656	35300	prot	prot
135	Providence Rd	SB	NC 51	Mecklenburg	Charlotte	0	0	0	0	106	76	105	68	30500	prot	none (chan)
136	NC 49 (University City Blvd)	NB	Suther / Broadrick	Mecklenburg	Charlotte	0	0	0	0	215	29	130	105	34600	prot	perm
137	NC 51	EB	Echo Forest	Mecklenburg	Charlotte	0	0	0	0	18	83	19	51	29200	prot/perm	prot
138	US 29	NB	Craighead	Mecklenburg	Charlotte	0	0	0	0	52	78	79	45	35300	prot	perm
139	US 29	SB	Tom Hunter	Mecklenburg	Charlotte	0	0	0	0	22	39	53	53	32500	prot	perm
140	NC 16 (Providence)	SB	Wendover	Mecklenburg	Charlotte	0	0	0	0	60	120	85	62	29200	prot	perm
141	NC 279 / New Hope		Pearl	Gaston	Gastonia	0	0	0	0					24000	prot	none (prohib)
142	US 64	NE	Sugarloaf / Francis	Henderson	Hendersonville	0	0	0	0					22600	prot	perm
143	US 70	SE	Amelia Church/Robertson	Johnston	Clayton	0	0	0	0					41500	prot	perm
144	US 70	NW	Shotwell	Johnston	Clayton	0	0	0	0					41600	prot	perm
145	US 301	NB	Stone Rose	Nash	Rocky Mount	1	0	0	1					46300	prot	perm
146	US 301	SB	Old Mill / May	Nash	Rocky Mount	0	0	0	0					46300	prot	perm
147	US 17 / Marine	SW	McDaniel / Workshop / Ramada	Onslow	Jacksonville	0	0	0	0					33000	prot	perm
147	US 17 / Marine	NE	McDaniel / Workshop / Ramada	Onslow	Jacksonville	0	0	0	0					33000	prot	perm
148	US 17 / Marine	SW	Western	Onslow	Jacksonville	0	0	0	0					32100	prot	perm
149	US 64 / Asheville		Ecusta	Transylvania	Brevard	0	0	0	0					24500	prot	none
150	US 264 Alt / Ward	SB	Black Creek	Wilson	Wilson	0	0	0	0					26400	prot	perm
151	US 264 Alt / Ward	NB	New Bern	Wilson	Wilson	0	0	0	0					26400	prot	perm

