EFFECTS OF INCREASED U-TURNS AT INTERSECTIONS ON DIVIDED FACILITIES AND MEDIAN DIVIDED VERSUS FIVE-LANE UNDIVIDED BENEFITS

Research Conducted for
The North Carolina Department of Transportation

By

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16. Abstract

Highway projects involving access management strategies are among the most hotly debated transportation issues. In particular, the choice of midblock left turn treatment is a controversial issue. The two main competitors for midblock left-turn treatment on four-lane arterials are raised medians and two-way left-turn lanes (TWLTL). This research focused on determining the effects of median installation on midblock road segments and the adjacent signalized intersections. The areas of focus were vehicular safety and operational impacts.

For the segment safety study, predictive collision models were calibrated using geometric, volume, land use, and collision data for 143 midblock segments. Analysis showed that collisions were significantly related to cross-section type, average daily traffic (AADT), segment length, predominant land use, and approach density (two-way total). For predominantly residential and industrial land uses, the raised median design was always associated with fewer collisions than the TWLTL design. For predominantly business and office land uses, the raised median design had a safety advantage for low approach densities. For higher driveway densities, the raised median was slightly safer at high traffic volumes and the TWLTL was slightly safer at lower traffic volumes.

The signalized intersection study dealt with the effects of U-turns in exclusive left turn lanes. This included analyses of the safety of U-turns and the operational impacts of U-turns on saturation flow rate. The safety study examined a set of 78 intersections in North Carolina, one-third of which were chosen because they were known to be 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.

The intersection operational analysis involved measurements of vehicle headways in exclusive left turn lanes at 14 intersections. Regression analysis of U-turn percentage versus saturation flow rate 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.

Overall, this research found that many of the typically cited drawbacks to median-oriented designs are not justified. Raised medians may increase U-turns at adjacent intersections, but this was found to have minimal effects on safety and operational performance. Additionally, raised medians are generally safer than TWLTLs on midblock segments.

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SUMMARY

Highway projects involving access management strategies are among the most hotly debated transportation issues. In particular, the choice of midblock left turn treatment is a controversial issue. The two main competitors for midblock left-turn treatment on four-lane arterials are raised medians and two-way left-turn lanes (TWLTL). This research focused on determining the effects of median installation on midblock road segments and the adjacent signalized intersections. The areas of focus were vehicular safety and operational impacts.

For the segment safety study, predictive collision models were calibrated using geometric, volume, land use, and collision data for 143 midblock segments. Analysis showed that collisions were significantly related to cross-section type, average daily traffic (AADT), segment length, predominant land use, and approach density (two-way total). For predominantly residential and industrial land uses, the raised median design was always associated with fewer collisions than the TWLTL design. For predominantly business and office land uses, the raised median design had a safety advantage for low approach densities (0-25 approaches per mile). For medium to high driveway densities (25-90 approaches per mile), the raised median was slightly safer at high traffic volumes and the TWLTL was slightly safer at lower traffic volumes.

The signalized intersection study dealt with the effects of U-turns in exclusive left turn lanes. This included analyses of the safety of U-turns and the operational impacts of U-

turns on saturation flow rate. The safety study examined a set of 78 intersections in North Carolina, one-third of which were chosen because they were known to be 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.

The intersection operational analysis involved measurements of vehicle headways in exclusive left turn lanes at 14 intersections. Regression analysis relating U-turn percentage to saturation flow rate 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.

Overall, this research found that many of the typically cited drawbacks to median-oriented designs are not justified. Raised medians may increase U-turns at adjacent intersections, but this was found to have minimal effects on safety and operational performance. Additionally, raised medians are generally safer than TWLTLs on midblock segments.

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INTRODUCTION

BACKGROUND

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. Highway projects that include access management strategies tend to be among the most hotly debated transportation issues with the public. In particular, the choice of midblock left turn treatment is often controversial and generates significant discussion at public hearings. The two main competitors for midblock left turn treatment on four-lane arterials are raised medians and two-way left turn lanes (TWLTL).

A raised median cross-section is depicted in Figure 1, while Figure 2 is a TWLTL roadway segment. Figures 3 and 4 are pictures of typical raised median and TWLTL segments, respectively.

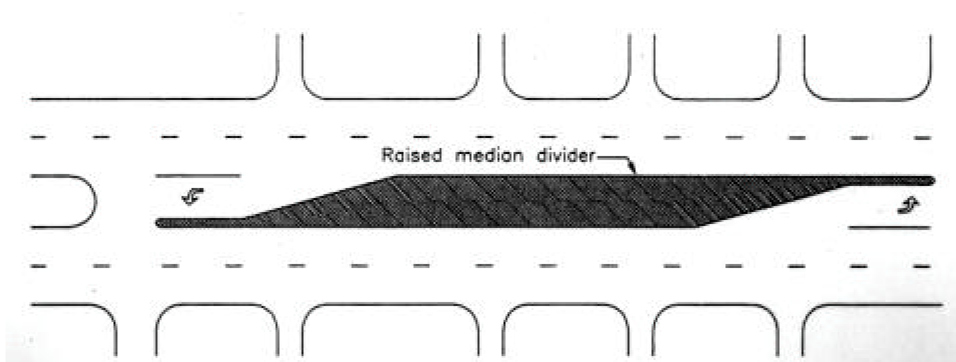


Figure 1. Four-Lane Raised Median Cross-Section [1]

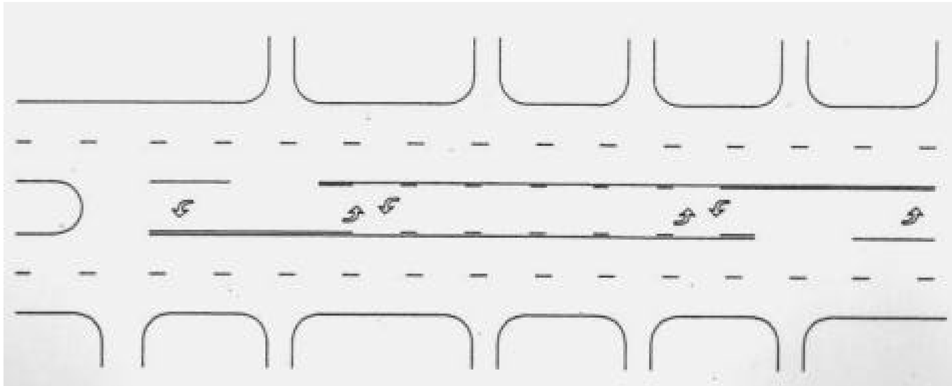


Figure 2. Five-Lane with TWLTL Cross-Section [1]

Figure 3. Segment of Four-Lane Road with Raised Median

Figure 4. Segment of Five-Lane Road with TWLTL

Both of these cross-sections have their advantages. The raised median reduces conflicts by preventing midblock left turns at locations without a median opening and provides a refuge for pedestrians crossing the street. Additionally, a raised median is generally more aesthetically pleasing. The TWLTL treatment tends to be preferred by adjacent land owners and may have some economic benefits by allowing direct left turn access to the arterial for businesses. A TWLTL cross-section generally requires a smaller roadway footprint, generally 3 to 5 feet less on each side of the road, although the right-of-way width typically required stays the same at 100 to 110 feet.

The choice of raised median or TWLTL has created controversy in several road widening projects in North Carolina. As an example, the North Carolina Department of Transportation (NCDOT) planned to widen US 70 in Salisbury from a three-lane undivided to a four-lane median divided highway. While transportation officials supported a median-divided design, the public took strong opposition to the median installation [2]. Those opposed to the median believed that it would pose an obstacle to emergency vehicles and would be more dangerous than the TWLTL cross-section. Business owners were also concerned about a lack of access due to the median. Many seemed to believe that NCDOT was in favor of the median solely for aesthetic reasons and did not believe that there was a safety benefit associated with a median. These views led to petitions with over 2,000 signatures as well as numerous letters, emails, telephone calls, and personal meetings. The chairman of the Rowan County Board of Commissioners said that he was favoring the side of business and convenience until there was a study that compared the safety of left-turns on five lane roads with that of making a similar number of U-turns [3].

Much of the concern over this issue pertains to the performance of the two basic parts of the roadway – the midblock segments and the intersections. Safety performance of the two cross-sections through midblock segments is a major factor in the design decision. The adjacent signalized intersections are examined not only for safety effects, but also for effects on operational performance.

The NCDOT currently uses collision rates on midblock segments to defend their decisions on median treatment issues. These rates are often criticized because they are not broken down by driveway density, land use, or other factors that may play a role in collisions. Additionally, NCDOT cites studies conducted in other states that show raised medians to be safer than TWLTLs. However, there is a tendency among many people to believe that studies conducted in one state will not be valid elsewhere due to differences in driving behavior and design standards. Collision models calibrated in North Carolina which account for differences in driveway density, land use, and other factors would help NCDOT in making future design decisions and better support their actions.

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 as shown in Figure 5.

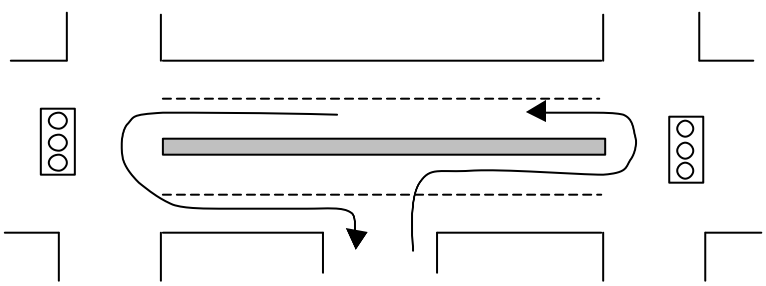


Figure 5. Flow of Traffic with Median on Main Highway

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 already serve a large number of left-turning vehicles. As can be seen in the prohibition 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.

Road planners, designers, and local officials are often faced with the issue of cross-section design. This project seeks to provide solid research to allow them to make informed decisions on this hotly debated topic.

OBJECTIVES

- *To calibrate an empirical collision model for four-lane roadways in North Carolina with raised medians or TWLTLs.* This research will be based on the model developed by Bonneson and McCoy to predict collisions on the two competing cross-sections [1]. These models, however, were calibrated with data from Nebraska and Arizona. Since these models were not calibrated in North Carolina, it is possible that they will not adequately predict collisions on North Carolina's highways due to differences in driving behavior as well as design differences such as median width, median openings, signing, and vegetation in the median.

- *To analyze the operational and safety impacts of U-turns at signalized intersections.*

SCOPE OF SEGMENT RESEARCH

This section of the research was limited to midblock segments on four-lane roadways in North Carolina with either a raised median or a TWLTL. Roadway segments with no median or with a traversable or painted median were not included in this study. No signalized intersections were included in the segments in order to remove the complicating factors that these intersections introduce.

Segments were randomly selected from a NCDOT database. Only those sites that had an AADT greater than or equal to 20,000 vehicles per day (vpd) and were at least ¼ of a mile long were included. Additionally, only segments with a posted speed limit of 35 to 45 miles per hour were chosen.

Sites with an AADT less than 20,000 vpd were not included because access management is typically not a problem on these lower volume roads. A minimum length of ¼ of a mile was selected to prevent complicating factors from adjacent signalized intersections from interfering with the midblock collision model. The 35 to 45 mile per hour speed limit requirement was set to maximize the number of suburban roadway segments. Typically, segments with speed limits less than 35 miles per hour are urban sites and segments with speed limits greater than 45 miles per hour are rural sites. Since the majority of access management issues occur on suburban highways, the range of 35 to 45 miles per hour was selected to capture these sites.

The quantity and type of median openings in the median-divided segments were not analyzed in this study. The spacing of median openings as well as the choice between full access and directional median openings should be considered in any median-divided design. Most, if not all, of the test sections for this research had two-way openings. Some material is given in the literature review from previous studies that may be helpful for these considerations.

Differences between these two cross-section types that do not involve vehicular safety were not directly addressed in this research project but previous research on these issues is summarized in the literature review section that follows.

SCOPE OF INTERSECTION RESEARCH

This section of the research was limited to signalized intersections in the state of North Carolina. All sites had raised medians at the intersection, but no restriction was placed on the median length or width. Study sites were located on either four- or six-lane facilities. The operational research focused only on the performance of passenger vehicles, thereby excluding capacity effects of heavy vehicles. This project only studied 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.

LITERATURE REVIEW

This project is focused on the impacts of cross-section design on midblock segments and signalized intersections. For the segments, this research aims to determine the likely impact, in terms of vehicular collisions, of the choice between a raised median and a TWLTL on a four-lane roadway. For the intersections, the research focuses on vehicular safety as well as operational impact, in terms of saturation flow reduction.

There are, however, many other areas of concern when deciding on median installation. Some of these issues are midblock operational impacts, economic and access impacts, pedestrian safety, and public perception. While these topics will not be investigated explicitly in this study, some previous research on each of these topics can give insight into how the choice of median treatment could affect each of these important areas.

VEHICULAR SAFETY IMPACTS

Impact of Cross-Section Type

Many previous studies have compared various median treatments and levels of access control in terms of their affect on safety. These studies differ dramatically both in terms of sample size used and type of analysis employed.

One of the more comprehensive of these studies, conducted by Bonneson and McCoy, involved the development of empirical models that could be used to evaluate midblock left-turn treatments in terms of operational, safety, and access impacts. The left-turn treatments evaluated were raised-curb median, flush median with TWLTL, and undivided cross-section. For the safety portion of the study, three-year collision histories that were collected for 189 segments (78.6 miles) in Omaha, Nebraska and Phoenix, Arizona were combined with geometric and land use data. Only midblock segments, excluding signalized intersections, were studied. Negative binomial regression was used to model the effect of volume, geometry, and land use characteristics on collision frequency. The factors that were found to have a significant impact on collisions were average daily traffic demand (AADT), segment length, driveway density, unsignalized public street approach density, the percentage of property damage only (PDO) collisions in the area, and the type of adjacent land use. The model form for this study will be discussed in more detail in the methodology portion of this report. The results of this research indicated that the sites with a raised-curb median were associated with fewer collisions that those with a TWLTL. This difference was most pronounced when the AADT of a segment was greater than 20,000 vpd [1].

Figures 6 and 7 illustrate the differences between the three cross-sections analyzed in the Bonneson and McCoy study under a set of typical conditions for business/office and residential/industrial land uses, respectively [1].

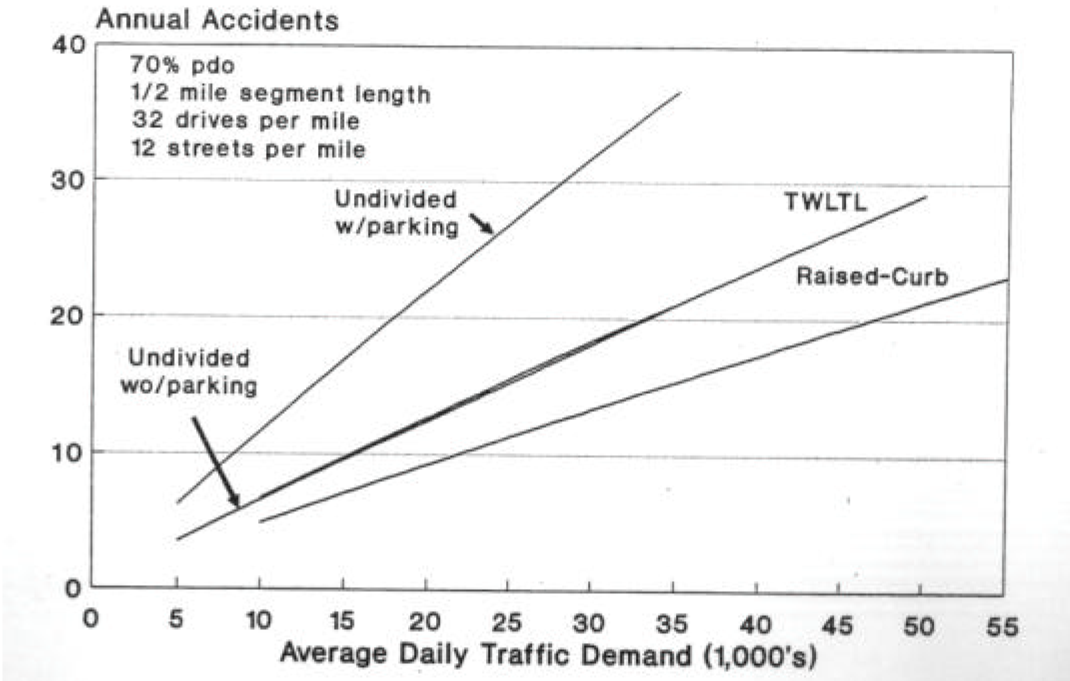


Figure 6. Cross-Section Comparison for Business and Office Land Uses [1]

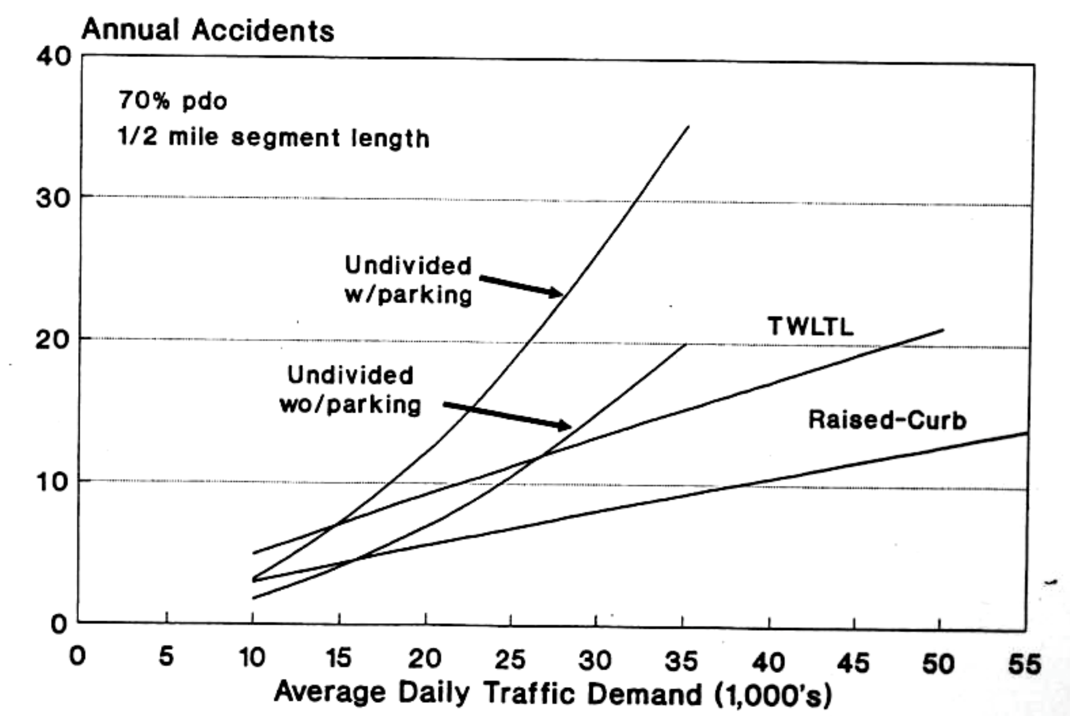


Figure 7. Cross-Section Comparison for Residential and Industrial Land Uses [1]

The Bonneson and McCoy model was one of two models cited in an NCHRP report on impacts of access management as yielding “logical and consistent” results [4]. The other model was developed by Bowman and Vecellio, who conducted a study similar to that of Bonneson and McCoy and only addressed safety issues. This study involved the analysis of 32,894 vehicular collisions on 15 arterials in Atlanta, Georgia; Phoenix, Arizona; and Los Angeles and Pasadena, California. Bowman and Vecellio evaluated the same three cross-section types as Bonneson and McCoy and collected similar data on each. However, this study did include signalized intersections and analyzed collision rates rather than frequencies. The results of this study indicated that there was no statistically significant difference in vehicular collision rate between a raised median and a TWLTL in a central business district but that in suburban areas raised median segments had a significantly lower collision rate than did TWLTL segments. As expected, the undivided segments performed poorly in all scenarios [5].

Prior to the Bonneson and McCoy project, Harwood conducted a very similar study in California and Michigan. Harwood, however, analyzed several different types of cross-sections and used expected collision rates rather than frequencies. This project included five-year collision histories for 469 miles of suburban highway. Differences in geometries, traffic volumes, traffic characteristics, and land use were statistically controlled. Data that were collected include AADT, truck percentages, type of development, estimated level of left turn demand, lane width, shoulder width, speed, driveways per mile, and unsignalized intersections per mile. Harwood developed tables with expected collision rates based on land use and cross-section type with adjustment factors for driveways per mile, intersections per mile, truck percentage, and presence of full shoulders [6].

The results of the Harwood study indicate that four-lane median-divided cross-sections are most appropriate on major arterials with high volumes of through traffic and fewer than 45 driveways per mile. Harwood found that five-lane cross-sections with TWLTLs are best suited for suburban areas with commercial development, driveway densities greater than 45 per mile, lesser volumes of through traffic, and high volumes of left turn traffic [6].

In 1993, Mukherjee, Chatterjee, and Margiotta conducted a survey of state design engineers in the United States as well as a review of existing literature on the choice between TWLTL and nontraversable median. Out of 49 distributed questionnaires, 31 were completed and returned. These surveys included 14 questions and three case studies related to the choice between a median and a TWLTL. Both the review of existing safety and operations models and the survey of practitioners revealed contradictory cross-section preferences in many situations. This study found that the choice between median and TWLTL was not “clear cut” and warranted more research [7].

In 1995, Chatterjee and Margiotta conducted their own safety comparisons between these two cross-sections. For this study, data were collected for 25 highway segments in

Tennessee including 12 median-divided segments and 13 segments with a TWLTL. Collision data were collected from the Tennessee Roadway Information Management System and geometric data were collected from the Tennessee Department of Transportation (TDOT) photolog. Average daily traffic data were also gathered from the annual traffic volume maps published by TDOT. Chatterjee and Margiotta analyzed the data using analysis of covariance to develop predictive collision models. The researchers noted that the severities of collisions were very similar for the two cross-sections but that there were differences in terms of the locations of the collisions. Collisions on median divided segments were more frequent at signalized intersections while those on TWLTL segments were more frequent at unsignalized intersections. Rear-end collisions were more likely to occur on a median divided segment while head-on collisions were more likely to occur on a segment with a TWLTL. The models that were developed for this study suggest that medians are generally associated with fewer collisions than TWLTLs, however, in the case of high driveway densities and low to medium traffic volumes the TWLTL may be the safer option [8].

Two other research projects that involved the development of safety models were conducted in Indiana and Georgia. The study in Indiana, by Brown and Tarko, developed regression models to predict collision frequencies on multilane urban arterials. This project, carried out for the Indiana Department of Transportation (INDOT), involved collecting geometric and access control data as well as collisions by severity type. Negative binomial regression models were created to predict the total number of crashes, the number of property damage only (PDO) crashes, and the number of fatal and injury crashes. This study found that the number of collisions increased with access density and proportion of signalized access points. Additionally, fewer collisions were associated with roadways that had an outside shoulder, TWLTL, or raised median without openings between signalized intersections [9].

The study in Georgia, conducted by Squires and Parsonson, involved a statistical comparison of collision rates as well as regression equations for raised median and TWLTL segments on four- and six-lane roads. Regression equations were developed for each of the different cross-section types in terms of both total and midblock collisions and in terms of both collisions per million vehicle miles and collisions per mile per year. In all situations investigated, in terms of both statistical comparison and regression equations, the collision rate of the raised median segments was lower than that of the TWLTL segments [10].

Several other studies employed statistical comparisons of safety for each of the left-turn treatments without the use of collision models. The largest of these studies, carried out in Georgia by Parsonson, Waters, and Fincher, included 986 segments (839 miles) with a TWLTL and 1,125 segments (1,295 miles) with a raised median. Statistics were collected for both total and mid-block collisions. There was no separation for four- and six-lane segments or for urban and rural setting. The raised median sites were found to have a 45% lower crash rate and a 43% lower injury rate than the TWLTL sites. The overall fatality rates were comparable. Similar results were found when comparing only midblock collisions. This study, conducted from 1995-1998, can be easily compared to a similar study conducted on the same study area from 1989-1992. Nearly every measure of safety improved in the interval between the two research projects, with the raised median segments improving more than the TWLTL segments. As a result, the safety gap between the two left-turn treatments in the study area appears to have grown over time [11].

One smaller, yet pertinent, study was carried out in Charlotte, North Carolina by Debbie Self. This study compared eleven major arterials in Charlotte, including 7.9 miles of median divided highway and 7.1 miles of roadway with TWLTLs. Three and one-half years of collision data including total crashes, fatalities, injury crashes, and crash types were collected. The median divided segments exhibited fewer collisions in every category than the TWLTL segments, including 64% fewer total crashes, 84% fewer left-turn and angle crashes, and fewer fatalities, injuries and PDO crashes. This project is particularly relevant because the study area of Charlotte, North Carolina is also included in the current research project and, despite differing scope and methodology, can provide early insight into the potential results of the project at hand [12].

Numerous other studies involving conversions from TWLTL to raised median or vice versa also exist. In general, these studies confirm the research given above and tend to have smaller sample sizes, so they were omitted from discussion here.