	Intersection Information								Segment Information									
					Width of Receiving Lanes									Land Use				
Study No.	No. LT lanes	LT Lane Width	Median Width	No. Lns Rcvg	Inside Lane	Next Lane Over	Shoulder or Extra Space	Total Rcvg Width	Segment Beginning	Segment Length (mi)	Public St Approaches	Driveways	Total Access Points	Residential	Office	Business	Industrial	Speed Limit
001	2	10	3	2	11	11	12	34	-	0.3	0	8	8	50	0	50	0	45
002	1	11	15	2	12	12	2	26	-	0.4	0	1	1	0	0	0	0	55
003	2	11	13	2	11	13	12	36	Yonkers	0.3	1	0	1	0	0	0	0	45
006	1	13	12	2	13	13	0	26	Windsor Sq	0.25	0	1	1	0	0	50	0	45
007	2	10	7	2	12	11	6	29	McCullough	0.4	1	4	5	0	0	100	0	45
009	1	11	3	2	12	14	5	31	Susan	0.2	0	4	4	30	0	70	0	45
010	1	12	12	2	12	13	0	25	Windsor Sq	1.3	0	12	12	0	0	70	0	45
011	2	11	4	2	13	12	8	33	-	-	-	-	-	-	-	-	-	-
012	1	12	4	2	12	12	0	24	Tiseland	0.75	3	1	4	30	0	0	0	45
014	1	12	10	2	12	12	0	24	-	0.75	2	1	3	10	0	0	0	55
015	1	12	8	2	12	12	0	24	-	0.8	1	4	5	10	0	50	0	55
016	1	12	10	2	12	12	0	24	-	0.5	1	3	4	20	0	20	0	55
028	1	12	7	2	12	12	0	24	Fairlawn	0.5	0	14	14	90	10	0	0	45
029	1	12	3	2	12	12	3	27	-	-	-	-	-	-	-	-	-	-
033	2	12	4	2	12	12	0	24	-	0.1	0	2	2	0	0	100	0	45
034	1	12	2	2	13	13	6	32	-	0.25	1	0	1	0	100	0	0	25
038	1	12	6	2	12	12	3	27	Suther / Bro	0.25	3	7	10	50	0	15	0	45
039	1	12	4	2	12	12	2	26	-	0.25	0	4	4	0	0	100	0	45
040	1	12	10	2	12	12	3	27	N Sandhill E	0.1	0	1	1	0	0	100	0	45
042	1	12	40	2	12	12	0	24	Bluemont	0.55	0	3	3	0	0	20	0	45
046	1	12	15	2	12	12	3	27	-	0.4	0	7	7	50	0	50	0	55
051	2	12	3	2	11	10	0	21	-	0.2	0	2	2	0	50	50	0	35
053	2	12	6	2	12	12	8	32	-	0.15	0	2	2	0	0	100	0	50
054	2	12	5	2	12	12	8	32	-	0.1	0	5	5	0	0	100	0	45
101	1	12	30	2	12	12	0	24	Pinecrest	1.08	1	2	3	0	0	30	0	45
102	1	12	30	2	12	12	0	24	Ebenezer C	1.08	1	10	11	0	0	70	0	45
103	1	12	46	2	12	12	0	24	Lake Pine	0.59	1	0	1	0	0	10	0	45
104	1	12	46	2	12	12	0	24	MacKenan/(	0.29	0	0	0	0	0	0	0	45
105	1	12	46	2	12	12	0	24	Knollwood	0.57	0	1	1	50	0	0	0	45
105	1	12	46	2	12	12	0	24	MacKenan/(	0.59	0	0	0	0	0	10	0	45
106	1	12	30	2	12	12	0	24	Dwight Rola	1.07	0	16	16	90	0	10	0	45
107	2	12	10	2	12	12	0	24	54	1.12	3	0	3	100	0	0	0	45
108	2	12	21	2	12	12	0	24	Morgan Cre	0.75	2	0	2	20	0	0	0	45
109	1	12	36	2	12	12	0	24	St. Andrews	0.99	0	1	1	50	0	30	0	45
110	1	12	30	2	12	12	0	24	Tall Tree	0.3	1	2	3	50	0	50	0	45
110	1	12	30	2	12	12	0	24	17th	0.39	1	2	3	50	0	50	0	45
111	1	12	30	2	12	12	0	24	Bragg	0.41	0	14	14	50	0	50	0	45
111	1	12	30	2	12	12	0	24	Holly Tree	0.55	1	9	10	50	0	50	0	45
112	1	12	30	2	12	12	0	24	Pine Valley	0.55	3	0	3	50	0	50	0	45
113	1	12	30	2	12	12	0	24	Pine Valley	0.41	0	8	8	50	0	50	0	45
113	1	12	30	2	12	12	0	24	17th	0.38	2	2	4	50	0	50	0	45
114	1	12	30	2	12	12	0	24	Bragg	0.38	0	8	8	50	0	50	0	45
115	1	11	16	2	11	11	0	22	Robin Wooc	0.3	1	1	2	100	0	0	0	45
117	1	12	32	2	12	12	0	24	Robin Wooc	1.59	0	0	0	30	0	0	0	45
118	1	12	2	2	12	12	0	24	Irving	0.57	0	16	16	0	0	100	0	35
119	1	12	42	2	12	12	0	24	Wallace	0.4	6	18	24	10	0	90	0	35
120	1	12	14	2	12	12	0	24	Delane	0.4	2	5	7	0	0	100	0	45
121	1	12	33	2	12	12	0	24	Eddleman	0.25	1	4	5	0	0	100	0	45
122	1	12	24	2	12	12	0	24	Cabarrus	0.52	1	3	4	0	0	100	0	45
123	2	12	40	2	12	12	0	24	McGill	0.88	2	14	16	0	0	100	0	45