Impact of Median Openings

The number and type of median openings in a segment can have a significant effect on both safety and operational characteristics. One study investigating these issues, conducted in Minnesota by Preston, Keltner, Newton, and Albrecht, looked specifically at the relationship between access and crash rates. A random sample of 432 segments (765 miles) was divided into eleven categories, with 9,545 access points and 13,700 collisions investigated. For each segment, data were collected on the number of access points, three-year collision statistics, and other characteristics. Video logs were used to count the access points and categorize them as a public street, a commercial driveway, a residential driveway, a field entrance, or other type of access point. Collision frequencies, rates, types, and severity information were gathered from the Minnesota Statewide Crash Database for the years 1994-1996. Other characteristics that were collected included length, AADT, vehicle miles traveled (VMT), speed limit, number of through lanes, median treatment, left turn treatment, environment, and design type [13].

In ten of the eleven categories that were investigated, crash rates were found to increase with access density. Additionally, crash rates increased with commercial driveway density in urban areas. Eleven case studies of collision management related projects in Minnesota and Iowa were also reviewed and a 40% collision reduction was observed after roadway improvements were implemented [13].

In addition to the number of median openings, the choice of full versus directional median openings is also important. Dissanayake and Lu conducted a before-and-after study comparing the operations and safety of these two opening types. In the before period the intersection operated as a full median opening and in the after period it operated as a directional median opening. Figure 8 shows the layout of the study site in both the before and after periods. One week of field data were collected using a video camera for both the before and after periods. Weighted average delay and weighted average travel time for left turning vehicles from the driveway served as the operational measures and conflicts served as the safety measure. The total weighted average travel delay was found to be reduced significantly when the intersection operated as a directional median opening. The travel times, however, were not found to be significantly different. Additionally, the average number of conflicts per hour and the conflict rate per thousand involved vehicles were reduced by nearly 50 percent when the intersection functioned as a directional median opening [14].

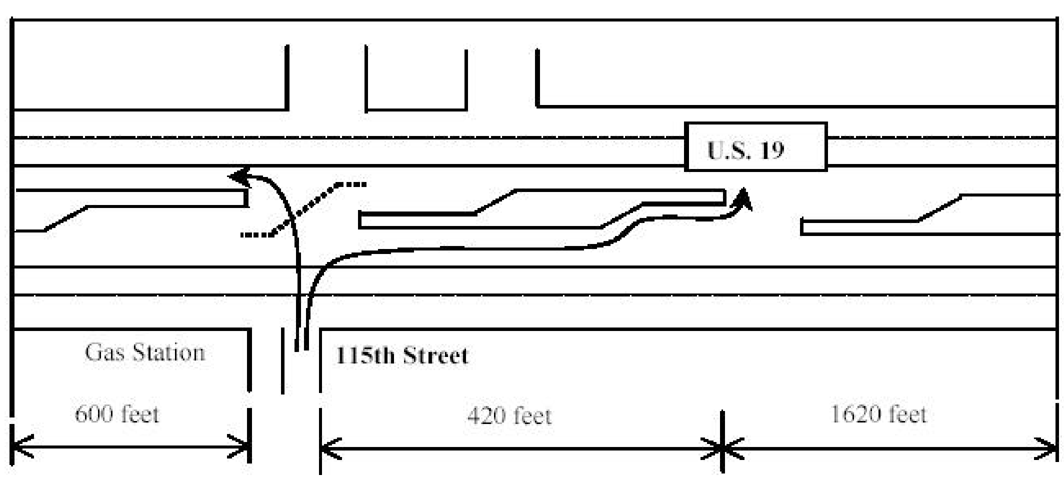


Figure 8. Layout of the Study Site [14]

One final issue related to median openings is the question of a direct left turn versus a right turn followed by a U-turn. If median openings are not allowed or are restricted at a driveway, drivers wishing to turn left must turn right and U-turn. Some concern has been expressed over the additional distance that these vehicles must travel as well as the danger that these U-turns may present. Dissanayake, Lu, Castillo, and Yi conducted a study on this very issue. Several locations on high-volume major arterials in the Tampa Bay Area were selected where left turns from a driveway were facilitated with either a direct left turn or a right turn followed by a midblock U-turn. The locations were video taped to determine number and severity of conflicts as well as traffic volumes. Over 300 hours of daytime traffic data and 1,654 conflicts were observed at these sites. For both the peak and off peak time periods, the right turn followed by U-turn option was associated with fewer conflicts than the direct left turn option. In the peak period, this difference was over 50 percent while in the off peak period it was just over 22 percent. The conflict rate for the right turn followed by U-turn sites was also lower than the sites that allowed a direct left turn [15].

In order to supplement the conflict data, a larger sample of sites was selected in order to investigate the collision history. The sample consisted of 133 sites that allowed direct left turns and 125 sites that required right turns followed by U-turns. For all categories, with the exception of sideswipes, the direct left turn sites experienced significantly more collisions. The right turn followed by U-turn sites did experience more sideswipe collisions, which may be attributed to excessive weaving necessitated in this design [15].

Impacts on Signalized Intersections

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 [16]. 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 [15]. 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.

OPERATIONAL IMPACTS

Impacts on Roadway Segments

The previously mentioned research project carried out by Bonneson and McCoy also involved the development of a model to address operational impacts of midblock left-turn treatments. Several key problems with various treatments were identified in the early stages of the project including left-turn bay overflow blocking through lanes, through traffic being slowed by turning vehicles or high traffic volume, spillback from a signalized intersection downstream, reduced through lane capacity at a signalized intersection downstream, and the effect of signalized intersections upstream on capacity of non-priority movements. The model that was created to address these issues is based on the 1994 Highway Capacity Manual procedures for signalized and unsignalized intersection analysis. To calibrate this model, traffic flow data were collected during 32 field studies in eight cities and four states. To expand the range of field data, they conducted 117 simulation runs [1].

The results of this portion of the Bonneson and McCoy study indicate that for traffic demands of no more than 40,000 vehicles per day, both TWLTL and raised median segments could function without hindering major-street movements. When left-turn volumes were low, there was no difference between the treatment types; however, at very high left turn and through volumes, the raised-curb median had slightly higher delays than did the TWLTL treatment [1].

One other study comparing operational aspects of the two cross-sections of interest employed simulation. A one-half mile segment was modeled in TRAF-NETSIM under a variety of conditions for each median type. Driveway densities of 32 and 64 drives per mile were modeled along with volumes of 600, 900, and 1200 vehicles per hour per direction. For the simulation, median openings were placed every 660 feet, arterial speed was set at 40 miles per hour, driveway speed was set at 25 miles per hour, and left-turn pockets were assumed to be 250 feet long. Because TRAF-NETSIM did not model U-turns, the researchers treated U-turns as if they were a left turn into a driveway followed immediately by a right turn out of the driveway in a way that likely did not greatly bias the travel time results. The selected measures of operational effectiveness were delay and fuel consumption. The factors found to affect the selected measures of effectiveness were driveway density, traffic volume, and median type [17].

For all scenarios, the TWLTL design was associated with less delay and better fuel efficiency for through traffic than the raised median design. For the simulations with low driveway density, the delay to left-turning traffic with the TWLTL was found to be 8.6% lower for low traffic volumes, 12.6% lower for medium traffic volumes, and 13.9% lower for high traffic volumes than was the delay to left-turning traffic with a raised median. For medium driveway density and low traffic volumes, the delay to left-turning traffic with the TWLTL was found to be 5.4% lower than that for the raised median. No significant difference was found between the cross-sections in terms of delay to left-turning vehicles for the cases of medium driveway density and medium or high traffic volume [17].

Impacts on Signalized Intersections

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 [18]. 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 [19,20].

Adams studied U-turns at signalized intersections to determine whether a U-turn factor should be included in HCM capacity analyses [21]. 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 [22]. 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:

$$SF = 1803 - 4.323 * UTURN - 0.484 * UTURN * RTOA \qquad \text{(Equation 1)}$$

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

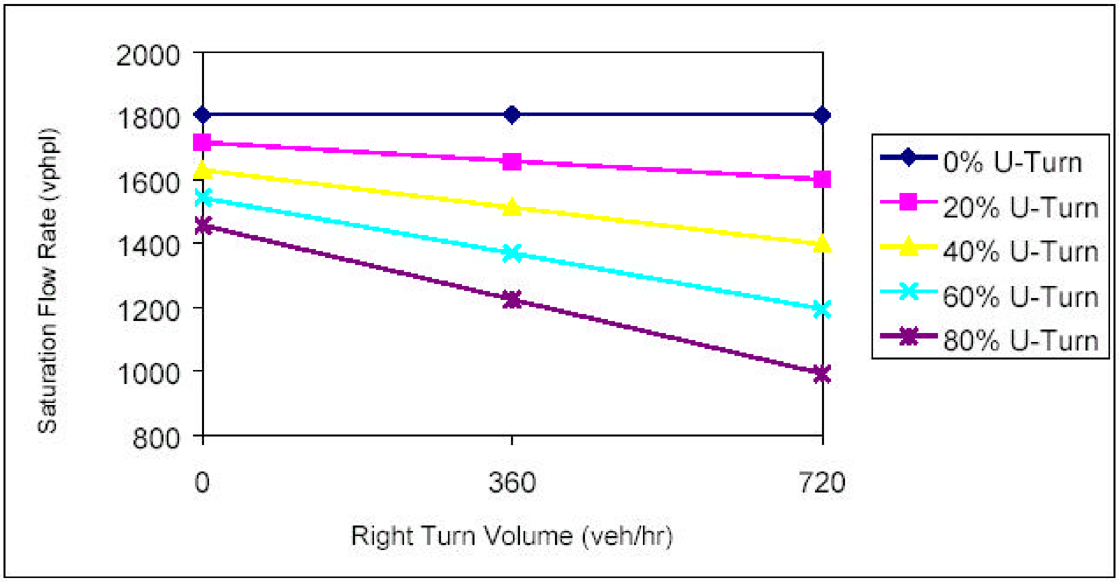


Figure 9. The Effect of Right Turn/U-Turn Movement Conflicts on the Saturation Flow Rate of U-turn/Left Turn Lane [22]

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 [23]. 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:

$$C = 799 - 0.31qc \qquad \text{(Equation 2)}$$

where:

C = capacity of U-turn movement (PCU/h); and

qc = 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.

OTHER ISSUES

Economic and Access Impacts

One of the main contentions with regard to median installation and left-turn treatment in general is the impact that it may have on businesses. In particular, a raised median with limited openings is generally thought to have negative impacts on businesses. Several studies have been done to assess these impacts. Most of these research projects depend upon surveys of businesses located on arterials that have undergone access management improvements. Four of the most important studies are discussed below.

The Bonneson and McCoy study, mentioned previously, also investigated the impact of left turn treatment on local economics due to access restrictions. For this portion of the study, 165 business owners or managers along four arterials in Florida, Illinois, and Wisconsin were surveyed. Each of these arterials had undergone some form of reconstruction in regards to the midblock left-turn treatment. The majority of the projects included in this study resulted in there being “no change” in the level of access provided to adjacent properties. In general, business owners believed that traffic and business conditions would improve with the conversion from an undivided cross-section to either a raised median with openings every 330 feet or a TWLTL. They did not believe, however, that business opportunities would be improved with the conversion from either a raised median with openings ever 330 feet or a TWLTL to a raised median with openings every 660 feet. Business owners tended to believe that customers value service and quality above property access. Therefore, most businesses felt that they could compensate for a loss in property access with good service and quality [3].

Eisele and Frawley created a methodology for estimating economic impacts of median design. The researchers first tested a trial methodology at one site in College Station, Texas. The research team later selected ten additional sites in Texas where data were collected in both pre- and post-construction periods. Both in-person interviews and mail-out surveys were used to assemble the opinions of business owners and managers on access management issues at these sites. Customer surveys were also administered at one site in order to compare customer responses with those of business owners and managers [24].

The project team found that business owners ranked accessibility fourth or lower in terms of factors that affected their ability to attract customers to their business. Customers ranked accessibility equally or even lower than did the business owners. Additionally, 85.7% of business owners present before, during, and after median installation felt that customers were as likely or more likely to patronize their business as before. These business owners also reported a 17.7% increase in customers per day after construction and a slight increase in gross sales. Perceptions before installation were more negative than after installation. Customers did indicate that they were less likely to visit a business during the construction phase [24].

The results of the study indicated that the type of business had an affect on the economic impact that a median would have. For example, business types such as durables and specialty retail, and fast-food and sit-down restaurants indicated increases in customers per day, gross sales, and property values after arterial improvement. On the other hand, gas stations, auto repair, and other service businesses indicated decreases in customers per day and gross sales after arterial improvement. As expected, the construction phase was identified as the most detrimental and should be alleviated by ensuring adequate, visible access, reducing construction time, and performing in small phases [24].

Vu and Shankar also conducted a study concerning perceived economic impacts of access management on businesses. They distributed 1900 surveys and received 280 responses along six major commercial corridors in the western portion of the state of Washington.

These responses were compiled with GIS data to relate the characteristics of individual businesses with their corresponding property access. This study employed the use of a bivariate probit model and a simultaneous logit model to analyze the data. Using these models, perception of accessibility was found to be related to the business type, business operational variables, corridor and street environment variables and the amount the business was willing to pay to move to a location with better access, as discussed below [24].

The more money a business was willing to pay to relocate, the more pessimistic it was about the effect of access management on customer patronage. Medium-sized businesses perceived customer patronage impacts more negatively. Businesses involved in retail service were more likely to perceive either no impact or a positive impact on customer patronage. Small businesses tended to not be concerned about accessibility [24].

In regards to the perceived economic impacts on a business due to existing access management,

- 33% reported no impact,
- 41% reported a minor negative impact,
- 11% reported a major negative impact,
- 7% reported a minor positive impact, and
- 8% reported a major positive impact.

The perceived customer impacts due to existing access management were very similar [25].

Figure 10 shows the preference of business owners for access management options at their business location. The preferences for corridor access management were very similar. Clearly, the more restrictive an access management option is, the fewer the number of business owners in favor of it [25].

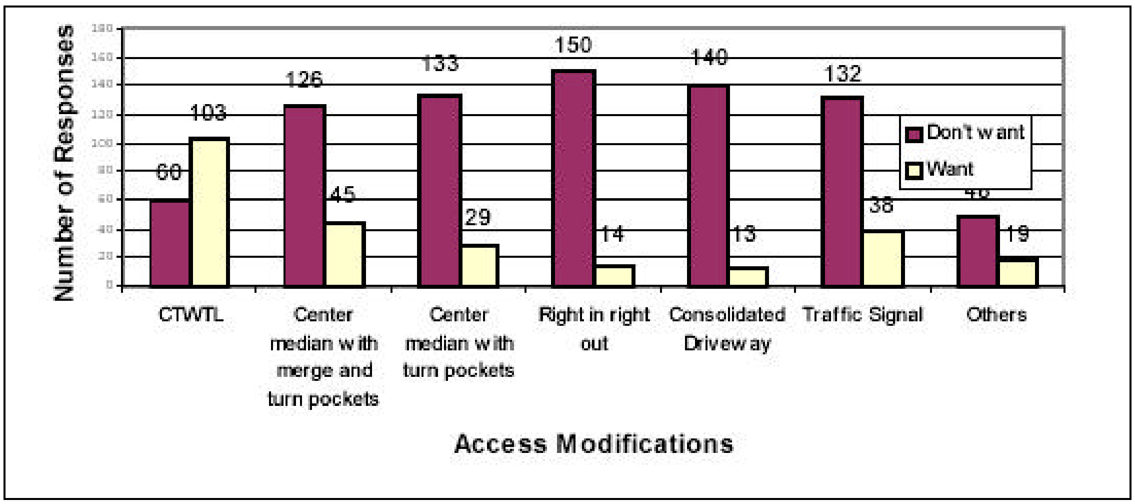


Figure 10. Preference for Access Modification at Business Location [25]

The final study, conducted by Levinson and Gluck, took a more theoretical approach to determining economic impacts from median treatments [26]. It is focused on the maximum adverse impact for locations where left-turn access is denied. They list five factors that will determine the impact associated with installing a raised median. These factors are:

- Size and type of adjacent land use where left turn access will be reduced,
- Reliance of adjacent land use on pass-by traffic,
- Number of vehicles turning left to access adjacent land use,

- Average purchase per vehicle or person, and
- Economic trends of the surrounding area.

The worst case scenario for those locations where median openings are not provided is that no pass-by vehicles will take an alternate route to the business or U-turn at the next median opening. If this is the case, the economic loss could be found by taking the product of the number of left turn entrants and the proportion of pass-by trips and multiplying that number of lost trips by the average purchase per vehicle or person. Summing this quantity for all businesses where left-turn access is denied would give the maximum adverse economic impact for a corridor. This worst case scenario, however, is highly unlikely. Additionally, economic losses for the corridor would be offset by transference of left turn pass-by trips to other nearby locations [26].

Pedestrian Safety

Pedestrian safety is a major concern when selecting left-turn treatment in areas with high pedestrian volumes. Raised medians are generally considered the safest option for pedestrians due to the refuge area they provide. However, at signalized intersections, where pedestrians often cross, TWLTL segments frequently transition into a short section of raised median such as that shown in Figure 11 [13]. As a result, some of the differences between these two cross-sections in terms of pedestrian safety are mitigated. Following are discussions of two studies investigating the impact, in terms of pedestrian collisions, of median treatments.

Figure 11. Short section of median on a TWLTL segment [12]

The Bowman and Vecellio study, discussed previously in regards to vehicular safety, also investigated pedestrian safety as it relates to median treatment [5]. For this portion of the study 1,012 pedestrian collisions were analyzed from the same segments as those used for the vehicular safety part of the study. The individual police records were analyzed for each pedestrian collision within 150 feet of the arterial center line to determine if it involved a major street vehicle. The size of any adjacent central business districts as well as the type of land use was recorded as a surrogate for pedestrian activity. Pedestrian collision rates in both the central business district and suburban areas were lowest for raised median segments and highest for undivided segments. There was no statistically significant difference between the collision rates for TWLTL sites and raised median sites [5].

The study conducted by Parsonson, Waters, and Fincher in Georgia that was mentioned previously, found a 78% lower pedestrian fatality rate on raised median segments than on TWLTL segments [11].

Pedestrian safety at crosswalk locations is a critical issue in designing facilities with high pedestrian activity. The Highway Safety Research Center in Chapel Hill, North Carolina recently conducted a study on the safety effects of marked versus unmarked crosswalks. They investigated 2,000 crosswalks in 30 cities across the United States. Pedestrian crash history, pedestrian volumes, AADTs, type of median, and other geometric characteristics were collected for each site. This study found that the "presence of a raised median (or raised crossing island) was associated with a significantly lower pedestrian crash rate at multi-lane sites". Additionally, they found that TWLTLs "did not offer significant safety benefits to pedestrians" [27].

Public Perception and Involvement

Public perception and involvement can be a major obstacle in projects that involve changes in access control, and especially in median projects. The following two studies review experiences in Florida and Georgia with access management projects and their involvement with the public. While public perception and level of involvement may vary substantially by area, these two studies can give some insight into how local citizens may view a median project, and how to address any concerns they may have.

In 1993, the Florida Department of Transportation (FDOT) adopted a policy of raised medians on all major multilane roadways with posted speeds greater than or equal to 45 miles per hour [28]. Since that time, they have had a great deal of experience dealing with the public in regards to median installation. They have made the following recommendations in regards to this issue:

- Even when it is not required, all median projects should include public involvement because of their controversial nature.
- Public involvement should occur in both the planning and project development phase and early in the design phase.
- Public involvement should include more than just public hearings.
- In order to establish and maintain credibility with the public, coordination and consistency in decision making is crucial.
- The public needs to be made aware of the reasons for median improvements.
- All communication should be documented and the public should be provided with feedback on key issues.
- The FDOT has found that a proactive approach to dealing with the public is the best way to avoid controversy and reduce impedance to projects.

Dixon, Hibbard, and Mroczka reviewed the public perception of raised medians and TWLTLs for three median improvement projects in Georgia. They noted that citizens in public hearings for these three projects focused on five basic areas of concern. These concerns are explained in detail in the following paragraphs [29).

The first area of concern is total project opposition. Some people in each project were completely against the project regardless of the median treatment. They felt that widening would create more traffic, encourage cut-through traffic, and encourage development.

The next issue is design based on abutting land use. Commercial property owners overwhelmingly preferred the TWLTL option. Citizens with adjacent property that was not located at a median opening also preferred the TWLTL. Nearly all other citizens preferred the raised median opening.

Access constraints are another area of contention. Access for emergency vehicles, businesses, and adjacent residential property were very important to the local citizens. Some people also stated they liked being able to use the TWLTL as an acceleration lane. On the other hand, most citizens felt that the raised median was better for pedestrian access.

One key concern for the public is safety. Some people viewed the TWLTL as a “suicide” lane. Others mentioned a potential hazard caused by raised medians due to the necessity to U-turn when direct left turn access was not permitted. The faster speeds that a raised median could encourage were felt to be dangerous; however, people felt that the raised median would be better for pedestrian safety.

The final area of concern was cost. The TWLTL is generally slightly less costly to build than the raised median. Most people were not overly concerned with this issue.

SEGMENT METHODOLOGY

Due to the quality of the Bonneson and McCoy project mentioned previously [1], the form of that collision model was utilized in this study. The collision models from that study were:

Model 1. Bonneson and McCoy Collision Prediction Model

$$A_R = ADT^{0.910} Len^{0.852} e^{(-15.162 - 0.296 I_{b/o} - 0.596 I_{r/i} + 0.00478 (DD+SD) I_{b/o} + 0.0255 PDO)}$$

$$A_T = ADT^{0.910} Len^{0.852} e^{(-15.162 + 0.018 I_{b/o} - 0.093 I_{r/i} + 0.00478 (DD+SD) I_{b/o} + 0.0255 PDO)}$$

where,

A_R = annual mid-signal collision frequency for raised median sites;

A_T = annual mid-signal collision frequency for TWLTL sites;

AADT = average daily traffic demand, vpd;

Len = segment length, feet;

DD = driveway density (two-way total), drives/mile;

SD = unsignalized public street approach density (two-way total), approaches/mile;

PDO = property damage only collisions as a percentage of total collisions, %;

$I_{r/i}$ = indicator variable for residential or industrial land uses (1.0 if res /ind; 0.0 otherwise);

$I_{b/o}$ = indicator variable for business or office land uses (1.0 if bus/office; 0.0 otherwise)

Note that, for the Bonneson and McCoy models, all coefficients are identical with the exception of those for the land use terms. After running the models individually, they found that the coefficients were generally very similar. As a result, they combined the models to expand the sample size per model and used an indicator variable for cross-section design [1].

Data that were collected for the Bonneson and McCoy model include the number of through traffic lanes, the segment length, the cross-section width, the median width, the driveway and unsignalized public street approach density, the speed limit, and the average daily traffic (AADT). Adjacent land use and the presence of parallel parking were also noted during the data collection process. Of these variables, only those shown in the equations above were found to be significantly related to collisions [1].

The terms that were included in the model can be split into two categories. Average daily traffic and segment length are the exposure factors, while driveway density, street density, left-turn treatment, and land use are explanatory factors. The percentage of property damage only collisions in the area is also included to account for collision cost reporting thresholds. The R^2 value for this model using the Bonneson and McCoy data is 0.69 [1].

Since the model developed by Bonneson and McCoy was judged to be "logical and consistent" [4], appears to adequately predict collisions in their study area, and is of the negative binomial form that is the current state of the art, the main concern for this research project was to recalibrate this model for North Carolina.

SITE SELECTION

In order to calibrate an empirical collision model, a large sample of each type of segment must be selected. To make the model as encompassing as possible, the sites needed to cover as much of the state as was feasible. Additionally, to remove any bias from the study, the sites needed to be randomly selected.

NCDOT provided a newly-updated inventory of all four- and five-lane roadway segments in North Carolina from which the segments for this project were selected. This database was filtered in order to find all segments that met the desirable characteristics for this study. These characteristics are:

- Either a raised median or a two-way-left-turn-lane dividing the through travel lanes,
- Segment length greater than ¼ mile long in order to lessen signalized intersection influence on collision history,
- 35-45 mile per hour speed limit to maximize the likelihood of selecting suburban highways,
- An AADT of at least 20,000 since few problems with either segment type are encountered at lower volumes, and
- No widening or major changes within the last three years in order to minimize impact of roadway changes on collision history.

The original NCDOT inventory, containing 5,917 segments from 87 of North Carolina's 100 counties, was filtered for the traits given above. Once the inventory was filtered, 429 segments, of which 214 are 4-lane median divided and 215 are 5-lane with TWLTL, were found to have the required characteristics. These sites were in 49 counties. From these remaining segments, 100 of each segment type were randomly selected for drive-through observation. The selected 4-lane segments encompass 26 counties and the selected 5-lane segments include 35 counties.

DATA NEEDS

A great deal of information regarding each roadway segment is needed in order to calibrate the models. These characteristics include volume, geometric, land use, and collision data on each segment. Some parameters that were not included in the Bonneson and McCoy model were collected to determine if they might be significantly related to collisions in North Carolina even though they were not significant in Nebraska and Arizona.

Volume Data

Average daily traffic (AADT) estimates from NCDOT for the year 2000 were used to account for exposure on a given segment.

Geometric Data

Geometric data that were needed for calibration include:

- All possible names of the main roadway,
- Names and mileposts of segment endpoints,
- Segment length,
- Posted speed limit,
- Number of public street approaches,
- Number of driveways,
- Type of intersection control, such as signalization, at endpoints,
- Presence of curb parking,
- Cross-section type (raised median or TWLTL), and
- Median width.

All segments were first located on one or more maps using the mileposts given by the NCDOT segment database and as much information as possible was collected prior to drive-through site visits. This information included segment location, approximate length, all possible names for the main roadway, and names of intersecting streets to use for reference purposes during the trips. Segment length, road names, speed limit, cross-section type, and median width were then recorded from the NCDOT database and confirmed during site visits. All other geometric data were collected exclusively through site visits. All segments visited had pavement and other roadway conditions that were not so poor or different that the site had to be excluded.