	Intersection Information								Segment Information									
					Width of Receiving Lanes									Land Use				
Study No.	No. LT lanes	LT Lane Width	Median Width	No. Lns Rcvg	Inside Lane	Next Lane Over	Shoulder or Extra Space	Total Rcvg Width	Segment Beginning	Segment Length (mi)	Public St Approaches	Driveways	Total Access Points	Residential	Office	Business	Industrial	Speed Limit
124	1	12	24	2	12	12	0	24	Cabarrus	0.25	0	4	4	0	0	100	0	45
125	1	12	48	2	12	12	0	24	McGill	1	4	17	21	0	0	100	0	45
126	1	12	40	2	12	12	0	24	Warren C. C	0.88	2	7	9	0	0	100	0	45
127	1	12	48	2	12	12	0	24	Minnie	1	5	21	26	0	0	100	0	45
128	1	12	24	2	12	12	0	24	Rock Hill Ch	0.25	0	1	1	0	0	100	0	45
129	1	12	24	2	12	12	0	24	Warren C. C	0.52	0	0	0	0	0	100	0	45
130	1	11	30	2	11	11	0	22	Elizabeth	0.38	0	4	4	0	0	100	0	45
131	1	12	5	2	12	12	0	24	Arboretum D	0.67	2	0	2	100	0	0	0	45
132	2	12	3	2	12	12	0	24	Harris	0.4	1	10	11	0	0	100	0	45
133	2	12	3	2	12	12	0	24	University C	0.8	4	22	26	10	0	90	0	45
135	2	12	4	2	12	12	0	24	Beverly Cre	0.58	3	4	7	70	10	0	0	45
136	1	12	18	2	12	12	0	24	Harris ramp	0.25	3	7	10	50	0	15	0	45
137	1	12	5	2	12	12	0	24	Beverly Cre	0.4	1	1	2	100	0	0	0	45
138	1	11	4	2	11	11	0	22	36th	0.39	1	21	22	0	0	90	0	45
139	1	12	24	2	12	12	0	24	Kemp	0.5	1	8	9	0	0	100	0	45
140	1	11	14	2	11	11	0	22	Vernon	0.39	3	3	6	100	0	0	0	45
141	1	12	4	2	12	12	0	24	Franklin	0.29	0	4	4	50	0	50	0	40
142	1	12	18	2	12	12	0	24	I-26 ramp	0.36	0	0	0	0	0	0	0	45
143	1	12	22	2	12	12	0	24	Shotwell	0.68	1	10	11	0	0	100	0	45
144	1	12	22	2	12	12	0	24	Amelia Chu	0.68	2	11	13	0	0	100	0	45
145	1	12	32	2	12	12	0	24	Old Mill / Ma	0.61	0	1	1	0	0	50	0	45
146	1	12	32	2	12	12	0	24	Stone Rose	0.61	0	1	1	0	0	50	0	45
147	1	12	8	2	12	12	0	24	Sunset	0.6	2	7	9	0	0	0	0	45
147	1	12	8	2	12	12	0	24	Western	0.37	1	2	3	0	0	100	0	45
148	1	12	8	2	12	12	0	24	McDaniel	0.37	0	2	2	0	0	100	0	45
149	1	12	30	2	12	12	0	24	Morris	0.7	2	13	15	0	0	100	0	45
150	1	12	31	2	12	12	0	24	New Bern	0.33	2	5	7	40	0	60	0	45
151	1	12	31	2	12	12	0	24	Black Creek	0.33	2	5	7	40	0	60	0	45

Appendix K. Statistical Tests for Safety Factors

Effect of Right Turn Overlap

Observed

	No Collisions	1+ Collisions	Sample	Percent of Total
RT overlap	17	6	23	0.30
No RT overlap	48	6	54	0.70
	65	12	**77**	

Expected

	No Collisions	1+ Collisions
RT overlap	19	4
No RT overlap	46	8
	65	12

Chi-Square p-value = 0.097

Fisher's Exact right-sided Pr>=F p-value = 0.097

At 90% conf, reject H0 and conclude that RT overlap has some signif impact.