Segment length was approximated using odometer readings and verified using milepost values of segment endpoints. Driveways and public street approaches were counted separately for each side of the road. For the purpose of this study, driveways were defined as low volume entrances to homes, businesses, or offices and public street approaches were defined as the larger, higher volume roads leading into residential, business, or office developments. Some practical judgment was used in differentiating between these types of entrances. Additionally, very large curb cuts, some greater than 50 feet wide, were recorded as more than one driveway if they were deemed to function as such.

Land Use Data

The approximate land use percentage in each segment was estimated during drive-through site visits. Land uses were categorized as business, office, residential, industrial, or undeveloped. Residential land use includes single family dwellings, residential complexes, and everything in between. Office land use includes buildings for which the predominant trips generated by the establishment are by professional employees and their clients. Business land uses are those that depend upon customer trips in addition to employee trips. Industrial designates those land uses for which most of the trips are made by the employees. For the purposes of the model, the predominant land use in terms of vehicle trips must be specified. In the Bonneson and McCoy study, it was noted that the business and office land uses exhibited similar collision patterns [1]. Similarly, the residential and industrial land uses had similar collision patterns. Due to these similarities, these land uses were combined. These same land use combinations were used for the present model as well. In the event that one land use, other than undeveloped, was judged to encompass more than 50% of the adjacent property, it was selected as the predominant land use. If two land uses were tied at 50%, the land use that would produce more traffic was selected as dominant. For example, a split of 50% residential and 50% business was recorded as predominantly business since the business land use would generate more trips than the residential land use. Only if the entire segment was undeveloped would the undeveloped land use be chosen as predominant.

Collision Data

Collision data were collected for each segment for the time period from October 1, 1999 through October 1, 2002 using NCDOT's Traffic Engineering Accident Analysis System (TEAAS). In order to remove collisions that may have been caused by a signalized intersection, all collisions in the first 150 feet from a signalized intersection were discarded and the segment length was reduced by this amount. Additionally, all rear-end collisions in the first 500 feet from a signalized intersection were removed. This process is depicted in Figure 12.

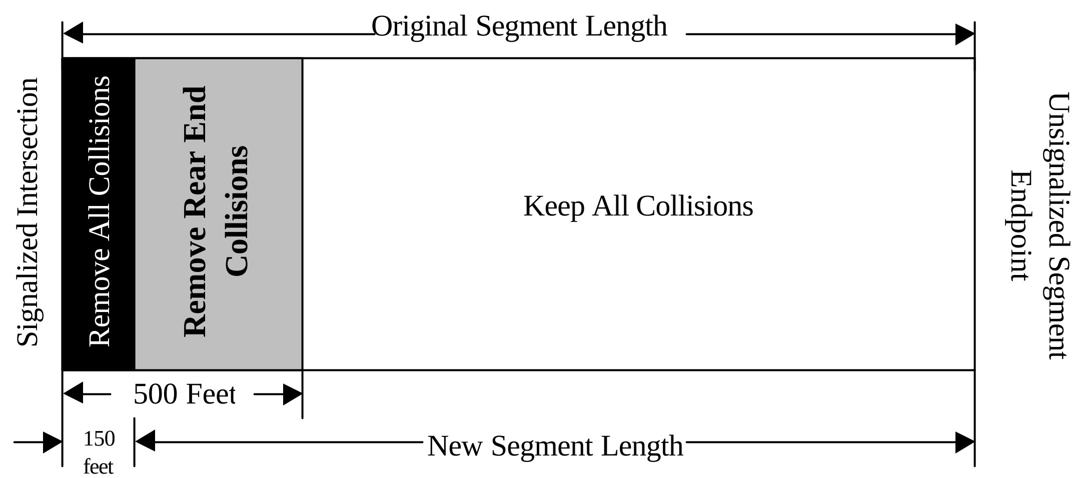


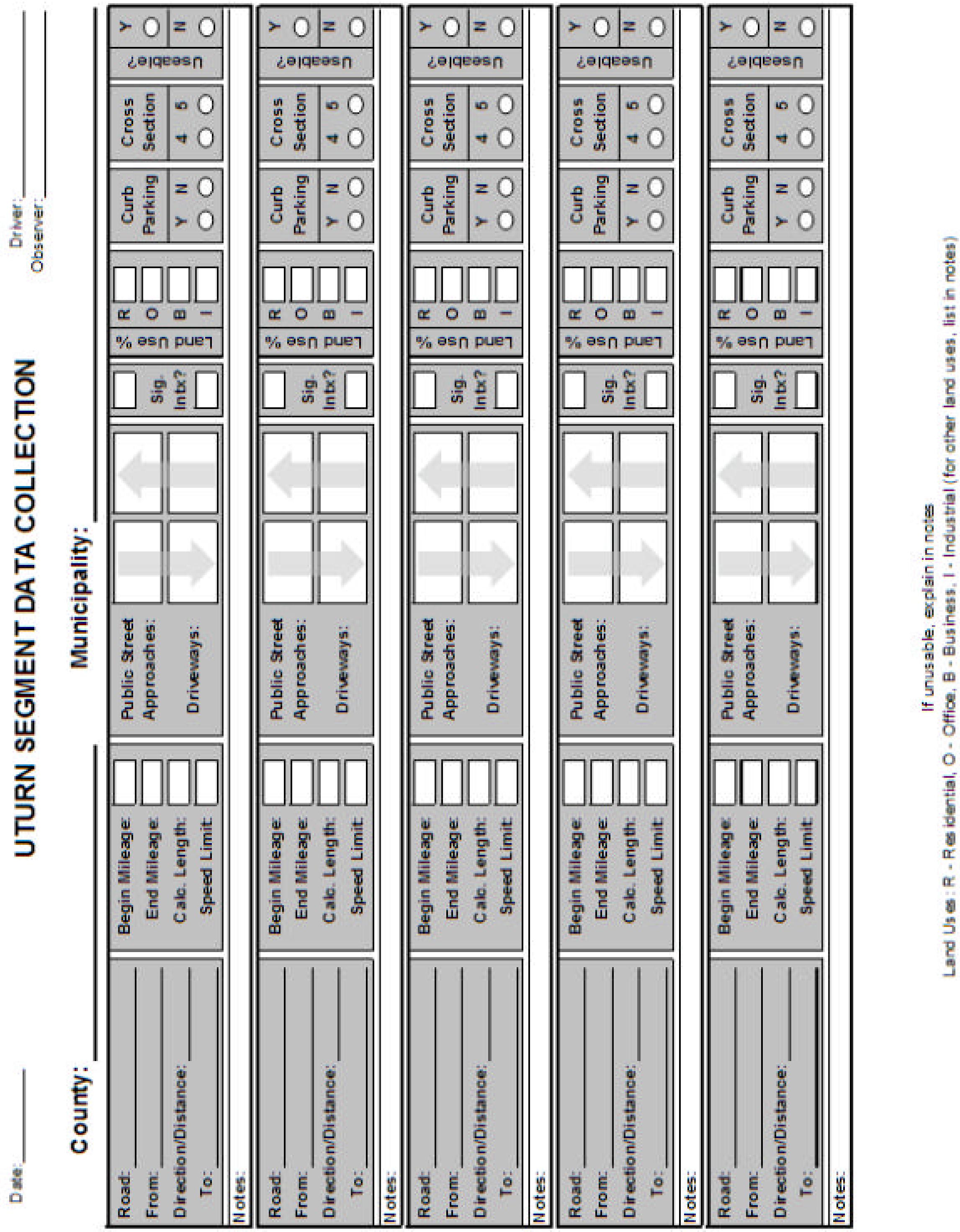
Figure 12. Removal of Collisions at Signalized Intersections

The total number of collisions as well as severity information was collected for each site. The NCDOT also provided property damage only (PDO) percentages by county in order to account for differences in reporting practices. An area where drivers tend to report more collisions will experience a high PDO percentage and a higher number of reported collisions than an area where drivers do not report collisions as often. The PDO term in the model accounts for this reporting threshold.

SITE VISITS

Each of the randomly-selected segments was visited during the summer of 2003. The data collection form shown in Figure 13 was used to record all the necessary information about each site. For each visit, two data collectors were used. The driver announced odometer readings at the beginning and end of each segment and counted driveways and public street approaches, when possible, on the left side of the road. The passenger recorded all other information.

Some segments were discarded during site visits due to a variety of factors. The most frequent cause for discarding a site was a signalized intersection located in the middle of the segment. If the signalized intersection was located in such a way that the distance between the signal and at least one of the original endpoints was still at least ¼ mile long, the portion from that original endpoint to the signal was used. Frequently, however, the distance between the signal and either of the endpoints was less than ¼ mile long and the site was subsequently discarded.



UTURN SEGMENT DATA COLLECTION

Date: ______ Driver: ______ Observer: ______

County: ______ Municipality: ______

Road: ______ From: ______ Direction/Distance: ______ To: ______
Begin Mileage: End Mileage: Calc. Length: Speed Limit:
Public Street Approaches: Driveways:
Sig. Intx?
Land Use % R O B I
Curb Parking Y N
Cross Section 4 5
Useable? Y N
Notes:

Road: ______ From: ______ Direction/Distance: ______ To: ______
Begin Mileage: End Mileage: Calc. Length: Speed Limit:
Public Street Approaches: Driveways:
Sig. Intx?
Land Use % R O B I
Curb Parking Y N
Cross Section 4 5
Useable? Y N
Notes:

Road: ______ From: ______ Direction/Distance: ______ To: ______
Begin Mileage: End Mileage: Calc. Length: Speed Limit:
Public Street Approaches: Driveways:
Sig. Intx?
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Sig. Intx?
Land Use % R O B I
Curb Parking Y N
Cross Section 4 5
Useable? Y N
Notes:

Road: ______ From: ______ Direction/Distance: ______ To: ______
Begin Mileage: End Mileage: Calc. Length: Speed Limit:
Public Street Approaches: Driveways:
Sig. Intx?
Land Use % R O B I
Curb Parking Y N
Cross Section 4 5
Useable? Y N
Notes:

If unusable, explain in notes
Land Uses: R - Residential, O - Office, B - Business, I - Industrial (for other land uses, list in notes)

Figure 13. Segment Data Collection Sheet

Other sites were discarded due to incorrect cross-sections, speed limits outside of the desirable limits, or inability to locate the segment due to lack of signage or renaming of roadways. In the event that a nearby segment, not already selected for the study, met all desirable criteria and had similar geometric, land use, and volume characteristics as the discarded site, the nearby segment was substituted into the study. This substitution kept the sample size sufficiently high, which prevented a second round of site selection and site visits.

DATA ANALYSIS

The first step in data analysis was to compare collision data to determine what differences, if any, exist between the two cross-sections in terms of collision severities and types. The next step was to determine if the models from the Bonneson and McCoy project could adequately predict collisions in North Carolina. These models were tested by plotting the actual number of collisions in a three-year period versus the number of collisions predicted by the models. Goodness of fit was judged on the basis of R^2, slope, and intercept values. The R^2 value gives an idea of how well the data fit their least squares regression line. The slope and intercept values indicate how close the least squares regression line is to "ideal". A perfect fit would include a R^2 value of 1.0, a slope of 1.0, and an intercept of 0.0.

MODEL CALIBRATION

The first step in calibrating the empirical collision models was removing a small portion of the data set to be used in validating the model. For each data set, 20% of the sites were randomly extracted and left out of the calibration process.

SAS® was used to perform the calibration of these models. Since the form of the Bonneson and McCoy model proved to be effective, the recalibration process mainly involved determining which parameters were significant and what the coefficients of those parameters should be. The variables that were considered for inclusion in the model are:

- Average Daily Traffic (AADT), vpd,
- Length of Segment, feet,
- Driveway Density, drives per mile (two-way total),
- Unsignalized Public Street Approach Density, approaches per mile (two-way total),
- Median Width, feet,
- Speed Limit, mph,
- Countywide Percentage of Property Damage Only Collisions, %,
- Business/Office Indicator Variable (1.0 if predominantly business or office land use, 0.0 otherwise),
- Residential/Industrial Indicator Variable (1.0 if predominantly residential or industrial land use, 0.0 otherwise), and

- All possible interactions between any two of the above variables.

Including all interaction variables, 45 parameters were examined for each of the models. The "genmod" procedure in SAS®, assuming a Poisson distribution for collisions, was used to perform the model fitting. Collisions are generally assumed to follow the Poisson distribution since a large proportion of the sites will have few collisions and very few sites will experience a large number of collisions. For each SAS® run, the parameter with the least significance was removed until all remaining parameters were significant. Additionally, variables were added back into the model periodically to determine if the removal of a previous parameter may have made it significant.

MODEL VALIDATION

Once the significant parameters and their coefficient estimates were determined, the model was tested with the previously removed validation sites. The same method for determining the goodness of fit of the Bonneson and McCoy model was utilized in determining how well this new model described both the calibration and validation sites. Predicted collisions were plotted versus actual collisions. Ideally, the plot of calibration sites would be similar to that of validation sites.

Residuals of the model were also examined to determine if the model assumptions were valid. Residuals are the difference between the predicted and actual collisions. These differences were plotted versus the predicted number of collisions to determine if any non-random pattern emerged or if there was any evidence of increased or decreased scatter as the collisions prediction increased. Ideally, there would not be any nonrandom patterns to the plot and there would be approximately the same degree of scatter at all collision prediction levels.

INTERSECTION METHODOLOGY

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, we 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.

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. The project team 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, we 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 [21].

4. *Local site.* To minimize travel costs, we 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. We 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 us 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, we 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, the team had 14 sites that were appropriate for these operational studies (see Table 1). These sites were selected from 106 sites that we 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. None of the selected sites had bulb-outs, but 10 of the sites had shoulders that ranged from 1 to over 12 feet wide.

Table 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, the team relied on the guidance given by 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.

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 14. 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 14. This was an unusual situation, but the high percentage of U-turns provided unique and valuable data.

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.

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.

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.

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 [30].

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, the project team 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.

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, we 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*. We 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. We 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 2 provides a summary of the number of sites selected for the safety study.

Random Sites

The group of random sites was selected as part of the segment portion of this study. The data collection for the segment portion of the 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 **S**ites*

To select sites with high U-turn volumes or a history of U-turn collisions, we contacted 120 city and state transportation engineers across North Carolina. We asked each person to give us a list of signalized intersections in their area that had high percentages of U-turns. Twenty-three people responded giving us 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 2. Sample Size for Safety Study

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

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 15 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.* The team collected data on signal phasing, lane widths, number of left turn lanes, median width, and number of lanes receiving U-turns.
3. *Segment information.* The team collected data on segment length, speed limit, number of access points, and approximate land use percentages.

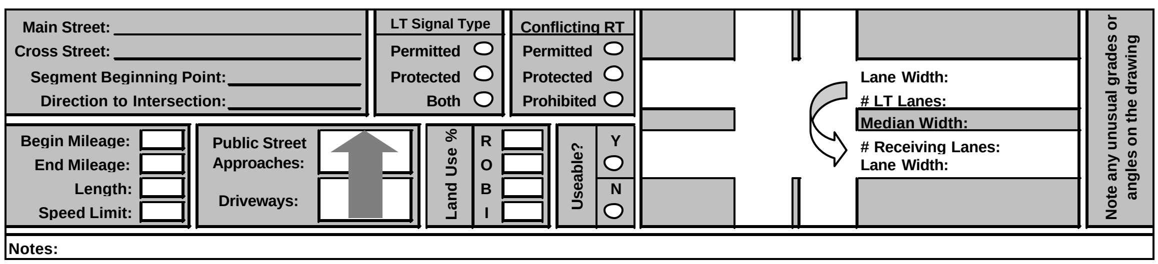


Figure 15. Physical data collection form

For each site in the group of recommended sites, we collected all applicable data during a site visit. The data for the randomly selected sites were collected during the data collection trips in the segment portion of the study. This collaboration resulted in efficient use of resources and sped up the data collection process for this project.

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 16 shows a diagram and narrative indicating that the collision involved a U-turning vehicle and a right-turning vehicle.

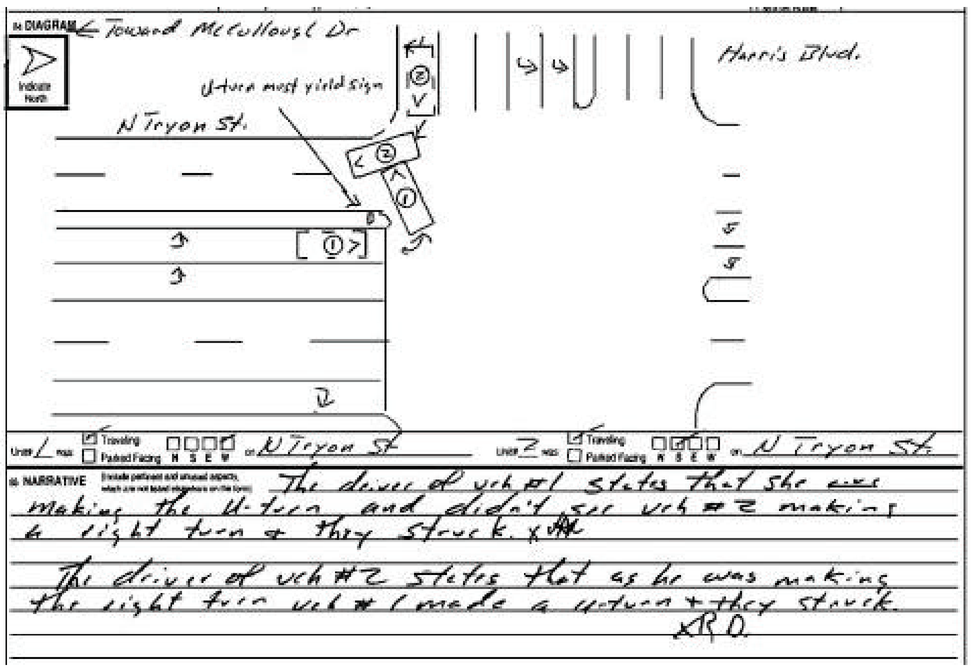


Figure 16. Sample Collision Report with U-Turn Collision

Collection of Traffic Volume Data

For each site in the study, we obtained information on main road Annual Average Daily Traffic (AADT). 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, we desired more specific information on the turning movements. We were 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.

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." [30]

SEGMENT RESULTS

Of the 200 randomly selected sites, 143 were found to meet all desirable characteristics and were used for calibration and validation of the empirical collision models. Of the 143 total sites, 62 have a raised median and 81 have a TWLTL. Table 3 gives some summary information about the data that were collected for each of the cross-section types. Approximately 87 miles of raised median and TWLTL roadways were included in this study. Of the 286 total segment endpoints, 201 were signalized intersections and 85 were unsignalized approaches. The unsignalized approaches either were original endpoints from the NCDOT database or were locations where either the cross-section or the speed limit changed.

DATA ANALYSIS

Once all data were collected, preliminary analysis could begin. First, the collision data were analyzed to determine what differences, if any, exist between the two cross-sections in terms of collision severity and type. Next, the fit of the collected data to the Bonneson and McCoy models was analyzed to determine if the models could adequately predict collisions or if they needed to be recalibrated for North Carolina.

Table 3. Summary Segment Data

	Parameter		Raised Median	TWLTL
Exposure Characteristics	Number of Segments	Total	62	81
		Calibration	50	65
		Validation	12	16
	Segment Length, miles	Minimum	0.25	0.25
		Average	0.59	0.62
		Maximum	1.59	1.3
		Total	36.49	50.5
	Annual Average Daily Traffic (AADT), vph	Minimum	20,000	20,000
		Average	31,000	27,000
		Maximum	56,000	50,000
Geometric Characteristics	Driveway Density, drives per mile	Minimum	0	0
		Average	22	46
		Maximum	100	123
	Public Street Approach Density, approaches per mile	Minimum	0	0
		Average	4	5
		Maximum	25	23
	Number of Segments with Curb Parking		0	0
	Median Width, feet	Minimum	2	10
		Average	26	12
		Maximum	48	17
Land Use Composition	Average % Residential Land Use		28	24
	Average % Office Land Use		3	2
	Average % Business Land Use		46	61
	Average % Industrial Land Use		0	1
	Average % Undeveloped Land Use		23	12
Safety Indicators	Total Collisions		2174	2562
	Collisions per Segment	Minimum	3	0
		Average	35	32
		Maximum	123	205
	Fatal and Injury Collisions per Segment	Minimum	1	0
		Average	12	12
		Maximum	52	72
	Property Damage Only Collisions per Segment	Minimum	1	0
		Average	23	19
		Maximum	79	133

Analysis of Collision Data

The collision severities were first analyzed. Figure 17 and Table 4 show comparisons of the two designs in terms of collision severities. As this table and graph show, the raised median cross-section had a slightly higher proportion of fatalities, class C injuries, and property damage only collisions than did the TWLTL cross-section. Half of the fatalities on the raised median segments were pedestrians. It is likely that there is a higher pedestrian exposure on raised median cross-sections than on TWLTL cross-sections due to perceived safety or lack thereof. This difference could account, in part, for the higher proportion of fatalities on raised median segments. Keep in mind, though, the well-established finding from the literature that the rates and severities of pedestrian crashes are higher on roads with TWLTLs than with raised medians. The TWLTL cross-sections exhibited a higher proportion of both class A and B injuries than did the raised median cross-section. Overall, the collision severities between these two designs are very similar.

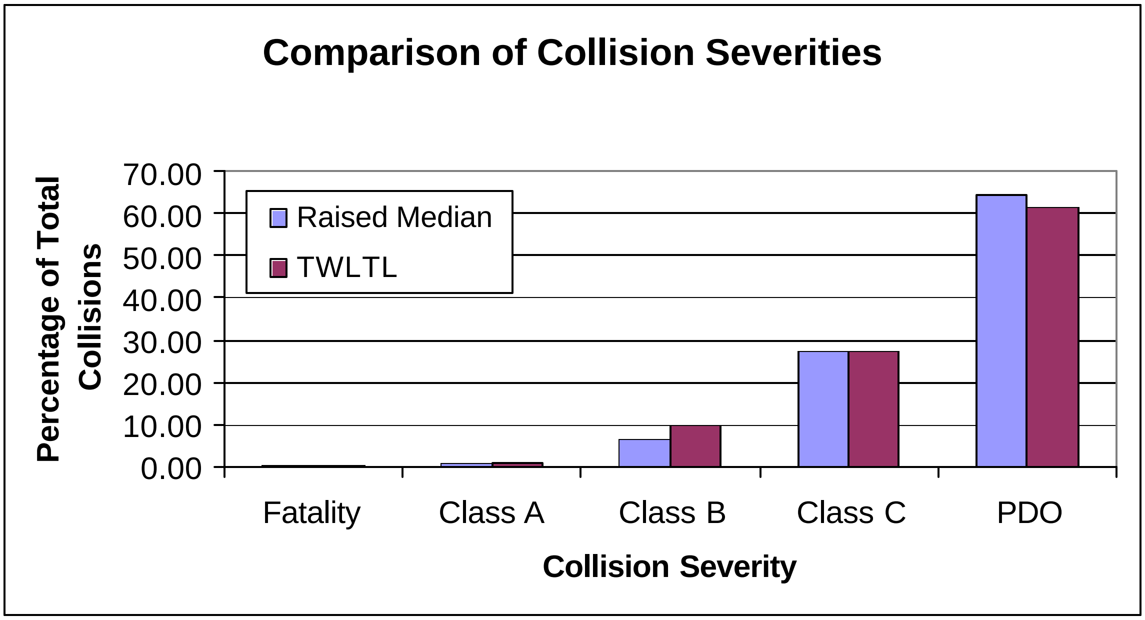


Figure 17. Comparison of Collision Severities

Table 4. Comparison of Collision Severities

Collision Severities		Fatality	Class A	Class B	Class C	PDO
Raised Median	Frequency	12	20	147	595	1400
	Percentage	0.55	0.92	7	27	64
TWLTL	Frequency	5	30	253	699	1575
	Percentage	0.20	1.17	10	27	61

Figure 18 and Table 5 show comparisons of collision types between the two left turn treatments. As would be expected, the raised median cross-section experienced a smaller proportion of angle, left turn, and head-on collisions than did the TWLTL cross-section. Rear end collisions were the most predominant collision type for both designs, with the raised median cross-section having a larger proportion of this type than the TWLTL cross-section. Other collision types exhibit very similar proportions for each segment design alternative.