Effect of Left Turn Signal Type

Observed

	No Collisions	1+ Collisions	Sample	Percent of Total
LT perm	4	3	7	0.09
LT prot	47	8	55	0.71
LT perm/prot	14	1	15	0.19
	65	12	**77**	

Expected

	No Collisions	1+ Collisions
LT perm	6	1
LT prot	46	9
LT perm/prot	13	2
	65	12

Chi-Square p-value = 0.086

Fisher's Exact Pr<=P p-value = 0.1286

Cannot reject H0 of independence. LT treatment has no signif impact.

Effect of Number of Left-Turn Lanes

Observed

	No Collisions	1+ Collisions	Sample	Percent of Total
1 LT Lane	57	7	64	0.83
2 LT Lanes	8	5	13	0.17
	65	12	0	0.00
			77	

Expected

	No Collisions	1+ Collisions
1 LT Lane	54	10
2 LT Lanes	11	2
	65	12

Chi-Square p-value = 0.013

Fisher's Exact right-sided Pr>=F p-value = 0.0256

Reject H0 of independence. Number of LT lanes has some signif impact.

No. Uturn Collisions	AM Left Turn	AM Opposing Right Turn	PM Left Turn	PM Opposing Right Turn
9	385	255	409	261
6	237	569	639	343
5	781	262	700	913
4	41	184	45	181
3	452	266	357	656
2	150	301	440	115
1	312	102	140	321
0	456	569	567	550
0	215	29	130	105
0	199	129	269	208
0	193	112	174	183
0	110	52	30	35
0	106	76	105	68
0	93	120	197	140
0	69	45	10	10
0	60	120	85	62
0	52	78	79	45
0	51	42	31	54
0	46	127	79	67
0	44	252	187	57
0	27	209	113	65
0	26	49	51	12
0	22	39	53	53
0	20	10	18	15
0	18	35	38	51
0	18	83	19	51
0	10	22	23	16
0	5	10	24	21
0	1	8	11	2

T-test p-value =	0.0002	0.0018	0.0001	0.0001
Wilcoxon Rank Sum test p-value =	0.0047	0.0019	0.0047	0.0007

Access Points					
Groups	Sample Size	Average No. Access Points	Standard Deviation	T-test P-value	Wilcoxon Rank Sum P-value
One or more collisions	12	6.42	8.3		
Zero collisions	65	6.46	6.0	0.49	0.26
Main Road ADT					
Groups	Sample Size	Average ADT (veh)	Standard Deviation	T-test P-value	Wilcoxon Rank Sum P-value
One or more collisions	12	31966	8250.0		
Zero collisions	65	31490	8028.0	0.42	0.19
Median Width					
Groups	Sample Size	Average Median Width (ft)	Standard Deviation	T-test P-value	Wilcoxon Rank Sum P-value
One or more collisions	12	16.7	15.3		
Zero collisions	65	20.7	13.8	0.18	0.17

Appendix L. SAS Output for Wilcoxon Rank Sum Tests

Wilcoxon Rank Sum Test Output for Significance of AM Peak Left Turns

```
The SAS System    17:26 Thursday, February 19, 2004   2

                         The NPAR1WAY Procedure

              Wilcoxon Scores (Rank Sums) for Variable AMLeft
                       Classified by Variable group

                          Sum of      Expected       Std Dev          Mean
group        N            Scores      Under H0      Under H0         Score
--------------------------------------------------------------------------
y            7             161.0         105.0     19.619000     23.000000
n           22             274.0         330.0     19.619000     12.454545

                    Average scores were used for ties.


                         Wilcoxon Two-Sample Test

                    Statistic             161.0000

                    Normal Approximation
                    Z                       2.8289
                    One-Sided Pr >  Z       0.0023
                    Two-Sided Pr > |Z|      0.0047

                    t Approximation
                    One-Sided Pr >  Z       0.0043
                    Two-Sided Pr > |Z|      0.0085

                Z includes a continuity correction of 0.5.


                          Kruskal-Wallis Test

                    Chi-Square              8.1475
                    DF                           1
                    Pr > Chi-Square         0.0043
```