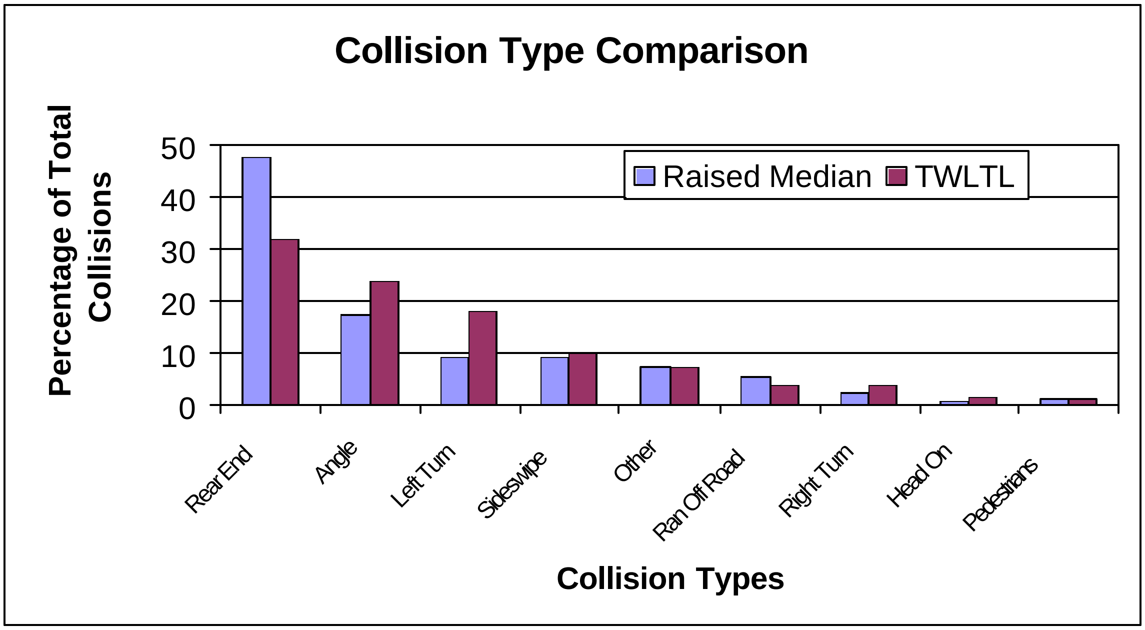


Figure 18. Comparison of Collision Types

Table 5. Comparison of Collision Types

Collision Types		Rear End	Angle	Left Turn	Sides wipe	Other	Ran Off Road	Right Turn	Head On	Pedestrians
Raised Median	Frequency	1032	382	196	199	159	119	52	13	22
	Percentage	47	18	9	9	7	5	2	0.60	1.01
TWLTL	Frequency	809	606	458	253	180	96	93	35	32
	Percentage	32	24	18	10	7	4	4	1.37	1.25

Fit of Data to Bonneson and McCoy Models

The next step was to test the models from the Bonneson and McCoy project to determine if they adequately predicted collisions in this data set. Figures 19 and 20 depict the relationships between actual collisions in the data set and collisions predicted using the Bonneson and McCoy models for raised median and TWLTL sites, respectively.

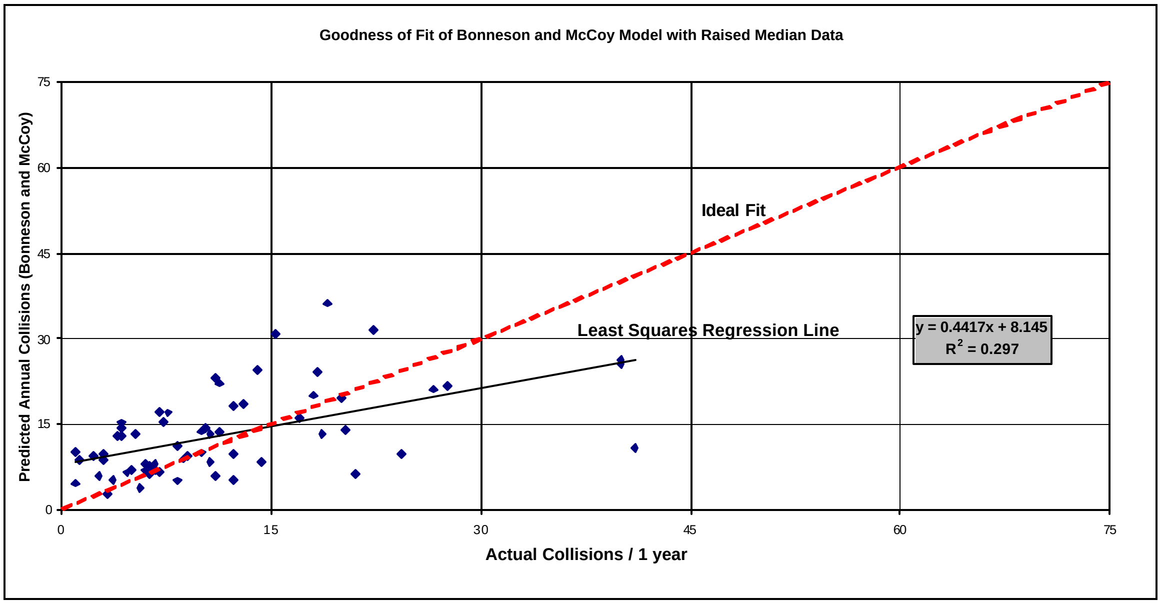


Figure 19. Fit of Bonneson and McCoy Raised Median Model

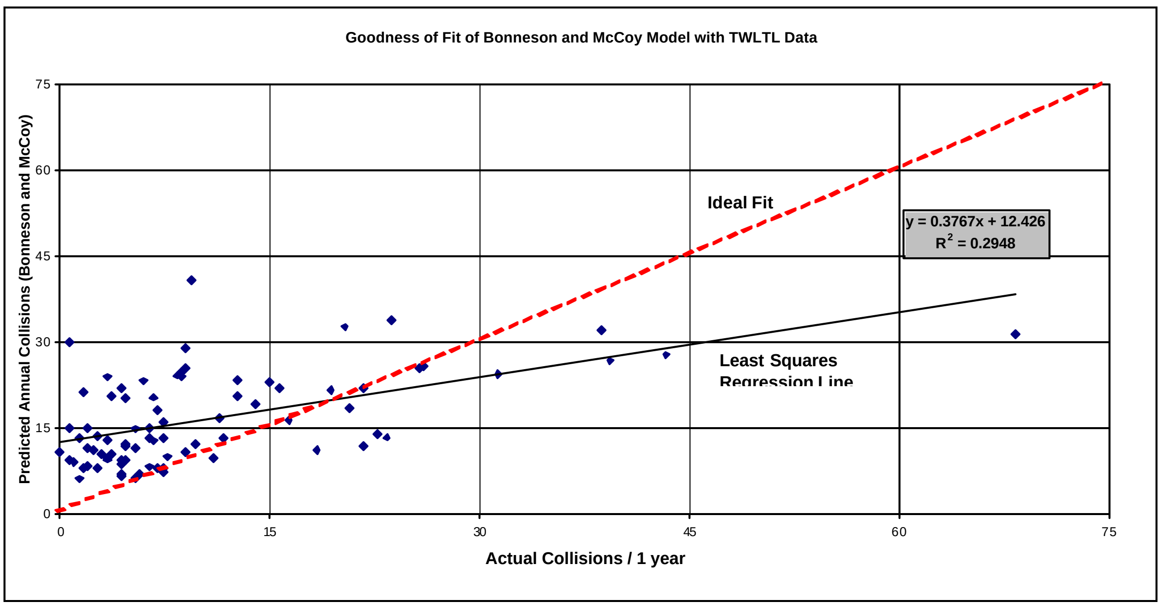


Figure 20. Fit of Bonneson and McCoy TWLTL Model

In each graph, the dotted line indicates where the least squares regression line should fall if there were a perfect fit between the model and the data. The solid line is the actual least squares regression line using the Bonneson and McCoy prediction. The R^2 value gives an idea of how well the data fit the solid line. If the data fit the least squares regression line perfectly, this value would be 1.0. The equation of the least squares regression line can be used to determine how close it is to the "ideal" dashed line. The ideal line has a slope of 1.0 and an intercept of 0.0. The R^2, slope, and intercept values for each of the models are shown in Table 6.

Table 6. Fit of Bonneson and McCoy Models to Collected Data

Goodness of Fit Parameters - Bonneson and McCoy							
	R^2	Slope			Intercept		
		Value	p-value	significantly different from 1.0	Value	p-value	significantly different from 0.0
Raised Median	0.1643	0.5822	0.0166	yes	13.413	<0.0001	yes
TWLTL	0.2948	0.3767	<0.0001	yes	12.4258	<0.0001	yes

At the 95% confidence level, there was statistically significant difference between both of the slope values and 1.0 and between both of the intercept values and 0.0. Additionally, since the R^2 values for both plots were far from ideal, it was determined that the Bonneson and McCoy models needed to be recalibrated.

MODEL CALIBRATION

For the purposes of validating the empirical collision models, 20% of the sites for each cross-section were held out of the calibration process. As a result, 50 raised median sites and 65 TWLTL sites were used for model calibration.

Model calibration was performed using SAS®. The "genmod" procedure, a method used to determine coefficients for model fitting, was used to develop a relationship between collisions and the exposure and explanatory variables. Collisions were assumed to follow a Poisson distribution. Additionally, to correct for any overdispersion, a scaling parameter was used. That is, in the Poisson distribution, the variance is assumed to be equal to the mean. If the variance is larger than the mean, then the data are overdispersed. The scaling parameter corrects for this problem. This procedure was carried out as described by Litell, Stroud, and Freund [31].

The natural link of the assumed statistical distribution is used in a generalized linear modeling procedure such as the genmod procedure. For the Poisson distribution, the natural link is the log function. Therefore, collisions are assumed to be related to the exponential of the other variables. In other words,

$$\text{Collisions} = e^{(\text{explanatory variables})}$$

This idea is supported by the form of the Bonneson and McCoy models [1].

Previous research has shown that exposure variables, such as AADT and length, are not linearly-related to collisions. If they were linearly-related, then the equation would simply be multiplied by these variables as shown below.

$$\text{Collisions} = (\text{AADT})*(\text{Length})*e^{(\text{explanatory variables})}$$

However, since they are not linearly-related, the log of these variables is included in the modeling procedure. The estimate provided by SAS® is then incorporated as the power to which these variables should be raised in the model. If they were linearly-related, the estimate would be 1.0.

Using the log function in the model fitting could cause a problem for those segments that experienced no collisions during the three year period. However, since the genmod procedure employs an iterative model fitting process, the zero collision sites were not an issue.

The independent variables that were tested in the model include:

- AADT, vpd,
- Segment length, feet,
- Median width, feet,
- Posted speed limit, mph,
- Driveway density, drives per mile (two-way total),
- Unsignalized public street approach density, approaches per mile (two-way total),
- Approach density (driveway density plus unsignalized public street approach density), approaches per mile (two-way total),
- Business/office indicator variable, 1.0 if predominantly business or office land use, 0.0 otherwise,
- Residential/industrial indicator variable, 1.0 if predominantly residential or industrial lane use, 0.0 otherwise,

- Countywide property damage only percentage, %, and
- Interactions between each of the above variables, and every other variable.

A total of 45 variables were tested to determine if they were significantly related to collision frequency. Once an exhaustive combination of these variables was tested, the final equations included AADT, segment length, the two land use indicator variables and the interaction between approach density and the business/office indicator variable. Although driveway density and unsignalized public street approach density were tested separately, they were not found to be statistically significant individually. As a result, they were combined into approach density. The results seem to indicate that the different approach types have the same overall impact on collisions.

One assumption that is built into these models is that the length term is an adequate representation of exposure. Since a raised median cross-section forces additional travel distance and U-turns, the length term may underestimate exposure in the raised median model. However, since the number of vehicles that will U-turn is a likely a small percentage of total traffic and since AADT is such a rough approximation of exposure, the additional exposure due to U-turning should not be a concern.

Model 2. Final Collision Prediction Model

$$C_{RM} = ADT^{1.327} Len^{0.7233} e^{(-16.6814-0.8463 I_{b/o}-0.6968 I_{r/i}+0.0132(AD) I_{b/o})}$$
$$C_T = ADT^{1.5829} Len^{0.8902} e^{(-21.2535+0.008(AD) I_{b/o})}$$

where,

C_{RM} = annual mid-signal collision frequency for raised median sites;

C_T = annual mid-signal collision frequency for TWLTL sites;

AADT = annual average daily traffic demand, vpd;

Len = segment length, feet;

AD = approach density (two-way total), approaches/mile;

$I_{r/i}$ = indicator variable for residential or industrial land uses (1.0 if res /ind; 0.0 otherwise);

$I_{b/o}$ = indicator variable for business or office land uses (1.0 if bus/office; 0.0 otherwise)

As evidenced from the above equations, fewer terms were found to be significantly related to collisions for these models than in the Bonneson and McCoy models. However, the general form is the same and the coefficients are very similar to the Bonneson and McCoy models. Additionally, there are fewer terms in the TWLTL model than in the raised median model. All terms in the model were significant at the 95% confidence level, as shown in Table 7.

Table 7. SAS® Estimates and p-values for Models

Parameter	Raised Median		TWLTL	
	Estimate	p-value	Estimate	p-value
Intercept	-16.6814	<0.0001	-21.2535	<0.0001
log(ADT)	1.327	<0.0001	1.5829	0.0001
log(Length)	0.7233	<0.0001	0.8902	<0.0001
$I_{b/o}$	-0.8463	0.005	-	-
$I_{r/I}$	-0.6968	0.0153	-	-
(AD)*$I_{b/o}$	0.0132	0.0078	0.008	0.0015
Scale	1.5749	-	2.2878	-

The PDO factor did not have a significant impact on collisions. This result is not unanticipated. Since the PDO term accounts for differences in collision reporting practices, it is likely that this term would not vary drastically over the state of North Carolina. Countywide PDO percentages in North Carolina for the time period that was studied vary from 57 to 76%. While this is a fairly large range, most values are concentrated in the range of 60% to 70%, as shown in Figure 21.

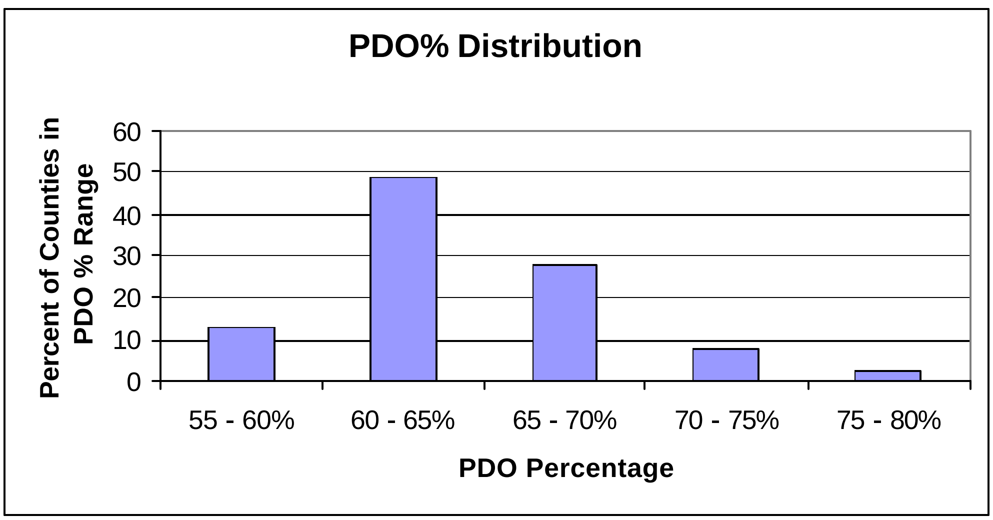


Figure 21. Distribution of Property Damage Only Percentages

Since the estimates for the length and AADT terms in the models were not 1.0, the assumption that they are not linearly-related to collisions is shown to be valid.

The signs on all of the terms are rational. The positive signs for the AADT and length terms indicate that collisions increase with exposure, which is logical. Since the AADT and length terms for the TWLTL model are larger than those for the raised median model, it appears that exposure has a greater impact on safety on TWLTL segments than on raised median segments. Since the intercept terms are negative, they serve to reduce the collision estimate. In this case, since the TWLTL intercept estimate is more negative than the raised median estimate, this term reduces the TWLTL prediction more than the raised median prediction. This term does not seem to give much insight but is rather a byproduct of the model form. The TWLTL model does not have the two independent land use indicator terms that the raised median model contains. These terms in the raised median model again reduce the collision prediction since their estimates are negative. When a land use is not predominant this term cancels out of the equation. Both models have a term that accounts for approach densities in the case of predominantly business or office land use. Both estimates for this term are positive, indicating that collisions increase with approach density. Since the estimate for this term in the TWLTL model is less than that for the raised median model, it appears that each additional approach has a slightly larger safety impact on raised median segments than on TWLTL segments, perhaps because the raised median segments with few approaches were so much safer. The SAS® output for the calibration of the models can be found in Appendix N.

MODEL VALIDATION

The next step was to validate these models. Goodness of fit was assessed for both the calibration sites and the validation sites. Ideally, the fit of the two sets of data would be the same. This outcome would indicate that the models can predict collisions for future sites just as well as it can for those sites used to create the model. The goodness of fit was performed in the same way as it was for the Bonneson and McCoy models. Figures 22 and 23 show goodness of fit plots for the raised median calibration and validation sites, respectively.

For both the calibration and the validation sites, the R^2 and intercept values are an improvement over those for the Bonneson and McCoy models, as shown in Table 8. However, the slope values are slightly worse than those for the Bonneson and McCoy models and both were significantly different from 1.0 at the 95% confidence level. The intercept for the calibration sites was also significantly different from 0.0 at the 95% confidence level. However, the intercept value for the validation sites gave a p-value of 0.09 and is therefore not significantly different from zero at the 95% confidence level.

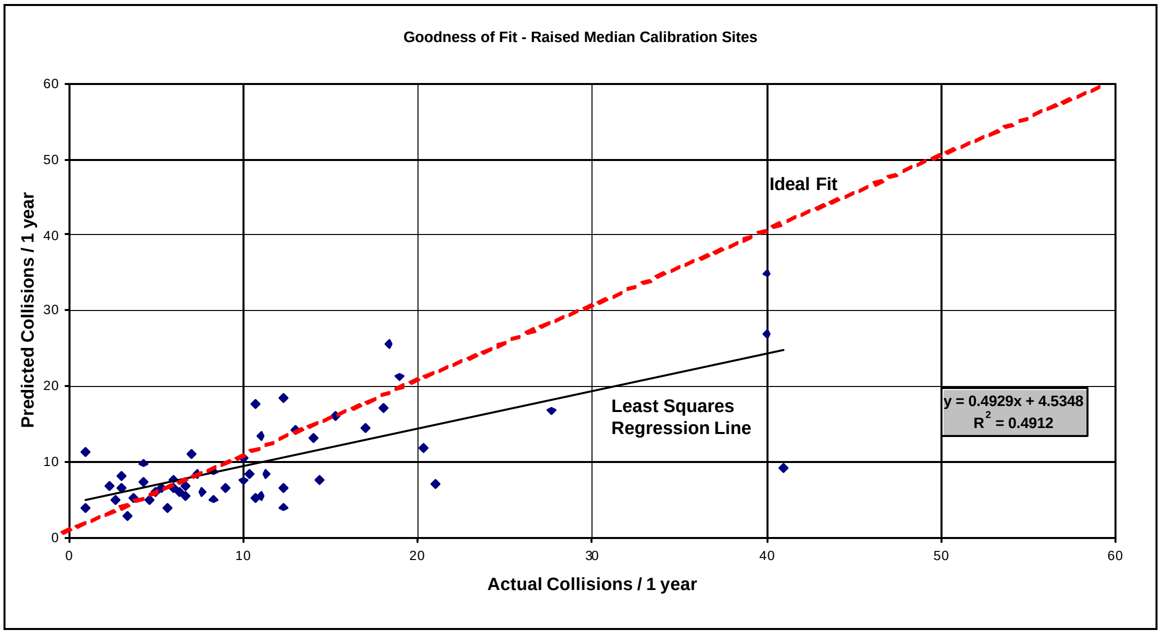


Figure 22. Goodness of Fit for Raised Median Calibration Sites

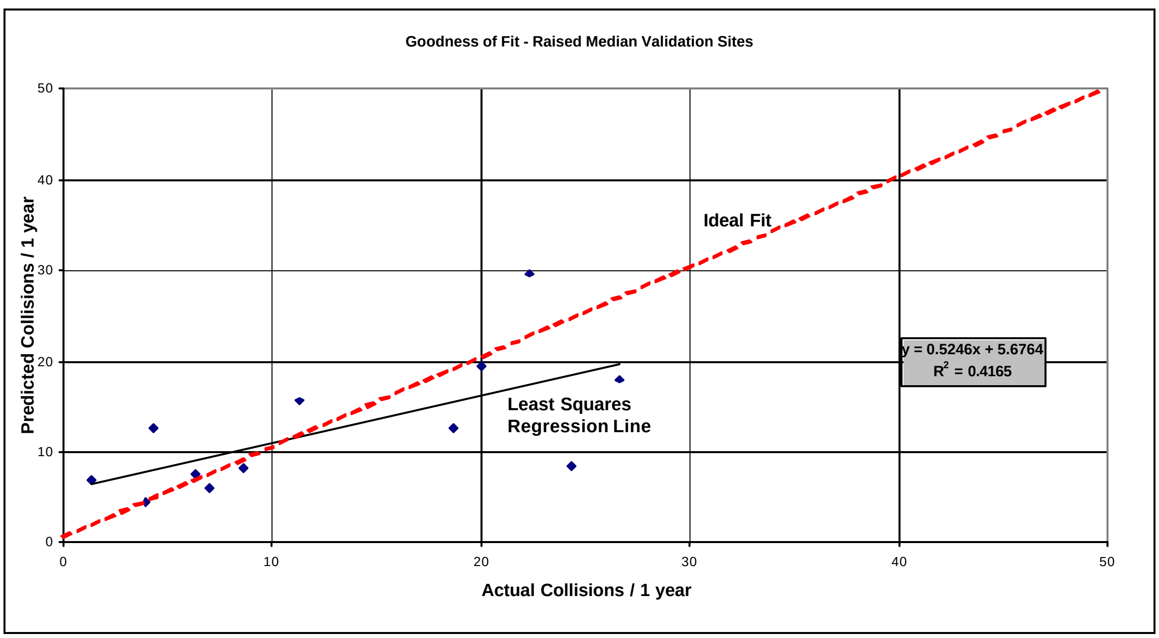


Figure 23. Goodness of Fit for Raised Median Validation Sites

Table 8. Fit of Raised Median Model to Collected Data

Goodness of Fit Parameters - Raised Median							
	R^2	Slope			Intercept		
		Value	p-value	significantly different from 1.0	Value	p-value	significantly different from 0.0
Calibration	0.4912	0.4929	<0.0001	yes	4.5347	<0.0001	yes
Validation	0.4165	0.5245	0.036	yes	5.6763	0.09	no

Due to the highly variable nature of collision data, R^2 values greater than 0.4 are generally considered acceptable. The R^2 values of 0.4912 and 0.4165, while not ideal, indicate that this model is more accurate for North Carolina segments than the Bonneson and McCoy model, which yielded an R^2 value of 0.1643. Most importantly, the calibration and validation sites give similar values for each of these measures of fit. As expected, the calibration sites have a better R^2 value; however, the validation sites are remarkably close. This finding is important in that, for future sites, the model should predict collisions just as well as it does for those used to calibrate the model.

A similar comparison was conducted for the TWLTL sites. The goodness of fit parameters from this comparison are shown in Table 9 and the plots are shown in Figures 24 and 25.