Wilcoxon Rank Sum Test Output for Significance of AM Peak Conflicting Right Turns

The SAS System 17:26 Thursday, February 19, 2004 3

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable **AMRight**
Classified by Variable group

group	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
y	7	166.50	105.0	19.614166	23.785714
n	22	268.50	330.0	19.614166	12.204545

Average scores were used for ties.

Wilcoxon Two-Sample Test

Statistic	166.5000
Normal Approximation	
Z	3.1100
One-Sided Pr > Z	0.0009
Two-Sided Pr > |Z|	0.0019
t Approximation	
One-Sided Pr > Z	0.0021
Two-Sided Pr > |Z|	0.0043

Z includes a continuity correction of 0.5.

Kruskal-Wallis Test

Chi-Square	9.8313
DF	1
Pr > Chi-Square	0.0017

Wilcoxon Rank Sum Test Output for Significance of PM Peak Left Turns

The SAS System   17:26 Thursday, February 19, 2004   4

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable **PMLeft**
Classified by Variable group

group	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
y	7	161.0	105.0	19.619000	23.000000
n	22	274.0	330.0	19.619000	12.454545

Average scores were used for ties.

Wilcoxon Two-Sample Test

Statistic	161.0000
Normal Approximation	
Z	2.8289
One-Sided Pr > Z	0.0023
Two-Sided Pr > |Z|	0.0047
t Approximation	
One-Sided Pr > Z	0.0043
Two-Sided Pr > |Z|	0.0085

Z includes a continuity correction of 0.5.

Kruskal-Wallis Test

Chi-Square	8.1475
DF	1
Pr > Chi-Square	0.0043

Wilcoxon Rank Sum Test Output for Significance of PM Peak Conflicting Right Turns

The SAS System   17:26 Thursday, February 19, 2004   5

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable PMRight
Classified by Variable group

group	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
y	7	172.0	105.0	19.619000	24.571429
n	22	263.0	330.0	19.619000	11.954545

Average scores were used for ties.

Wilcoxon Two-Sample Test

Statistic	172.0000
Normal Approximation	
Z	3.3896
One-Sided Pr > Z	0.0004
Two-Sided Pr > |Z|	0.0007
t Approximation	
One-Sided Pr > Z	0.0010
Two-Sided Pr > |Z|	0.0021

Z includes a continuity correction of 0.5.

Kruskal-Wallis Test

Chi-Square	11.6626
DF	1
Pr > Chi-Square	0.0006

Wilcoxon Rank Sum Test Output for Significance of Median Width

```
The SAS System     13:12 Thursday, January 22, 2004   7

                              The NPAR1WAY Procedure

                   Wilcoxon Scores (Rank Sums) for Variable width
                            Classified by Variable group

                            Sum of      Expected       Std Dev          Mean
group          N            Scores      Under H0      Under H0         Score
______________________________________________________________________________
zero          65            2603.0        2535.0     70.966740     40.046154
crash         12             400.0         468.0     70.966740     33.333333

                         Average scores were used for ties.


                              Wilcoxon Two-Sample Test

                           Statistic             400.0000

                           Normal Approximation
                           Z                      -0.9511
                           One-Sided Pr <  Z       0.1708
                           Two-Sided Pr > |Z|      0.3415

                           t Approximation
                           One-Sided Pr <  Z       0.1723
                           Two-Sided Pr > |Z|      0.3445

                      Z includes a continuity correction of 0.5.


                                Kruskal-Wallis Test

                           Chi-Square              0.9181
                           DF                           1
                           Pr > Chi-Square         0.3380
```