Table 9. Fit of TWLTL Model to Collected Data

Goodness of Fit Parameters - TWLTL							
	R^2	Slope			Intercept		
		Value	p-value	significantly different from 1.0	Value	p-value	significantly different from 0.0
Calibration	0.3422	0.3508	<0.0001	yes	6.9938	<0.0001	yes
Validation	0.3397	0.23021	<0.0001	yes	8.8829	<0.0001	yes

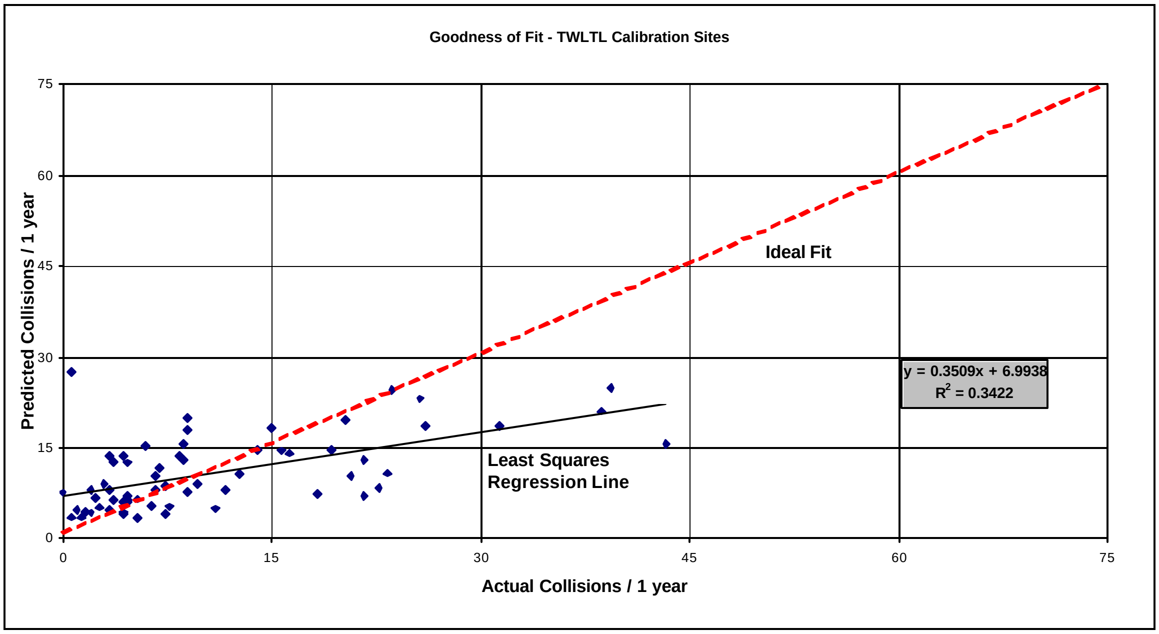


Figure 24. Goodness of Fit for TWLTL Calibration Sites

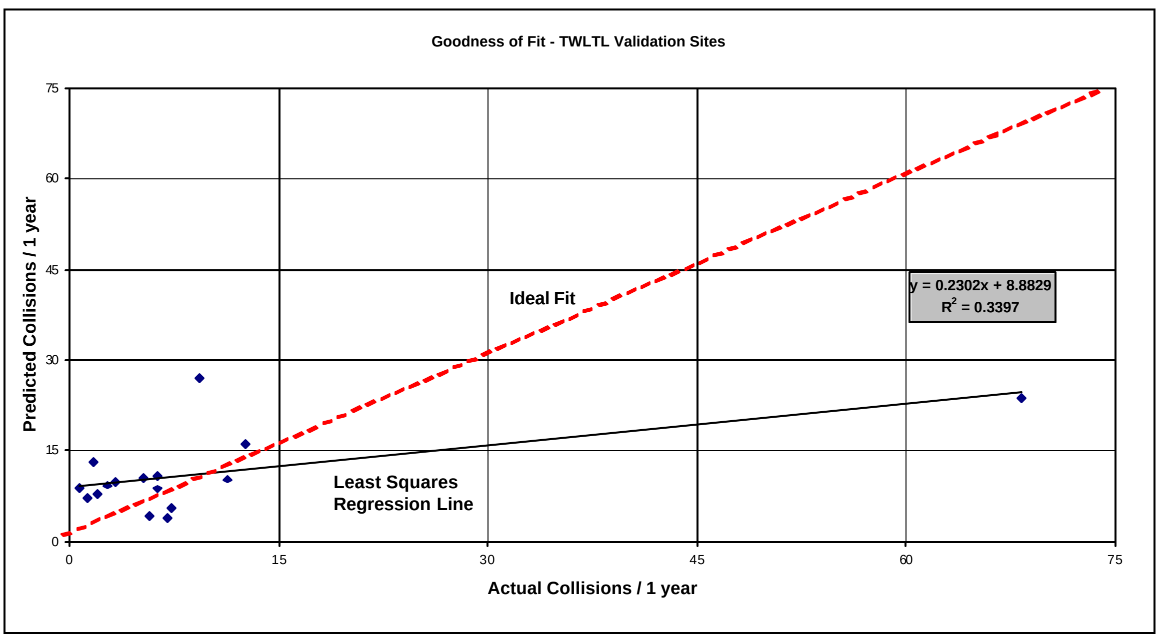


Figure 25. Goodness of Fit for TWLTL Validation Sites

While the TWLTL model does not fit the data as well as does the raised median model, it is still an improvement over the fit using the Bonneson and McCoy model. The R^2 values for the calibration and validation sites are 0.3422 and 0.3397, respectively. The Bonneson and McCoy model yielded a value of 0.2944. Additionally, the intercept values are closer to zero for this model than for the Bonneson and McCoy model, even though they are both significantly different from zero. The slope values, on the other hand, are farther away from 1.0 than that found for the Bonneson and McCoy model and are both significantly different from 1.0. Again, the measurements of goodness of fit are quite similar for the calibration and validation sites, indicating that the model should do as well predicting collisions for future segments as it has done for those that were used to calibrate it.

One way to ensure that there is no bias in the models is to plot the residuals versus the predicted collision values [31]. Residuals are the differences between predicted collisions and observed collisions. These plots should not exhibit any obvious patterns and should have approximately the same amount of scatter over all predicted values. The residual plots for the raised median and TWLTL models are shown in Figures 26 and 27, respectively. In these plots, "pred" is the predicted number of collisions and "Resdev" is the residual between predicted and actual collisions. There is no evidence of a non-random pattern in these graphs. The data points seem to be centered around zero and there does not seem to be an increasing or decreasing trend in scatter as the number of predicted collisions increases.

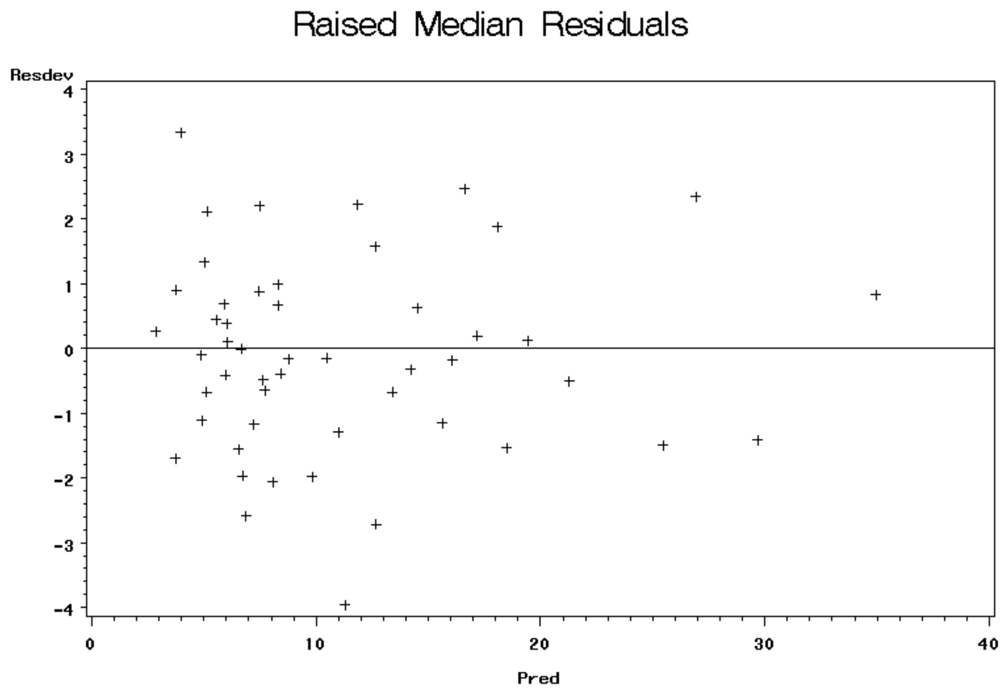


Figure 26. Plot of Residuals for the Raised Median Model

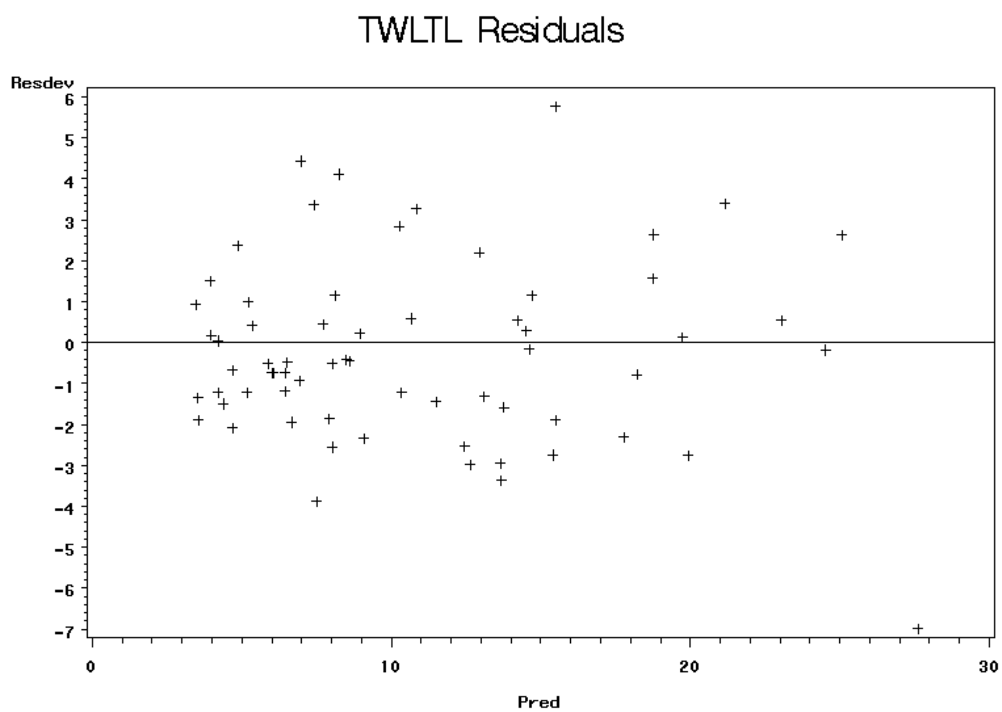


Figure 27. Plot of Residuals for the TWLTL Model

It should be noted that in several of the goodness of fit graphs one or more outliers can be observed. Recalibration without these outliers was attempted; however, the recalibrated models did not fit the data as well as the original models. As a result, the recalibrated models were discarded. Additionally, the models were recalibrated without the eight segments that have an undeveloped land use. Again, the fit of these models was poor and they were subsequently discarded.

DATA RANGES

With the models validated, it can now be determined under what conditions each median treatment is preferable from a safety standpoint. It is important at this point to ensure that any analysis is confined to the range of data used to calibrate the models. Extrapolation outside of the range of collected data could yield unreliable results. As a result, what follows is a brief summary of the distribution of values for each of the key variables that were collected.

Figure 28 shows the ranges of AADT values for segments in this study. Most of the sites fall in the range of 20,000 to 40,000 vpd. No sites were used with an AADT less than 20,000 vpd, therefore the model will not accurately predict collisions for AADTs below this boundary. The model is also unlikely to give reliable predictions for AADTs greater than 50,000 vpd due to the small number of sites, only three, with AADT values greater than 50,000 vpd. Additionally, there are very few TWLTL sites with AADTs greater than 35,000 vpd, so the model should be applied cautiously for TWLTL segments above this limit.

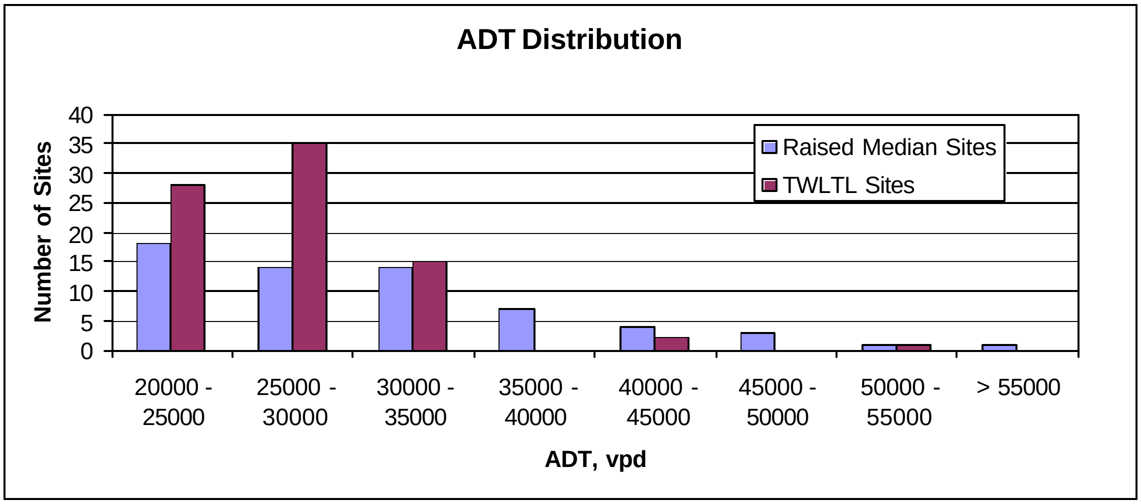


Figure 28. Range of AADT Values

Figure 29 shows the range of segment lengths collected in this study. No sites were visited with segment lengths less than ¼ mile, so the model should not be used for segments shorter than 1320 feet. Additionally, the small number of sites with lengths greater than 6,000 feet, only six, means that the models may be unreliable for segments much longer than one mile. Instead, for longer segments, the roadway should be split up at logical, convenient points that are less than or equal to one mile in length and analyzed separately.

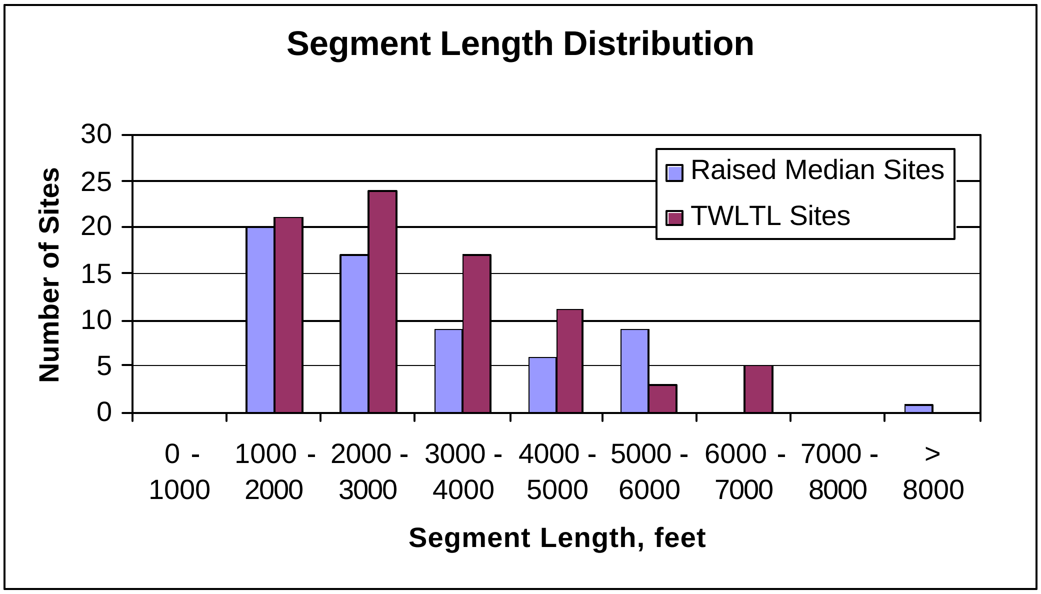


Figure 29. Range of Segment Lengths

Figure 30 indicates how many sites had each of the predominant land uses. Clearly, the undeveloped land use is the scarcest with only four sites from each median treatment with this type of land use. Results for this land use type are not reliable. As a result, the model should not be used for completely undeveloped land uses.

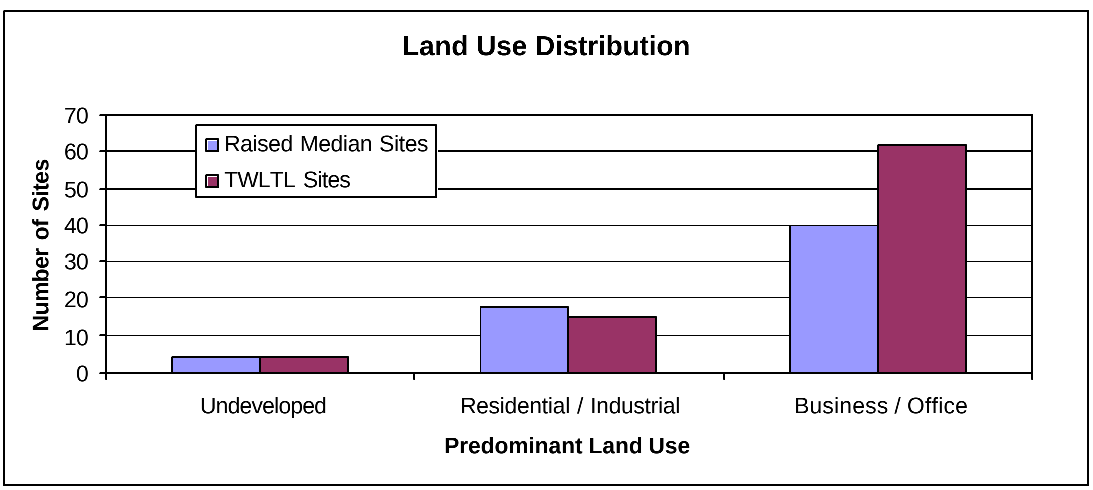


Figure 30. Distribution of Predominant Land Use

Figure 31 shows the distribution of approach densities for this study. Approach density is the two-way total of all types of approaches including driveways and unsignalized public street approaches. Clearly, the TWLTL sites and the raised median sites have very different distributions for this parameter. The majority of the raised median sites have approach densities of 40 per mile or less. The TWLTL sites, on the other hand, exhibit an approximately normal distribution, with the largest number of sites having 40 to 50 approaches per mile. The raised median model is unlikely to accurately predict collisions at approach densities greater than 90 approaches per mile and the TWLTL model is likely unreliable at approach densities greater than 120 approaches per mile.

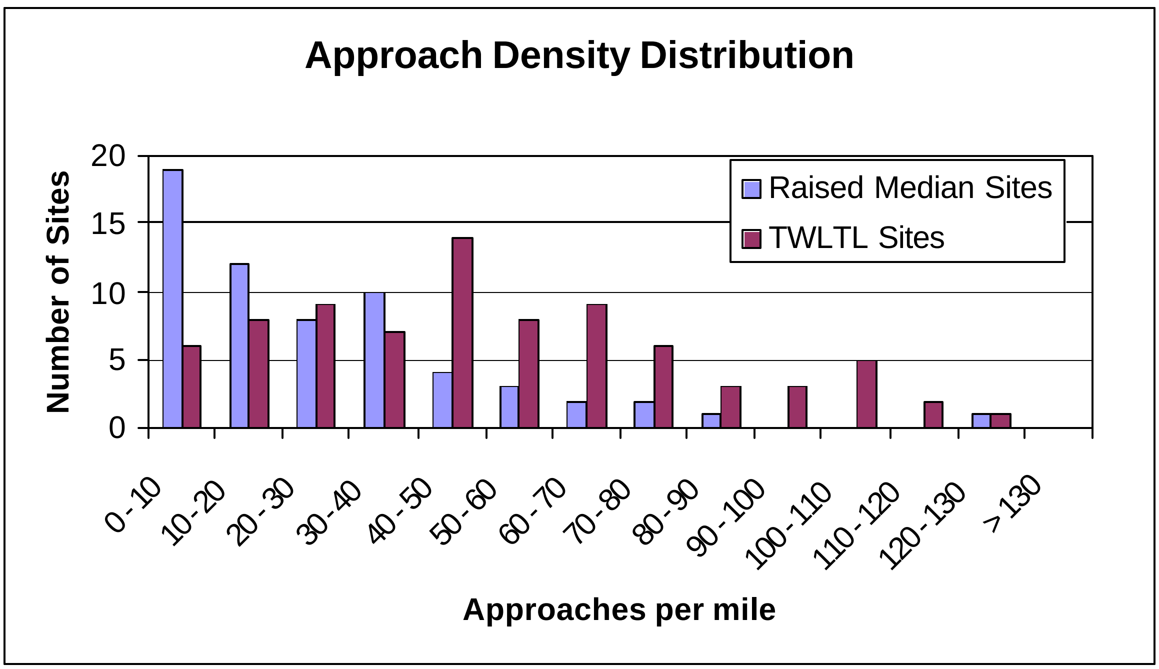


Figure 31. Range of Approach Densities

The drastically different approach density distributions for these two cross-sections could be due to a variety of causes. It is possible that it is NCDOT's policy to install raised medians on segments with lower approach densities and TWLTLs on segments with higher approach densities. Alternatively, higher approach densities may naturally occur on segments with TWLTLs due to greater access.

SENSITIVITY ANALYSIS

The models can now be compared within the constraints of the ranges mentioned above. AADT values from 20,000 to 50,000 vpd, segment lengths from 1,320 to 6,000 feet, and approach densities from zero to 90 approaches per mile were compared for both business/office and residential/industrial land uses. Since the most common segment length was approximately ½ mile, this length was used for all analyses. Since the parameter estimate for the TWLTL model was higher than that for the raised median model, the raised median design will become increasingly safer than the TWLTL design with higher segment lengths.

Figure 32 shows the comparison between the model results for a ½ mile segment with predominantly residential/industrial land use over all values of approach density. As shown below, the raised median segment is associated with fewer collisions over all AADT values. At an AADT of 20,000 vpd, the raised median has almost no safety advantage over a TWLTL. As AADT increases, so does the safety margin between the two cross-sections.

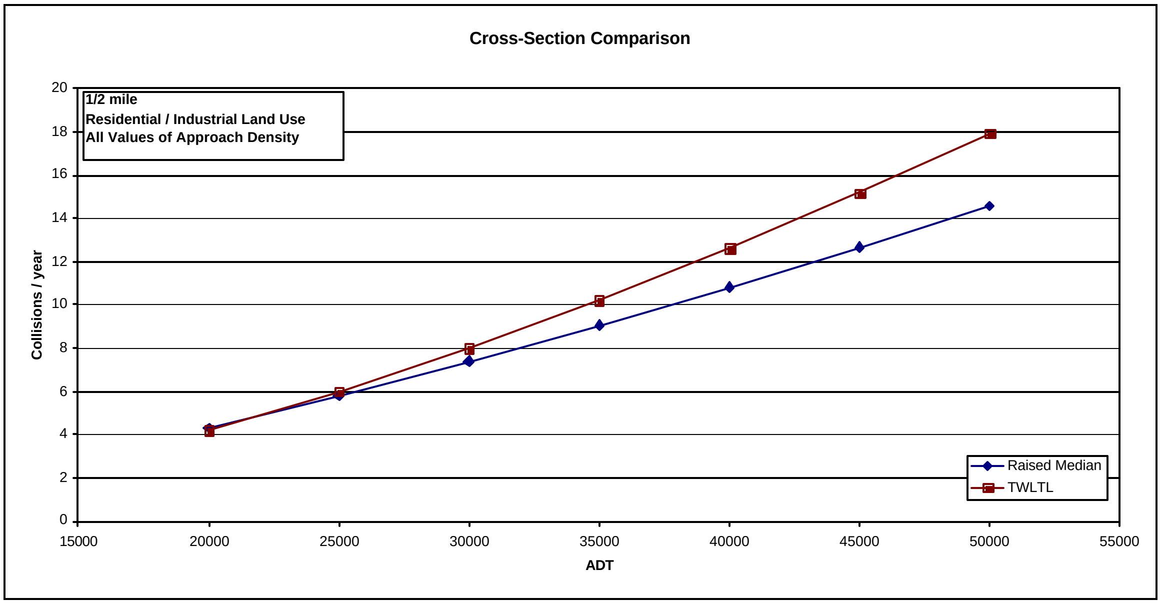


Figure 32. Cross-Section Comparison for Residential/Industrial Land Use

Figure 33 shows the same comparison for predominantly business/office land uses for an approach density of 25 approaches per mile. The results are similar to the previous comparison. Again, the two cross-sections yield nearly identical collision predictions at low AADTs, with the raised median having an increasing safety advantage at higher traffic volumes.

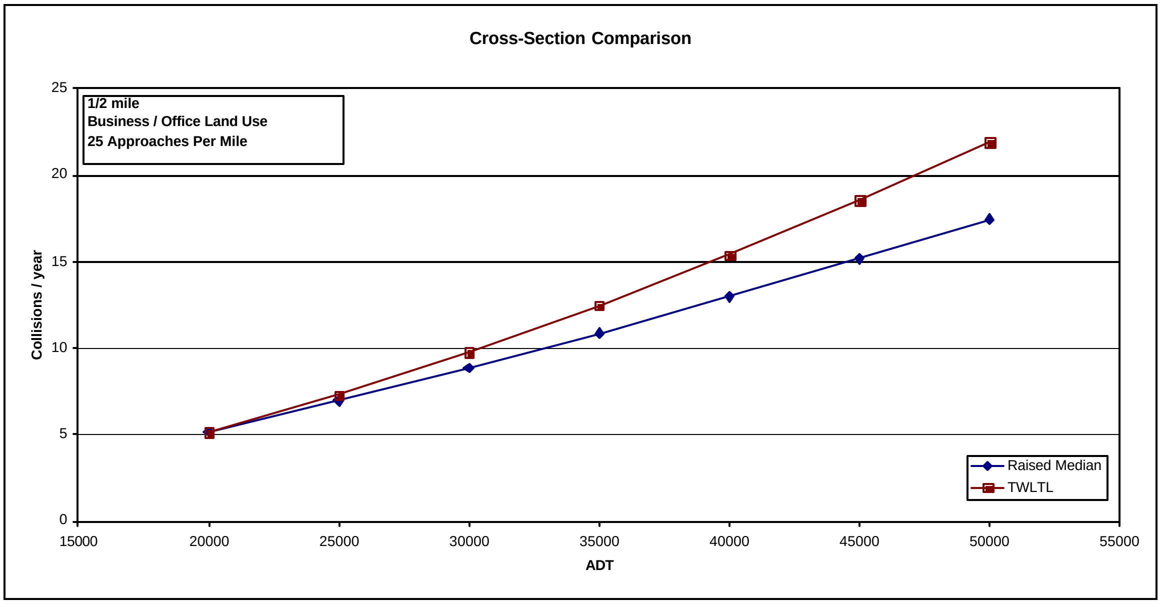


Figure 33. Cross-Section Comparison for Business/Office Land Use, 25 Approaches Per Mile

For business and office land uses, as approach density increases, the safety margin between the two cross-sections narrows. At approximately 50 approaches per mile, the collision prediction around 35,000 vpd is identical, as shown in Figure 34. At lower AADTs, the TWLTL cross-section is associated with slightly fewer collisions and at higher AADTs, the raised median is associated with fewer collisions. As mentioned previously, the results are slightly different for various segment lengths. For example, at this same approach density, for a one mile segment, the two cross-sections yield identical predictions at 20,000 vpd and the raised median is associated with fewer collisions at all other traffic volumes.

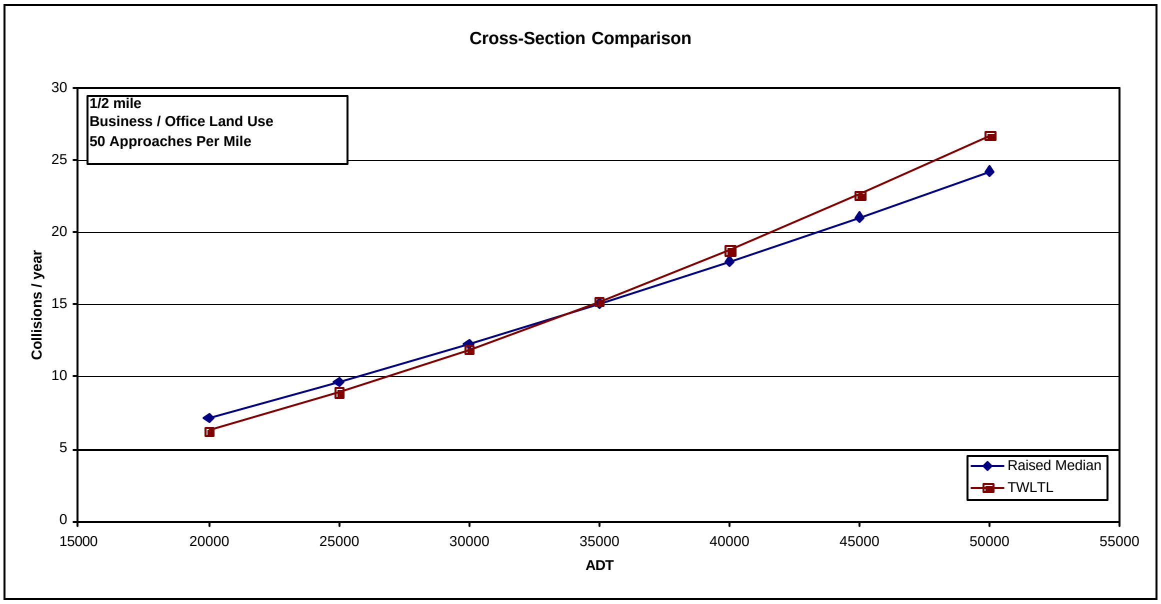


Figure 34. Cross-Section Comparison for Business/Office Lane Use, 50 Approaches per Mile

The highest value of approach density that is considered to be within the limitations of the collected data is 90 approaches per mile. At this approach density, for business and office land uses, the TWLTL cross-section is associated with four or five fewer collisions per year at all traffic volumes, as shown in Figure 35. Again, results are slightly different for a longer segment length. For a one mile segment with this same approach density, the TWLTL cross-section is associated with three or four fewer collisions per year at low traffic volumes but is nearly identical to the raised median cross-section at higher traffic volumes.

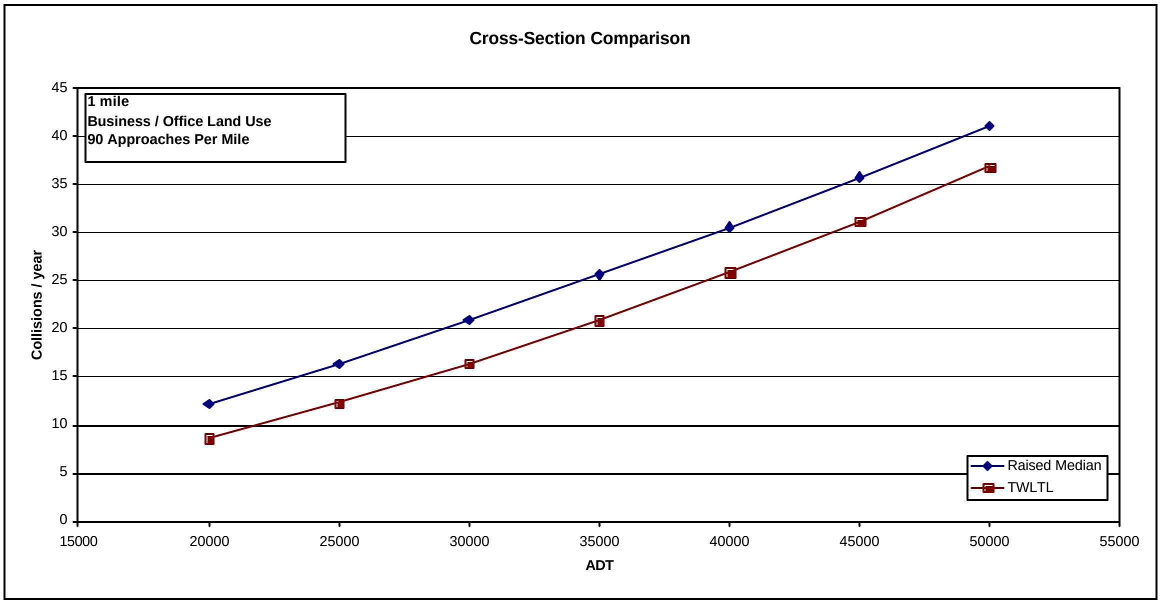


Figure 35. Cross-Section Comparison for Business/Office Land Use, 90 Approaches per Mile

Figure 36 presents a simpler way to visualize the conditions under which each of the cross-sections is preferable from a safety standpoint for business and office land uses. The dashed line in the middle of the graph represents the values of AADT and approach density for which the two designs will yield identical collision predictions. Below this line, the raised median cross-section is associated with fewer collisions. Above the dashed line, the TWLTL cross-section is associated with fewer collisions. The number in parenthesis between each of the contour lines represents the collision savings per ½ mile segment per year that can be attributed to the cross-section that precedes it. From this graph, it is easy to see that the largest differences between these designs occur at very high approach densities and at high volumes with very low approach densities. Remember that Figure 36 only applies to business and office land uses.

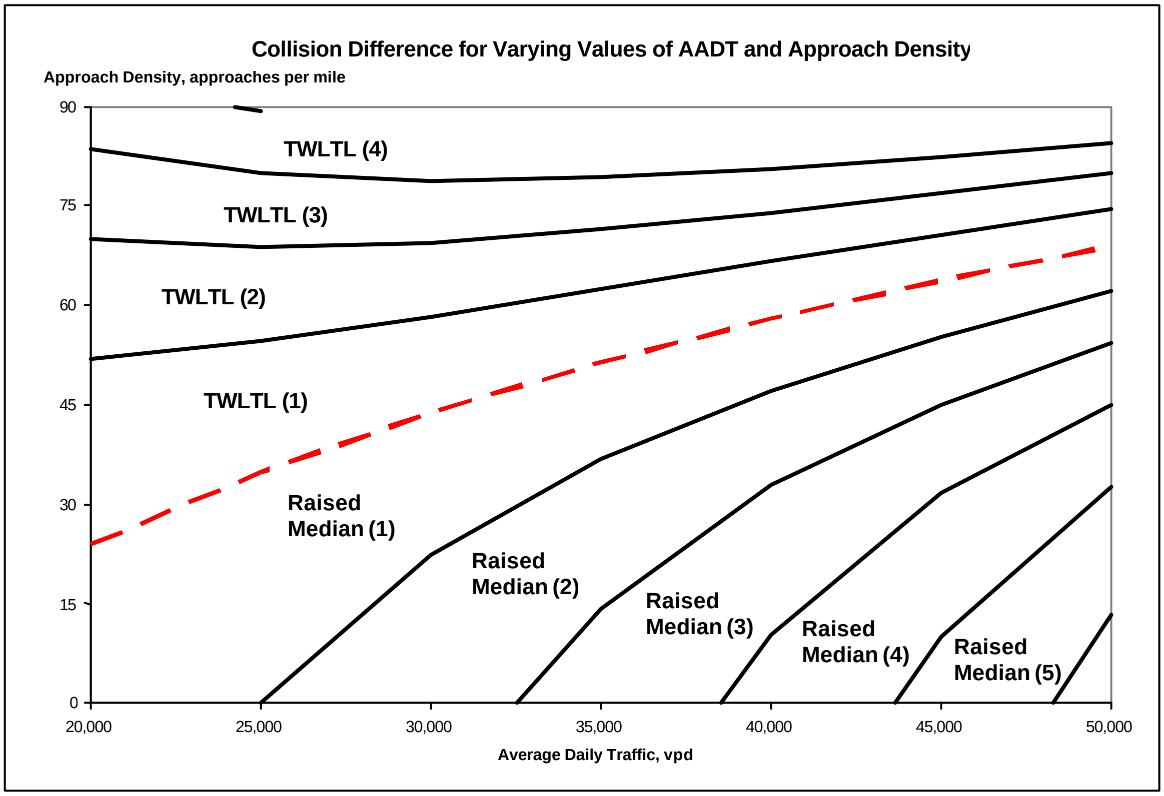


Figure 36. Collision Comparison for ½ mile Segment of Business / Office Land Use

Figure 37 shows the range of data collected in this study in terms of AADT and approach densities. It should be noted that there was little data collected in the upper right quadrant of this graph. Additionally, there were very few TWLTL segments with AADTs greater than 35,000. As a result, the graph in Figure 37 should be applied cautiously to segments with both high approach densities and high traffic volumes. Application of the models to TWLTL segments with high volumes should also be avoided where possible.

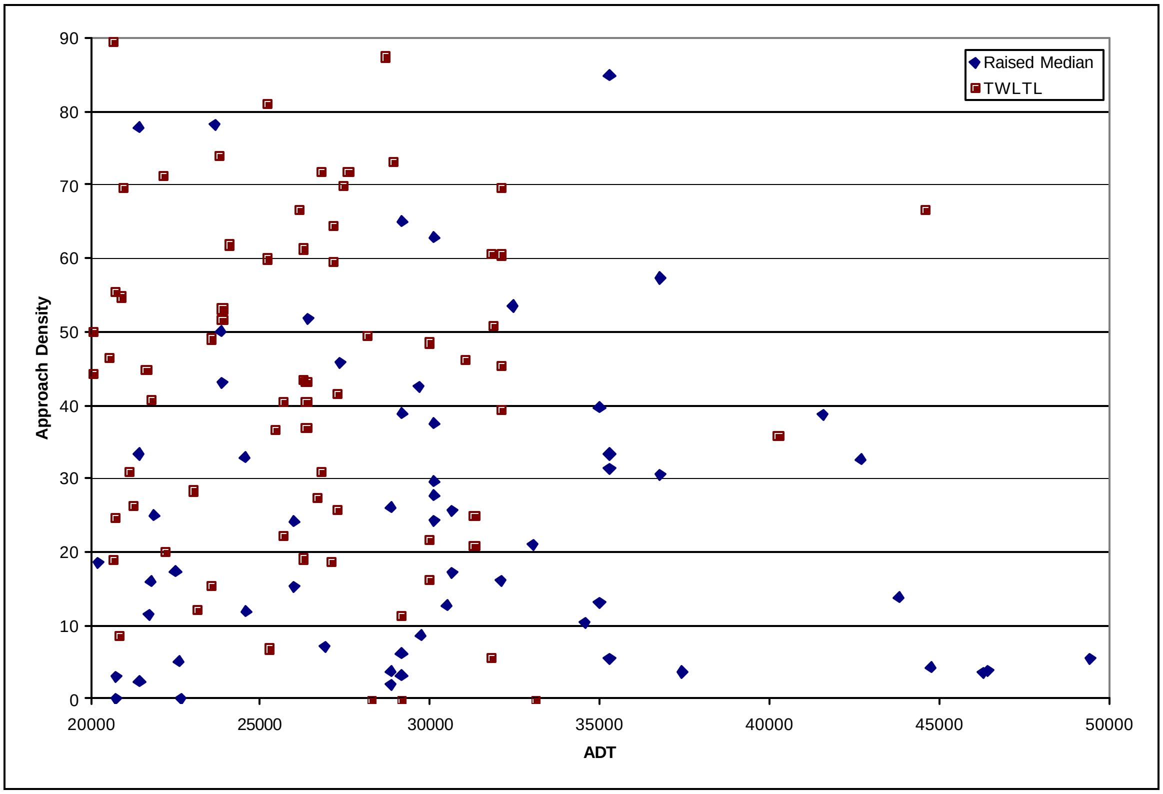


Figure 37. Distribution of AADT and Approach Density Values

Due to the small number of segments with completely undeveloped land uses, the models should not be used for this land use. If a decision needs to be made for a segment that is entirely undeveloped, anticipated future land use should be used for the purposes of the model. It is interesting to note that the Bonneson and McCoy model yield identical predictions for the TWLTL and raised median cross-sections in the case of undeveloped land uses [1].

One final point of interest is the marginal effect of one driveway or unsignalized public street approach per mile on a collision prediction. There is no impact due to additional approaches if the land use is not predominantly business or office. In the case of a business or office land use, however, the collision prediction is multiplied by 1.0 plus approximately 0.02 per approach per mile for raised median segments or 0.01 per approach per mile for TWLTL segments. In other words, for each additional approach per mile, there is a 2% collision increase per year on raised median segments and a 1% increase per year on TWLTL segments. This factor is not exact but gives a good idea of the impact of an additional driveway or unsignalized public street approach per mile. It is also interesting to note that an additional approach per mile impacts raised median segments twice as much as it impacts TWLTL segments. Due to this factor, as approach densities reach very high levels, the other benefits of a raised median will be canceled out by this term and the TWLTL will become the safer option. Analysts should keep in mind that very high approach densities are relatively unsafe regardless of the median treatment.

MODEL APPLICATIONS

The following provides some guidelines in applying the collision models we developed:

1) The first step is to split up the arterial in question into segments. Segments should be between ¼ mile and one mile long. Appropriate segment endpoints include signalized intersections, locations where cross-sections change and unsignalized intersections. No signalized intersections are permitted within a segment.
2) The next step is to determine the initial length, in feet, of the segments. For the purposes of this study, length was determined by mileposts and confirmed using odometer readings.

3) The segment lengths of those segments with a signalized intersection at one or more endpoints should next be reduced by 150 feet per signalized endpoint.

4) The next step is to determine the predominant land use of the segments. Predominant land use for this purpose is defined as the land use that comprises at least 50% of the segment. It two land uses are approximately equal, the land use that would generate more trips should be chosen as predominant. If the segment is undeveloped, future land use should be assumed for using the models. Figures 38 through 41 are pictures of typical residential, industrial, business, and office land uses, respectively, for use in determining which land use should be selected. If the land use is business or office, $I_{b/o}$=1.0 and $I_{r/i}$=0.0. If the land use is residential or industrial, $I_{r/i}$ =1.0 and $I_{b/o}$ =0.0.

Figure 38. Typical Residential Land Use

Figure 39. Typical Industrial Land Use

Figure 40. Typical Business Land Use

Figure 41. Typical Office Land Use

5) The next step is to determine the approach density of each of the segments. Since there was found to be no difference between driveways and unsignalized public street approaches, no distinctions need to be made between different approach types. First, count the total number of approaches on each segment on both sides of the roadway. Next, divide the two-way total number of approaches by the segment length in miles. This is the approach density.

6) Finally, the Average Daily Traffic (AADT) of the segment needs to be determined. If the purpose of using the models it to determine which design is appropriate for a future time, AADT estimates for that future year will need to be

established. If the desire is to predict collisions for the current year, NCDOT AADT maps can be used to estimate the traffic volume.

7) The models can now be applied using the data collected in the first six steps. The following example scenario illustrates proper application of the models.

EXAMPLE SCENARIO

Segment Characteristics:

- ½ mile long
- 40,000 vehicles per day
- 20 approaches (two-way total)
- 50% business land use, 50% residential land use

Determining Parameters:

- Len = 2640 feet
- AADT = 40,000 vpd
- DD = (20 approaches / ½ mile) = 40 approaches per mile
- Since there is an equal land use split between business and residential land use, the business land use is considered dominant.
- $I_{r/i} = 0.0$, $I_{b/o} = 1.0$

Model Form:

$$C_{RM} = ADT^{1.327} Len^{0.7233} e^{(-16.6814 - 0.8463 I_{b/o} - 0.6968 I_{r/i} + 0.0132 (AD) I_{b/o})}$$

$$C_{T} = ADT^{1.5829} Len^{0.8902} e^{(-21.2535 + 0.008 (AD) I_{b/o})}$$

Applying Models:

Raised Median Model

- $C_{RM} = (40{,}000)^{1.327}(2{,}640)^{0.7233}e^{(-16.6814-0.8463*(1.0) - 0.6968*(0.0) + 0.0132(40)(1.0))}$
- $C_{RM} = 1{,}279{,}186*298*\ e^{(-16.6814 - 0.8463 + 0.528)}$
- $C_{RM} = (381{,}750{,}600)*\ e^{(-16.9997)}$
- $C_{RM} = 15.81$

TWLTL Model

- $C_T = (40{,}000)^{1.5829}(2{,}640)^{0.8902}e^{(-21.2535 + 0.008(40)(1.0))}$
- $C_T = 19{,}257{,}538*1111*e^{(-21.2535 + 0.32)}$
- $C_T = (21{,}404{,}869{,}084)*e^{(-20.9335)}$
- 17.35

Results:

The raised median and TWLTL designs will likely result in approximately 16 and 17 collisions per year, respectively. As a result, the raised median design is slightly better than the TWLTL design, in terms of safety.

INTERSECTION RESULTS

OPERATIONAL IMPACTS OF U-TURNS

Fourteen sites were used in the operational study (see Table 10). 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 the below section entitled "Calculation of Regression Variables".

The team 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. The following section details the process the team used to calculate these two values used in the regression. Table 10 shows a summary of the values for each site.

One note should be made about queue eligibility in these calculations. Although the team observed queues of many different lengths, a queue was only considered eligible for calculations if it contained five or more vehicles. Since we wanted to measure only the effect of U-turns on saturation flow rate, we 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 10. 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.

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 the section entitled "Hypothetical Queues". The U-turn percentages in Table 10 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. We could not measure the comparison saturation flow rate for site 206 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. The team 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 the team conducted to obtain data points for the regression analysis. The queue in Table 11 is similar to queues obtained during field headway measurements.

Table 11. 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 11 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 5 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. The team 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.

Factors Affecting Saturation Flow Reduction

Although the saturation flow adjustment factors in Table 10 seem to vary between sites based mainly on U-turn percentage, the team 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, the team 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 us examine the effect of other intersection characteristics. In Table 12, the team 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 12. 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 12 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 12. 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, the team 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. The insignificance of this factor 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.

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:

$$f_{uturn} = 1.0 - 0.0018*UTURN - 0.0015*UTURN*OVERLAP \qquad \text{(Equation 3)}$$

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 we 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, the team 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 42 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.

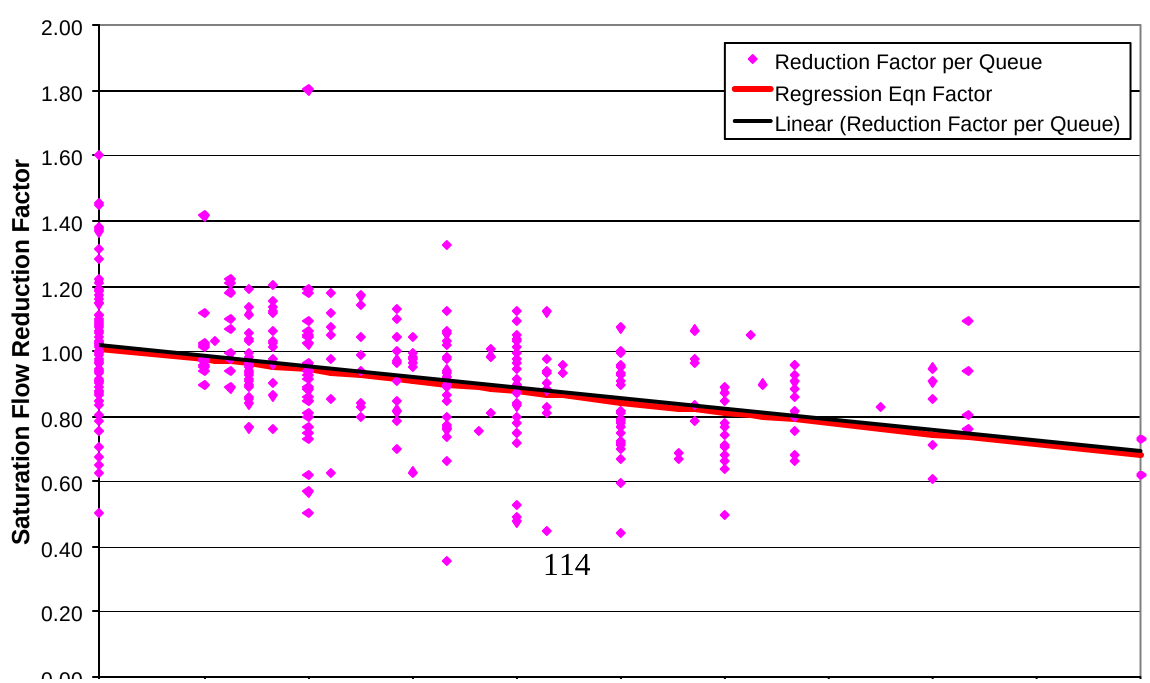


Figure 42 Plot of Saturation Flow Reduction Factor by Individual Queue

The team 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, the team wanted to try a multivariate regression to produce a better fit. The intersection characteristic that proved the most significant in Table 12 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 we 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 below entitled “Sensitivity Analysis of U-Turn Percentage on Lane Performance”. 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 [21,22]. In Figure 43, we compared our regression results to the results given by the other two methods when used on our 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. The team used the average RTOA volume from the 14 sites for the RTOA variable. For our analysis, the team plotted lines of the saturation flow reduction factor with and without protected right turn overlap.

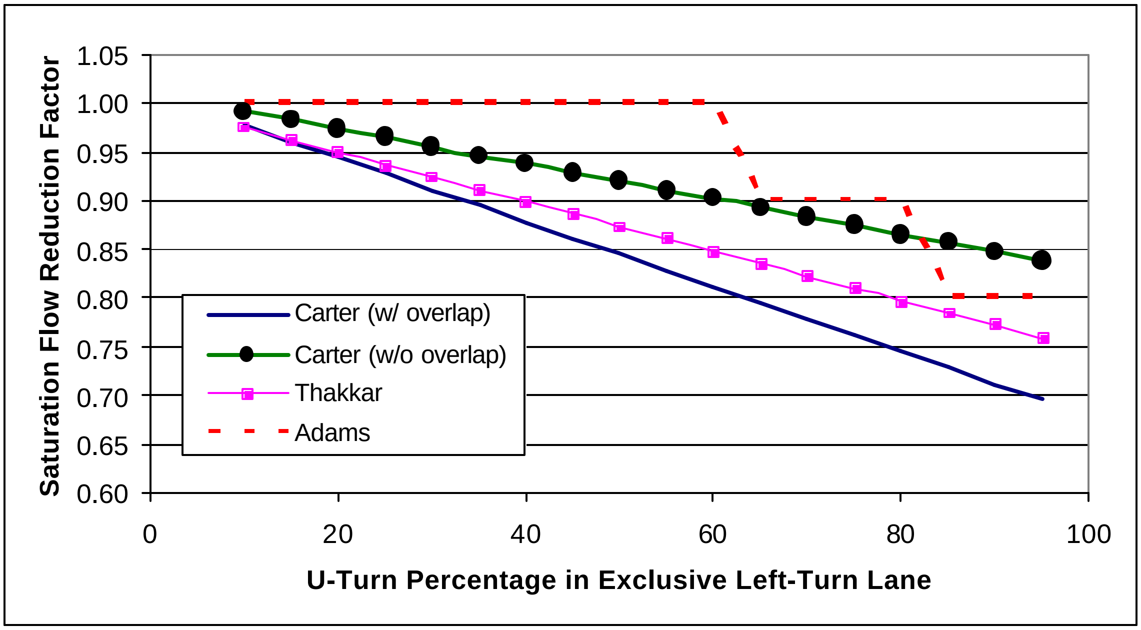


Figure 43. Comparison of Saturation Flow Reduction Studies

The overall trends of the three methods are similar, with Thakkar showing the closest results to our own. Her saturation flow reduction equation fell almost directly in between our two lines, though her results were closer to our 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 13 shows the predicted saturation flow reduction factors for common U-turn percentages, for intersections with protected right turn overlap and those without.

Table 13. 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

Individual Driver Behavior

The methodology used in this project to collect saturation flow data involved precise measurements of individual vehicle headways. 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, the team 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, the team 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 14 lists the category descriptions. The right column of Table 14 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 14. 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 15 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 15. 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 15. 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 14 for category descriptions

As can be seen clearly in Table 15, 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. The team did not analyze the effects of intersection characteristics, such as median width, on the headways of individual vehicles.

Table 15 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 16, the team divided the sites into four categories based on the type of conflicting right turn and the number of lanes receiving.

Table 16. Proportions of Comparison Headway by 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 14 for category descriptions

The values in Table 16 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 15.

Pursuant to the methodology described in the calculation of regression variables, the team 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 16 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 16 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 users to calibrate these models (and others) for U-turning vehicles based on validated data.