Wilcoxon Rank Sum Test Output for Significance of Main Road ADT

```
The SAS System       08:56 Monday, January 26, 2004   5

                              The NPAR1WAY Procedure

                     Wilcoxon Scores (Rank Sums) for Variable adt
                             Classified by Variable group

                         Sum of      Expected       Std Dev          Mean
group          N         Scores      Under H0      Under H0         Score
_________________________________________________________________________
crash         12          531.0         468.0     71.141196     44.250000
zero          65         2472.0        2535.0     71.141196     38.030769

                          Average scores were used for ties.


                             Wilcoxon Two-Sample Test

                           Statistic             531.0000

                           Normal Approximation
                           Z                       0.8785
                           One-Sided Pr >  Z       0.1898
                           Two-Sided Pr > |Z|      0.3797

                           t Approximation
                           One-Sided Pr >  Z       0.1912
                           Two-Sided Pr > |Z|      0.3824

                     Z includes a continuity correction of 0.5.


                               Kruskal-Wallis Test

                           Chi-Square              0.7842
                           DF                           1
                           Pr > Chi-Square         0.3759
```

Wilcoxon Rank Sum Test Output for Significance of Number of Access Points

```
The SAS System      08:56 Friday, February 20, 2004   1

                          The NPAR1WAY Procedure

              Wilcoxon Scores (Rank Sums) for Variable access
                        Classified by Variable group

                          Sum of      Expected       Std Dev          Mean
group          N          Scores      Under H0      Under H0         Score
--------------------------------------------------------------------------
crash         12           421.0         468.0     70.924938     35.083333
nocrash       65          2582.0        2535.0     70.924938     39.723077

                    Average scores were used for ties.


                         Wilcoxon Two-Sample Test

                     Statistic             421.0000

                     Normal Approximation
                     Z                      -0.6556
                     One-Sided Pr <  Z       0.2560
                     Two-Sided Pr > |Z|      0.5121

                     t Approximation
                     One-Sided Pr <  Z       0.2570
                     Two-Sided Pr > |Z|      0.5140

                Z includes a continuity correction of 0.5.


                           Kruskal-Wallis Test

                     Chi-Square              0.4391
                     DF                           1
                     Pr > Chi-Square         0.5075
```

Appendix M. Comparison of Conflict and Collision Data

	U-Turn Conflicts per Hour				U-Turn Collisions				
Site No.	Left Turn Same Direction per hour	RT and U-turn per hour	Near Sideswipe per hour	Total conflicts per Hour	Left Turn Same Direction	RT and U-turn	Sideswipe	Total Collisions (3 yrs)	U-turn Collisions per year
202	0.6	0.1	0.0	**0.7**	0	0	0	0	0.0
203	0.2	0.2	0.0	**0.3**	0	0	0	0	0.0
204	0.1	0.0	0.0	**0.1**	0	0	0	0	0.0
205	0.0	0.2	0.0	**0.2**	0	0	0	0	0.0
206	1.1	0.0	0.2	**1.2**	1	0	2	3	1.0
207	1.9	0.8	0.2	**2.9**	3	2	3	9	3.0
210	1.4	1.2	0.0	**2.8**	1	3	1	5	1.7
211	0.2	0.2	0.0	**0.3**	0	0	0	0	0.3
212	0.0	0.0	0.0	**0.0**	0	0	0	0	0.0
213	1.0	0.2	0.2	**1.3**	0	0	0	0	0.0
215	1.5	0.2	0.3	**2.0**	2	0	1	3	1.0
216	0.0	0.3	0.0	**0.3**	0	0	0	0	0.0
217*	0.2	0.0	0.0	**0.2**	-	-	-	-	-
218	0.5	0.0	0.0	**0.5**	1	0	2	3	1.0

* Recent construction at site 217 precluded the collection of reliable collision data.