Hypothetical Queues

As previously mentioned, the headway data gathered for this project contain a high degree of detail pertaining to individual vehicles. The team 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 14). This knowledge of the average headway associated with each category gave us 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:

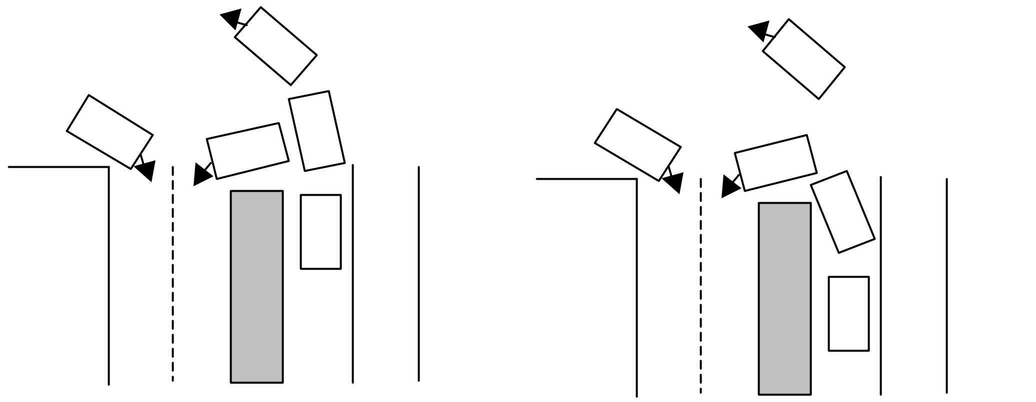
Queue Position	1	2	3	4	5	6	7	8	9	10
Movement (left or U-turn)	L	L	L	**U**	L	L	**U**	**U**	L	L

Consider that this queue occurred at Site 207. We can then use the headway data provided in Table 15 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 previously mentioned process, 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, the team 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 44). This delay causes headway measurements to increase, thereby decreasing the saturation flow rate. [None of the saturation flow intersections had median widths capable of stroing more than one vehicle. Additional observations are needed with wider medians to observe and measure headways for these conditions.]



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

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

Figure 44. 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 the section above entitled “Calculation of Regression Variables” 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 43. To serve as a visual representation of this averaging process, Figures 45 and 46 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 16. The data behind these graphs can be found in Appendix I.

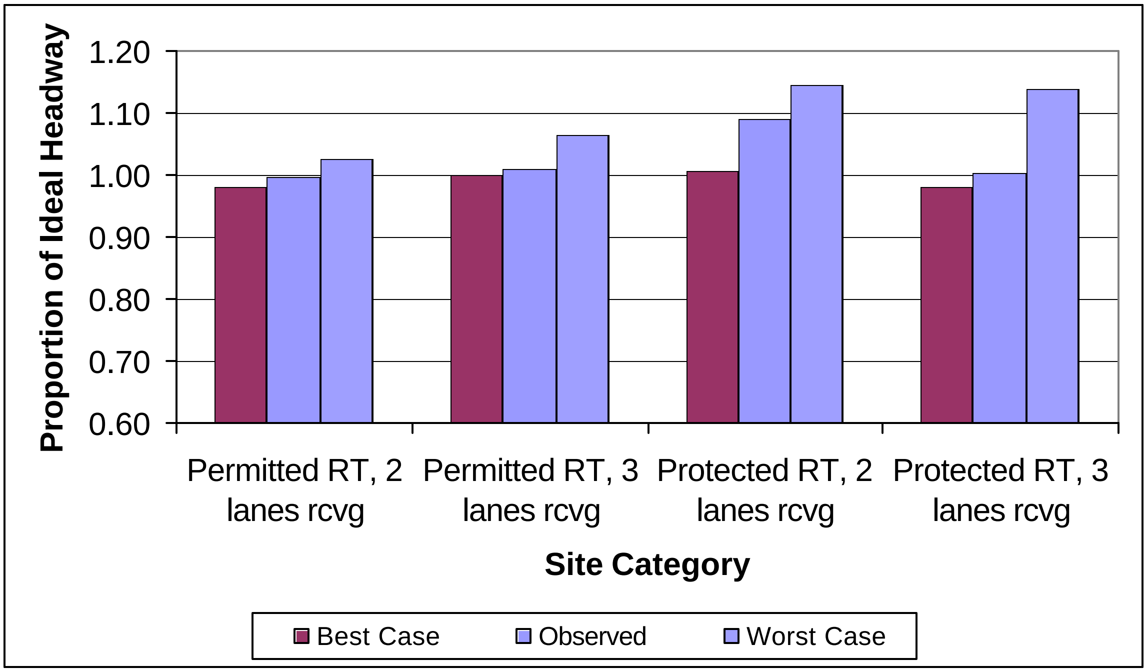


Figure 45. Comparison of Average Observed Headway to Best and Worst Case Scenarios for Queues with 20% U-Turns

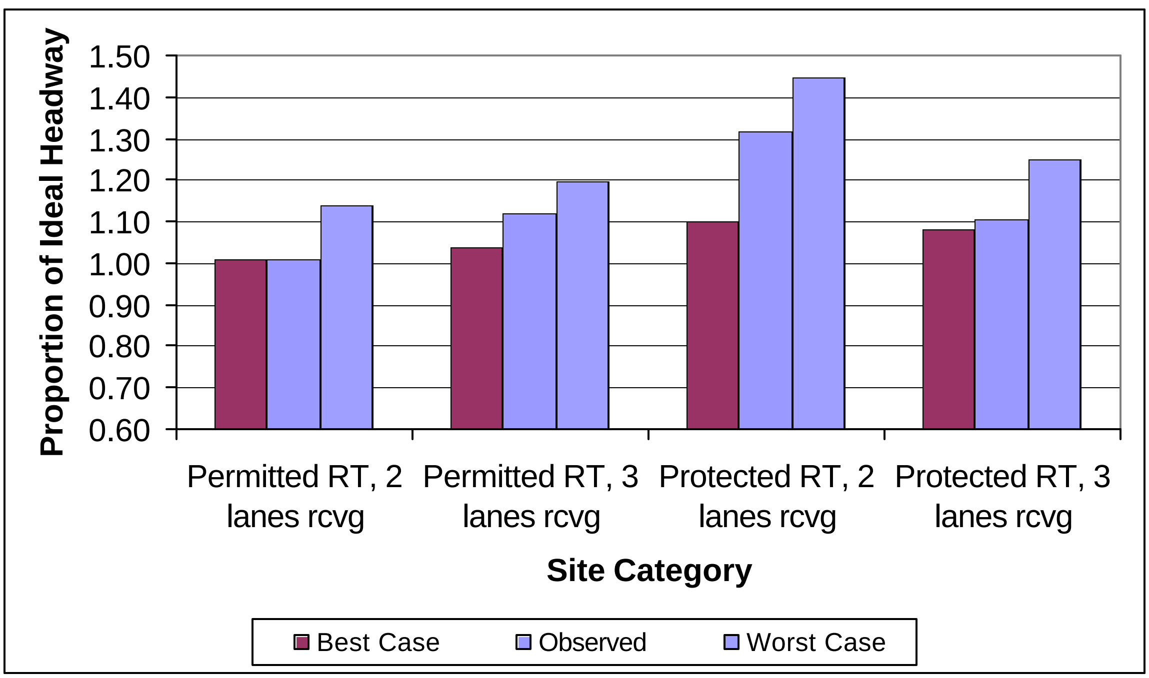


Figure 46. Comparison of Average Observed Headway to Best and Worst Case Scenarios for Queues with 50% U-Turns

Delay Data Results

One of the studies conducted on the 14 operational study sites was a control delay study. Using a Jamar electronic count board, an observer measured queued vehicles (fully stopped) in 15-second intervals during the entire data collection period. These queued delays were converted into average stopped delay using data reduction software and then further adjusted to provide average control delay. To determine the effects of U-turns on delay, the team compared this observed control delay to an estimate of control delay calculated with the Highway Capacity Software (HCS2000).

Given the volumes, signal timing, and other intersection characteristics observed in the field, the team calculated the estimated control delay per vehicle. Table 17 compares the average observed control delay to the HCS-calculated control delay for peak hour traffic.

Table 17. Comparison of Observed and Estimated Control Delay

Site	Delay in seconds per vehicle			Average Percentage U-turns	Conflicting Right Turn	No. Left Turn Lanes
	HCS Calculated Control Delay	Observed Control Delay	Difference (Obs-Calc)			
202	78.5	90.0	11.5	16	permitted	1
203	52.0	86.9	34.9	6	protected	2
204	72.1	83.7	11.6	14	permitted	1
205	67.1	107.6	40.5	41	protected	2
207	60.0	88.6	28.6	32	protected	2
210	41.8	56.9	15.1	15	protected	2
211	54.5	64.6	10.1	28	protected	1
212	70.4	86.0	15.6	27	permitted	1
213	84.7	96.1	11.4	34	permitted	2
215	64.5	87.2	22.7	52	protected	2
216	74.1	78.0	3.9	32	permitted	1
217	76.5	85.5	9.0	50	permitted	1
218	82.3	87.3	5.0	13	protected	2
Average	**67.6**	**84.5**	**16.9**			

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. The team 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 16.9 seconds. This is significant at a 95% confidence level. Similarly, t-test results indicate that grouping intersections into protected right turn overlap and permitted right turn on red overlap are both significant at a 95% confidence level. There does not seem to be a clear answer for why all observed delays are significantly different than the HCS calculated delays.

This comparison does indicate that the operational effects of U-turns are significant. Since the HCS procedure did not take U-turns into account with its delay estimation, its calculated delays were shown to be significantly lower than the observed delays for all sites with U-turns, regardless of right turn overlap phasing.

Sensitivity Analysis of U-Turn Percentage on Lane Performance

The multivariate regression equation presented in the section above entitled "Saturation Flow Adjustment Factor Determination by Regression" 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, the team 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 the section above entitled "Saturation Flow Adjustment Factor Determination by Regression".

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:

$$f^*_{uturn} = P_{uturn} * f_{uturn} + (1 - P_{uturn}) \qquad \text{(Equation 4)}$$

where:

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

f_{uturn} = adjustment factor calculated in Equation 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 48, the team 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 47 through 49 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 47 and 48. 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 49, which shows the one site in our 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.

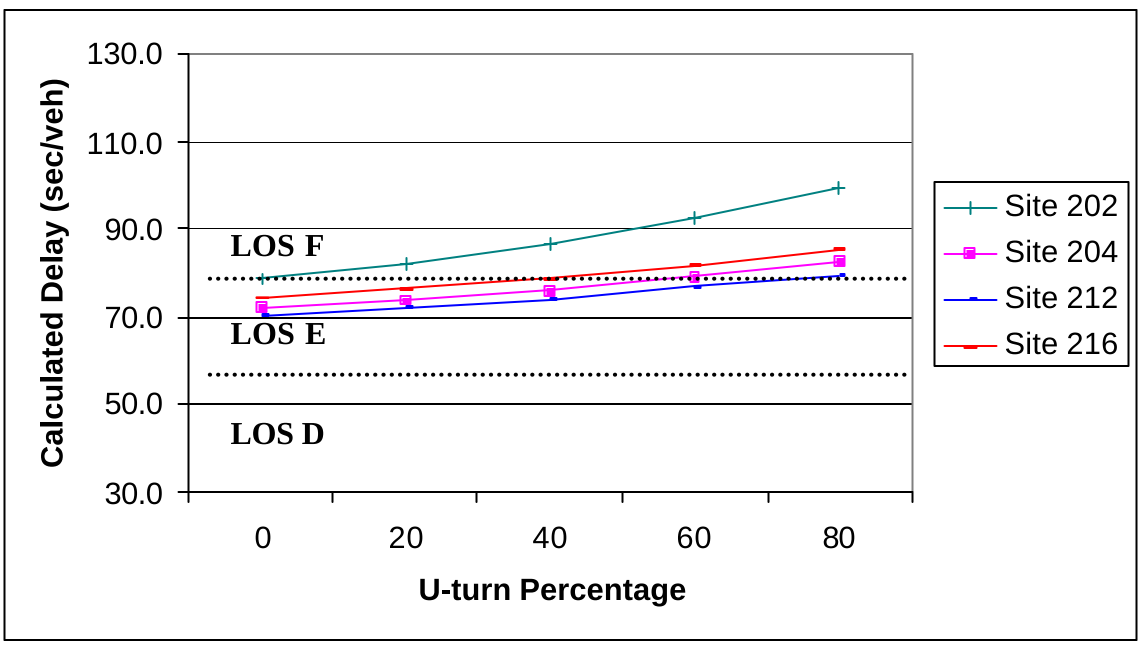


Figure 47. Effect of Increased U-Turn Percentage on Delay at Approaches with No Protected Right Turn Overlap and Single Left Turn Lane

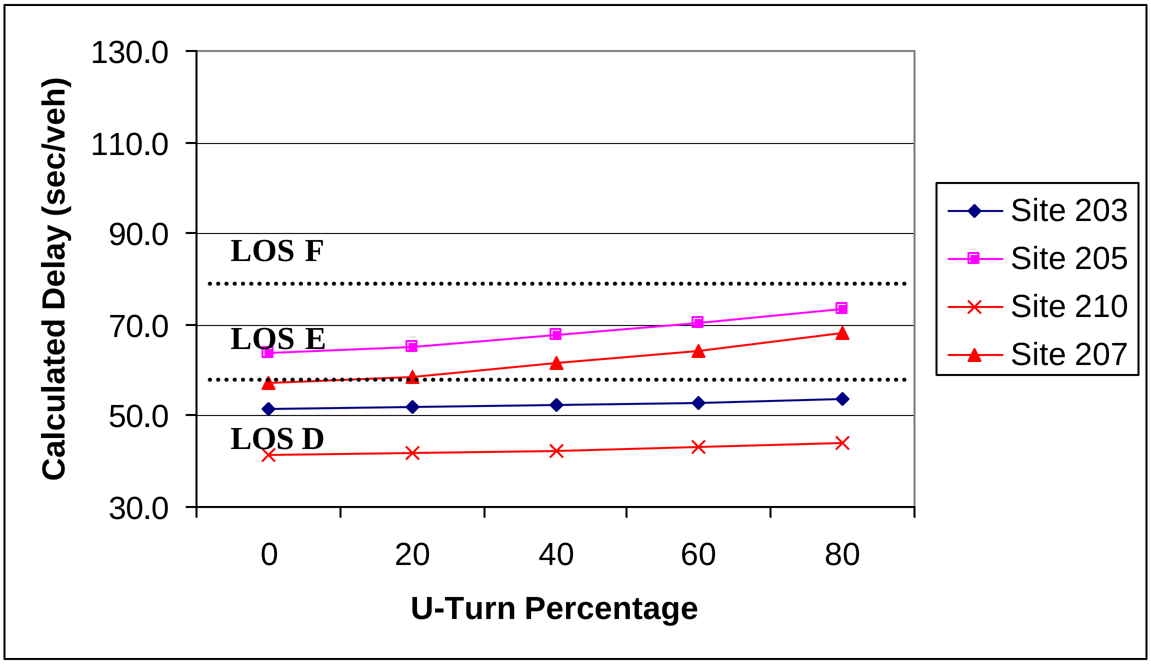


Figure 48. Effect of Increased U-Turn Percentage on Delay at Approaches with Protected Right Turn Overlap and Double Left Turn Lanes

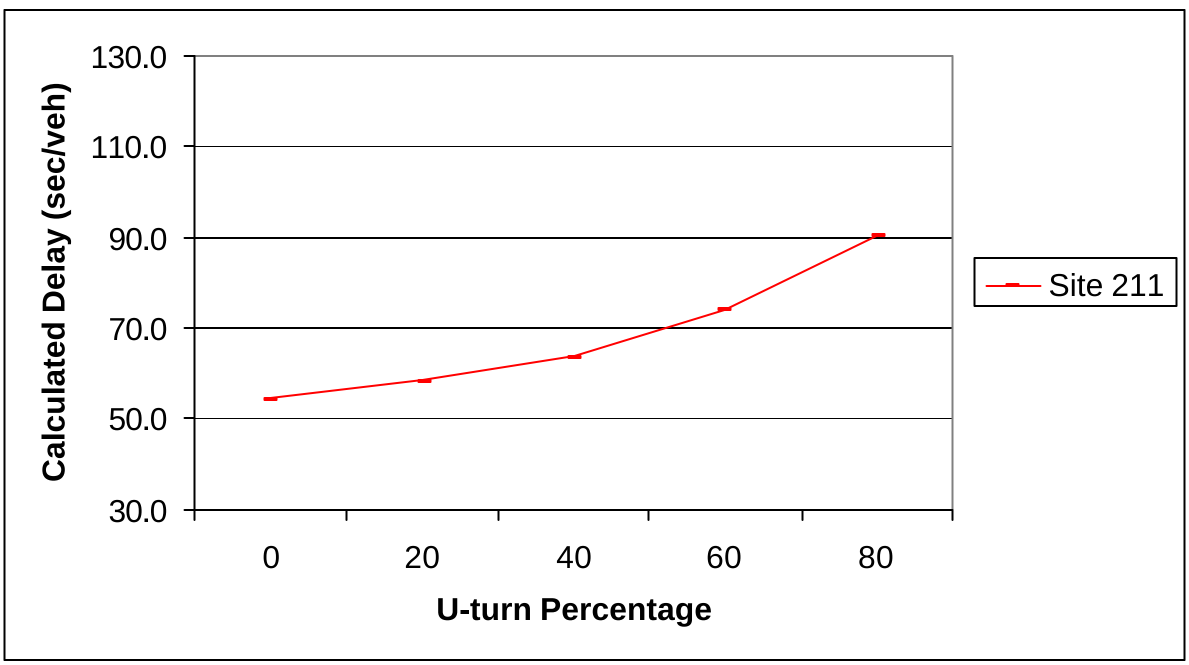


Figure 49. Effect of Increased U-Turn Percentage on Delay at an Approach with Protected Right Turn Overlap and Single Left Turn Lane

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.

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 50 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 50 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.

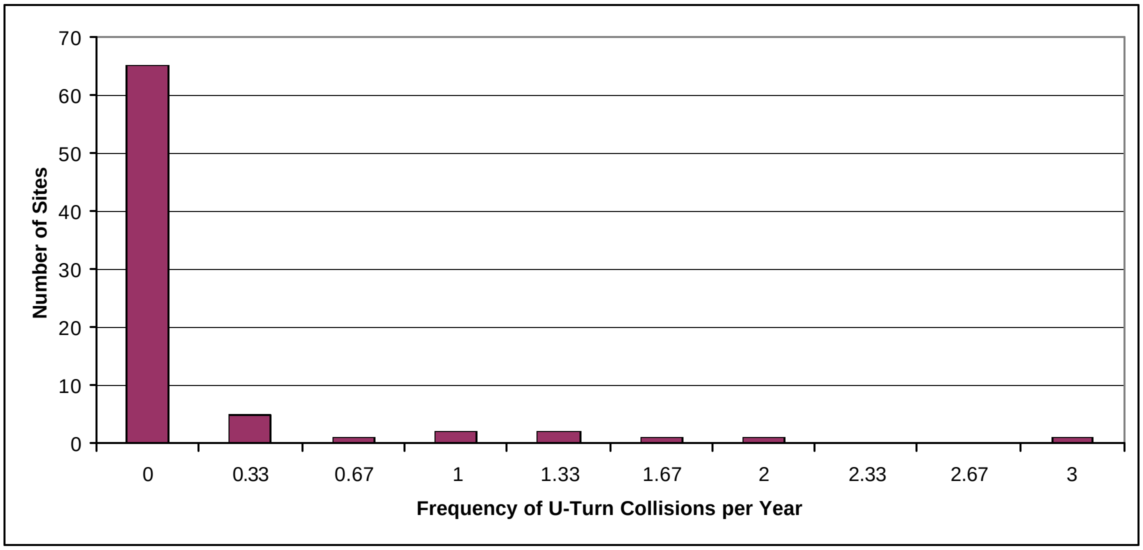


Figure 50. Histogram of U-Turn Collision Frequency

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

Table 18. 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.

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. We decided instead to focus on the site characteristics that correlate significantly with U-turn collisions.

The team examined factors pertaining to geometry of the intersection, signal type, and traffic volume. Table 19 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, the team used a t-test to compare the mean value of the two groups. To verify the t-test results, the team 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), the team 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, we used the Fisher's Exact test, which gives a more accurate analysis for low expected values.

Table 19. 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 AADT	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 44) 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 (AADT).* For this characteristic, the main road is defined as the road whose left turn lane is being studied. The main road AADT 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 AADT to be a significant factor.

7-10. *AM and PM peak turning movements.* Because AADT was too broad a measure, we 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.

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 20 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 [32]. 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 20. Summary of U-Turn Conflict Data

Study No.	U-Turn Conflict Type: Left Turn Same Direction	Right Turn and U-turn	Side-swipe	Ped and U-turn	Total Conflicts Observed	U-Turn Conflicts per Hour
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 21 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 21. 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 22 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 22. 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.

FINDINGS AND CONCLUSIONS

SEGMENT CONCLUSIONS

The purpose of this portion of the research was to develop empirical models to predict collisions on four-lane median-divided segments and five-lane with two-way left turn lane (TWLTL) segments in North Carolina. Geometric, volume, collision, and land use data were collected on 143 segments totaling approximately 87 miles.

The form of the models was adopted from a previous study conducted by Bonneson and McCoy in Nebraska and Arizona [1]. Their models were judged as logical and were created using the current state of the art in terms of collision modeling. Negative binomial regression in SAS® was used to recalibrate the Bonneson and McCoy models. Traffic volume, segment length, predominant land use, and approach density were found to be significantly related to collisions.

For predominantly residential or industrial land uses, the raised median design was always found to be associated with fewer collisions than is the TWLTL. The raised median design also has a safety advantage over the TWLTL for predominantly business or office land uses with low to medium approach densities (0-25 approaches per mile). For business and office land uses with medium to high approach densities (25-90 approaches per mile), the TWLTL appears to be slightly safer at low traffic volumes and the raised median appears to be slightly safer at high traffic volumes.

If decisions need to be made for segments with AADT values greater than 50,000 vpd or less than 20,000 vpd, or for approach densities greater than 90 approaches per mile, additional data will need to be collected to validate the models presented here.

For land uses that are predominantly residential or industrial, approach densities do not need to be collected in order to apply the models. For these cases, only segment length and average daily traffic are needed to use either of the models. For land uses that are predominantly business or office, approach densities do need to be collected or estimated in addition to segment length and average daily traffic.

The primary application of these models should be for NCDOT to determine which cross-section is safest in the planning stages of a road widening project. The models could also be used to determine how much a median or TWLTL retrofit would improve safety on a segment. Additionally, the NCDOT can use these models to support their design decisions for the benefit of local citizens and business owners at public hearing.

As driver behavior changes, the predictive quality of these models may deteriorate with time. As a result, the collision models should be recalibrated periodically to ensure that they are continuing to adequately predict collisions. It would be wise to recalibrate the models approximately every ten years.

While the form of these collision models has now been shown to work reasonably well in at least three states, the coefficient estimates appear to be unique to each region. Consequently, other state departments of transportation should not use these models directly but rather recalibrate them with data from their own state. Similarly, these models should not be used in a national Highway Safety Manual. However, if the models could be recalibrated with data from a nationwide database, they could be very useful for those states without their own models.

INTERSECTION CONCLUSIONS

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.

Qualitative Observations

Throughout the course of data collection for this project, we 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.

The team observed that large intersections provide room for left turners to circumvent a paused U-turn (see Figure 45). 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 wait).

Heavy vehicles would be expected to have more difficulty with U-turns, but the team observed that their more experienced driver's 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, we did not observe any conflicts. This 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.

IMPLEMENTATION AND TECHNOLOGY TRANSFER PLAN

The primary means for sharing project results will be the four-page brochure, similar in format to the FHWA *TechBrief*, developed in conjunction with this report. However, the content of the brochure reflects more of a user's guide than just an explanation of the research results. One part of the brochure discusses the segment-based collision model, the equation and variables, and limitations of the model. A second part includes results from studying U-turns at signalized intersections, including the recommended saturation flow adjustment factor for exclusive left turn lanes when U-turns are present. There is some discussion of observations of driver behavior for U-turning vehicles, as well as limitations of the research.

Planners, designers, local officials, business owners, and others can use the brochure to compare divided versus TWLTL cross-sections for safety considerations. This information can then be used in coming up with the safest, most efficient, and least costly choice for a particular situation.

The brochure can stand alone, be inserted into NCDOT planning and design manuals, or be incorporated into training materials and workshops. The brochure can be handed out at public hearings, or whenever and wherever the issue of using a divided highway versus a TWLTL is being discussed.

The NCDOT can use the brochure (and report) throughout the planning and design of an arterial widening or rehabilitation project. The collision model can be run very early in the planning stages with preliminary information and can be rerun periodically throughout preconstruction as model inputs become clearer. The U-turn results and information from the literature summary can be used in much the same way. Anyone with questions about the selection of a cross-section on a particular project can look at the brochure, examine the input data assembled by the planners and designers, and hopefully reach the same conclusion as they did.

The brochure has been prepared in a format suitable for NCDOT to incorporate into both a paper version and an electronic version for dissemination to appropriate NCDOT staff, practitioners, local officials, and others, as well as other state DOTs. It is not intended at this time for the brochure to be in a camera-ready format for publication. The intent is for NCDOT to have the flexibility to put the content into a format of their choosing.

This research report provides details should anyone need more back-up than the brochure provides. The report, of course, includes a literature summary, the full data sets, and details of the statistical analyses.

The research team is already on the agenda for presentation of this project at the upcoming Traffic Engineering Conference in early September in Asheville, N.C. The team members are available to present the findings at similar planning or design conferences for NCDOT users and other practitioners.

While not a direct implementation for NCDOT, the research team has submitted abstracts of this research for possible presentation at TRB's upcoming International Geometric Design Symposium in Chicago in 2005. This research is significant from a national perspective in its depth of study sections and hours of observations of this controversial topic. The research team will also submit abstracts and papers for other conferences and journals.

RECOMMENDATIONS FOR FUTURE RESEARCH

Future segment research should include collecting the number of median openings per segment or per mile. This parameter is important because the greater the number of median openings per mile, the less concentrated are the left turns from the main arterial. Additionally, for economic studies, the more median openings allowed, the less the economic impact of the median. The type of median opening, such as full access, directional, etc., should also be collected since this factor indicates the number of conflict points present at that location.

More data should also be collected for the regions of the segment database that were lacking. For example, segments with approach densities greater than 100 approaches per mile should be collected to determine which left turn treatment is best in this region. In particular, raised median segments with high approach densities should be investigated. One major area of the database that is lacking is TWLTL segments with AADTs greater than 35,000 vpd. Additionally, there is little data for industrial and office land uses which have been combined with the residential and business land uses, respectively. If this model is to be applied to these land uses, more data should be collected in these areas in order to validate the model. None of the segments randomly selected for this study had on-street parking. Previous research has indicated that on-street parking has a large impact on the safety of a segment [1]. More research needs to be done to determine how great of an impact parking can have on safety and in what way it could be incorporated into the collision models.

For further research on the operational impacts of U-turns on left turn lanes, we 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. The team used a front-axle reference point in the data collection for this research. The team 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 geometric improvements.

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Appendices

Appendix A. Data Collection Form for Safety Study Intersections

County: ______________________ **Municipality:** ______________________

Main Street: ______________________

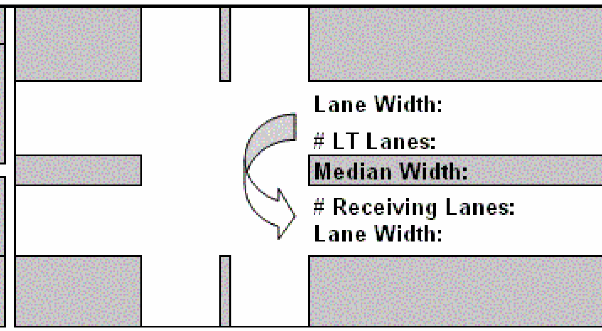
Cross Street: ______________________

Segment Beginning Point: ______________

Direction to Intersection: ______________

LT Signal Type	
Permitted	○
Protected	○
Both	○

Conflicting RT	
Permitted	○
Protected	○
Prohibited	○



Note any unusual grades or angles on the drawing

Begin Mileage:	
End Mileage:	
Length:	
Speed Limit:	

Public Street Approaches:

Driveways:

Land Use %	
R	
O	
B	
I	

Useable?	
Y	○
N	○

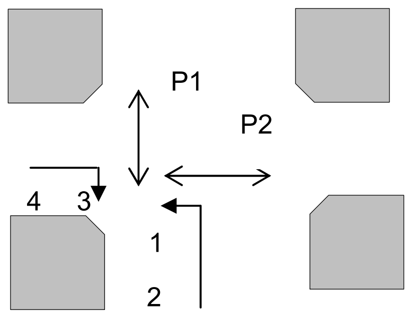
Notes:

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	

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

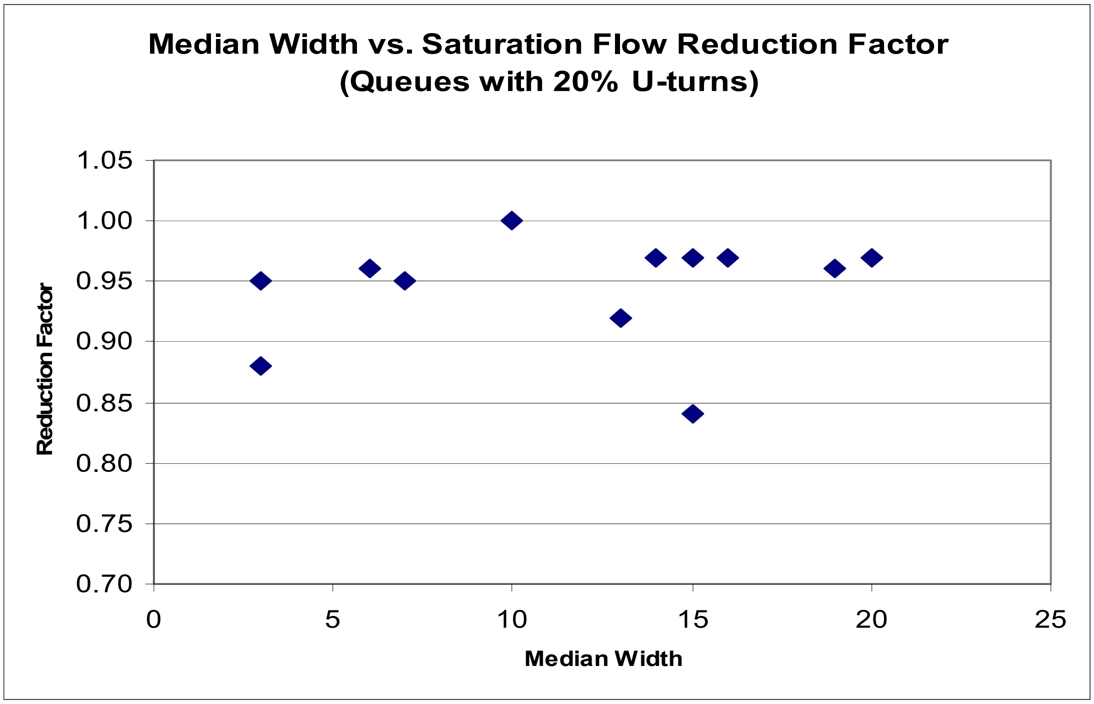
RTOA/RTOR	Reduction Factor for Queues of 20% U-turns
4	0.97
41	0.97
1	1.00
19	0.84
53	0.95
149	0.92
7	0.95
5	0.96
33	0.96
57	0.97
17	0.97
20	0.88

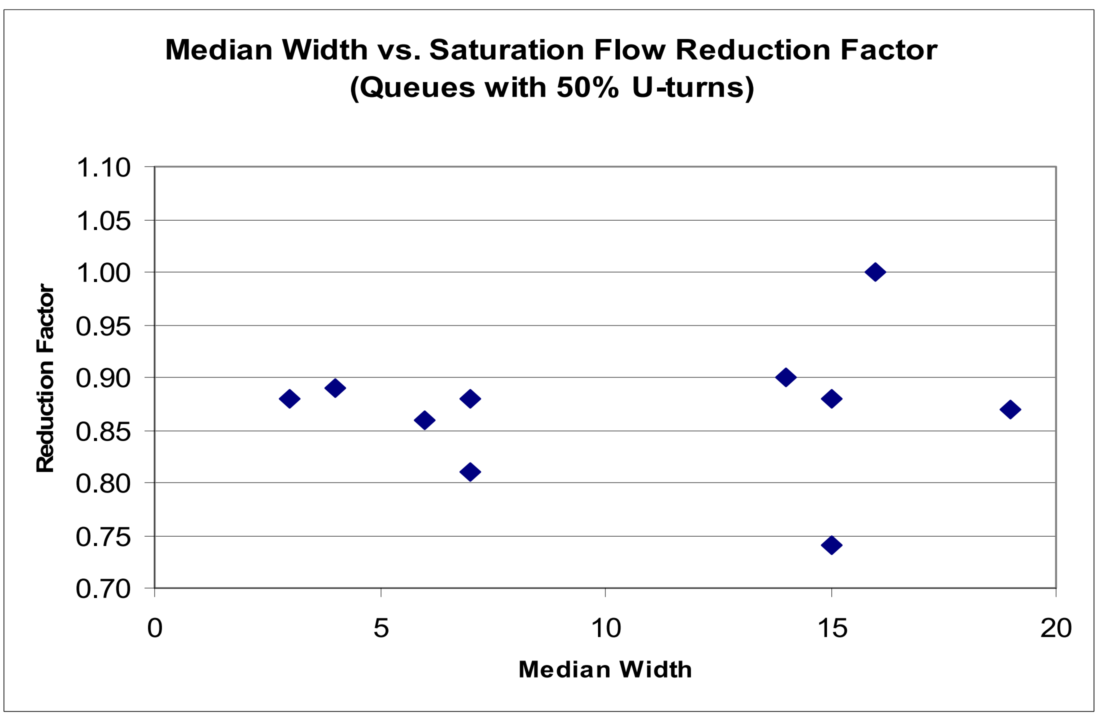
Regression slope p-value = 0.63

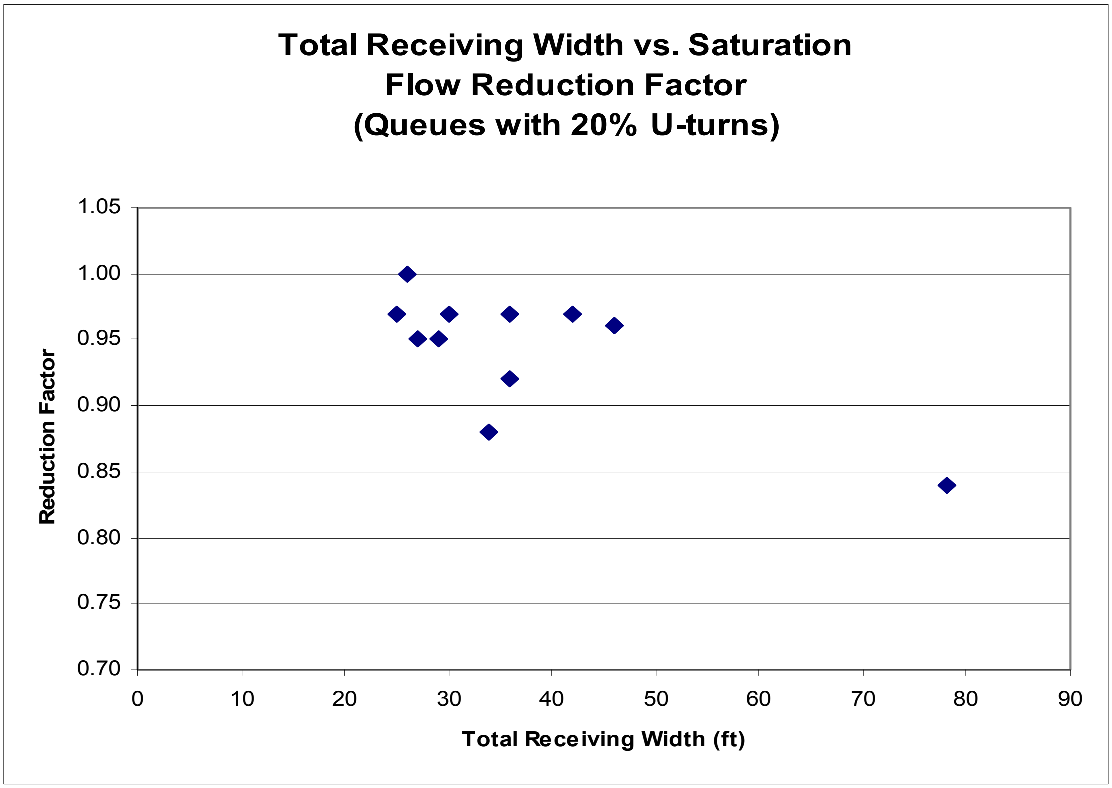
RTOA/RTOR	Reduction Factor for Queues of 50% U-turns
4	1.00
19	0.74
0	0.89
53	0.81
7	0.88
5	0.87
33	0.86
57	0.90
17	0.88
2	0.88

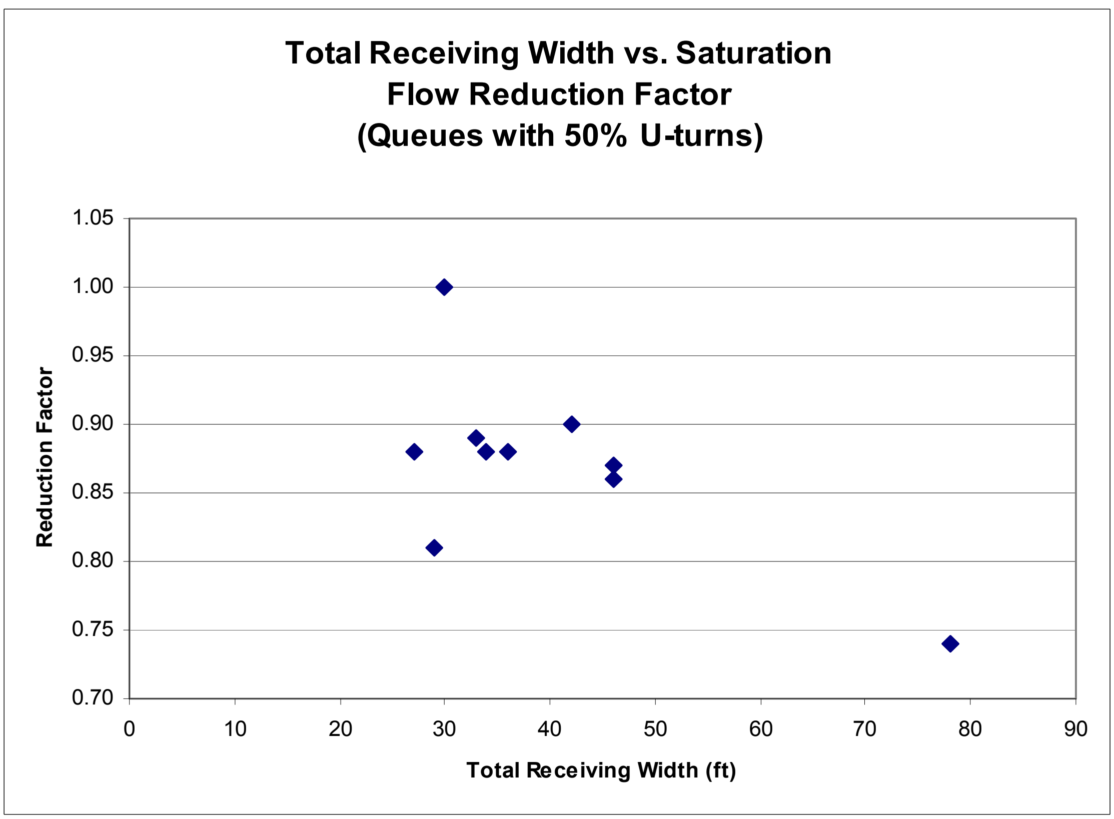
Regression slope p-value = 0.41

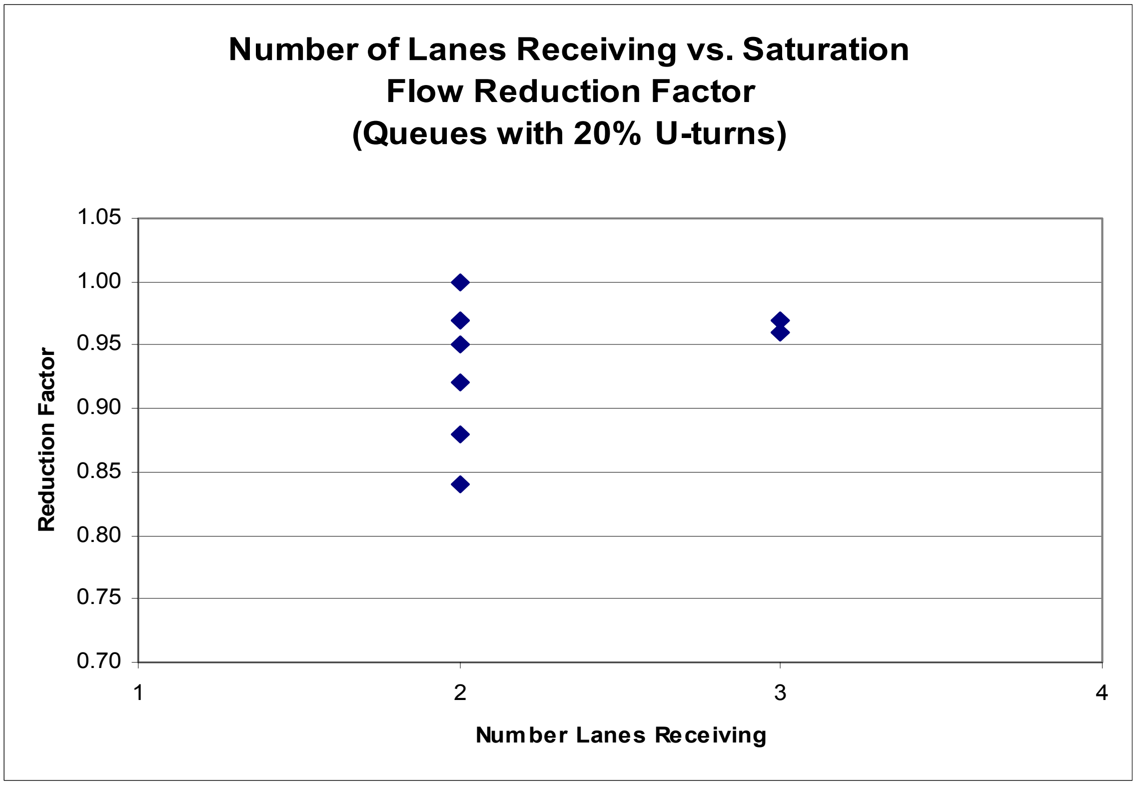
Appendix H. Graphs of Reduction Factor versus Various Site Characteristics

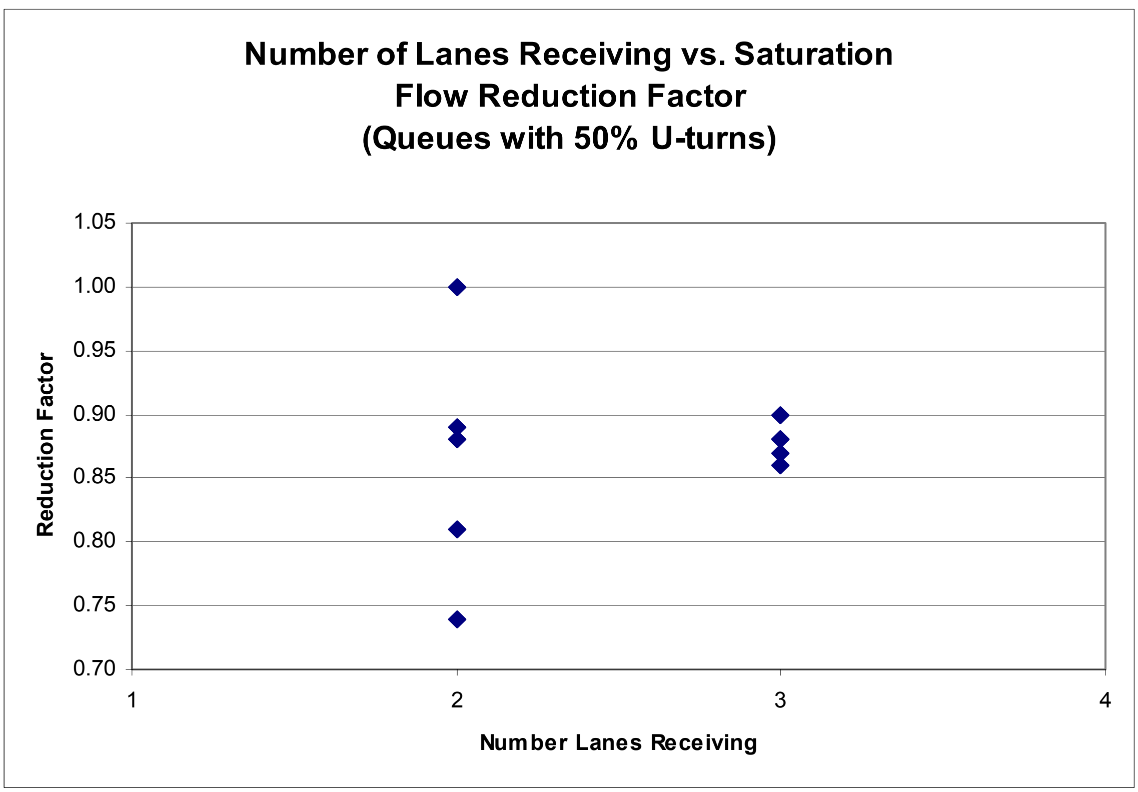


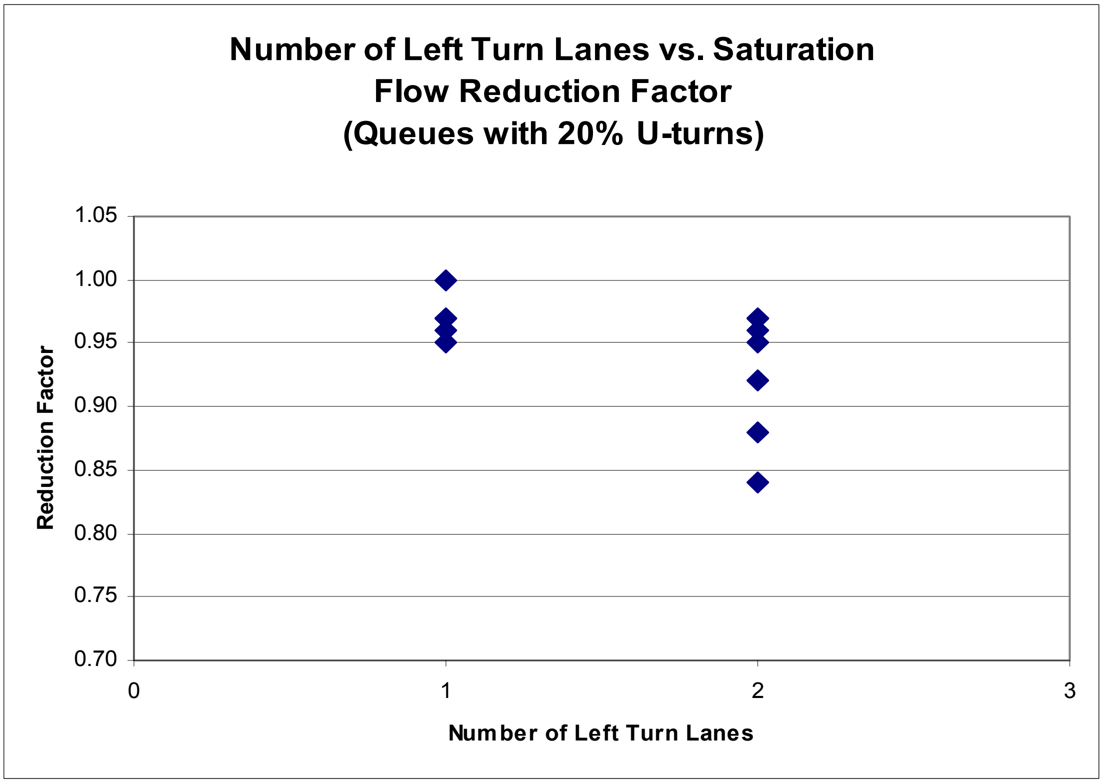


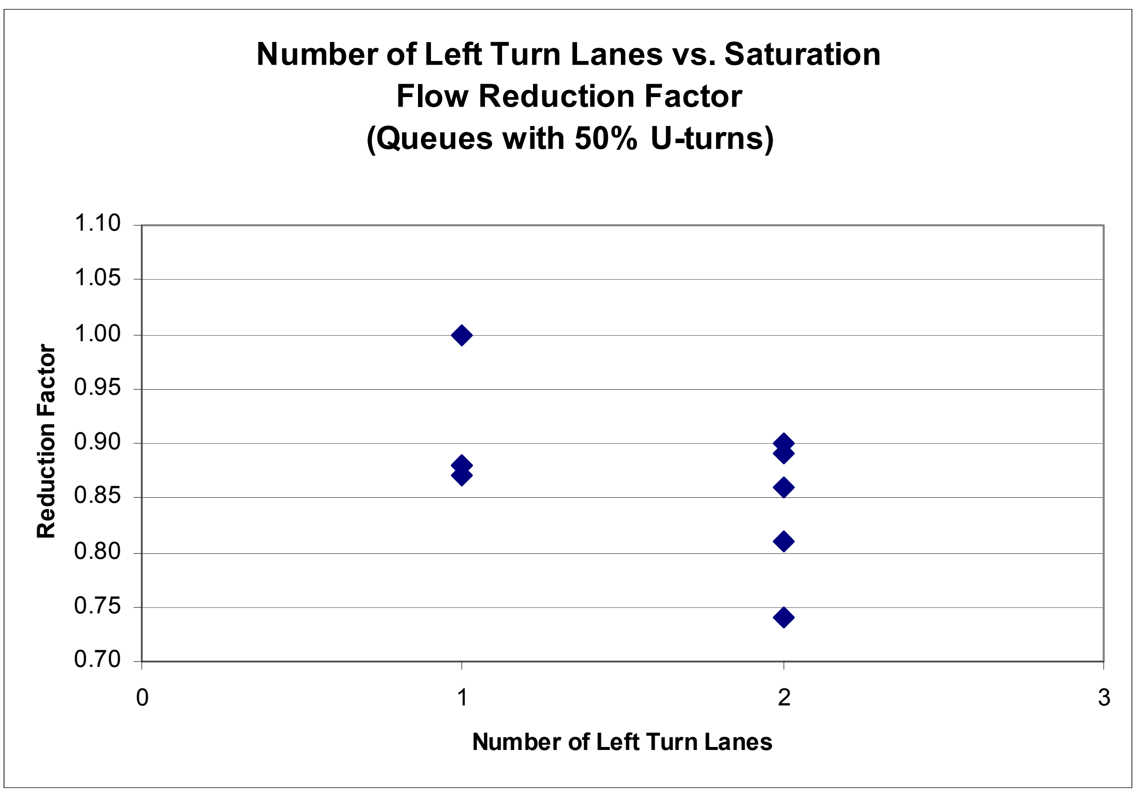

Total Receiving Width vs. Saturation
Flow Reduction Factor
(Queues with 20% U-turns)
Reduction Factor
Total Receiving Width (ft)
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
10
20
30
40
50
60
70
80
90

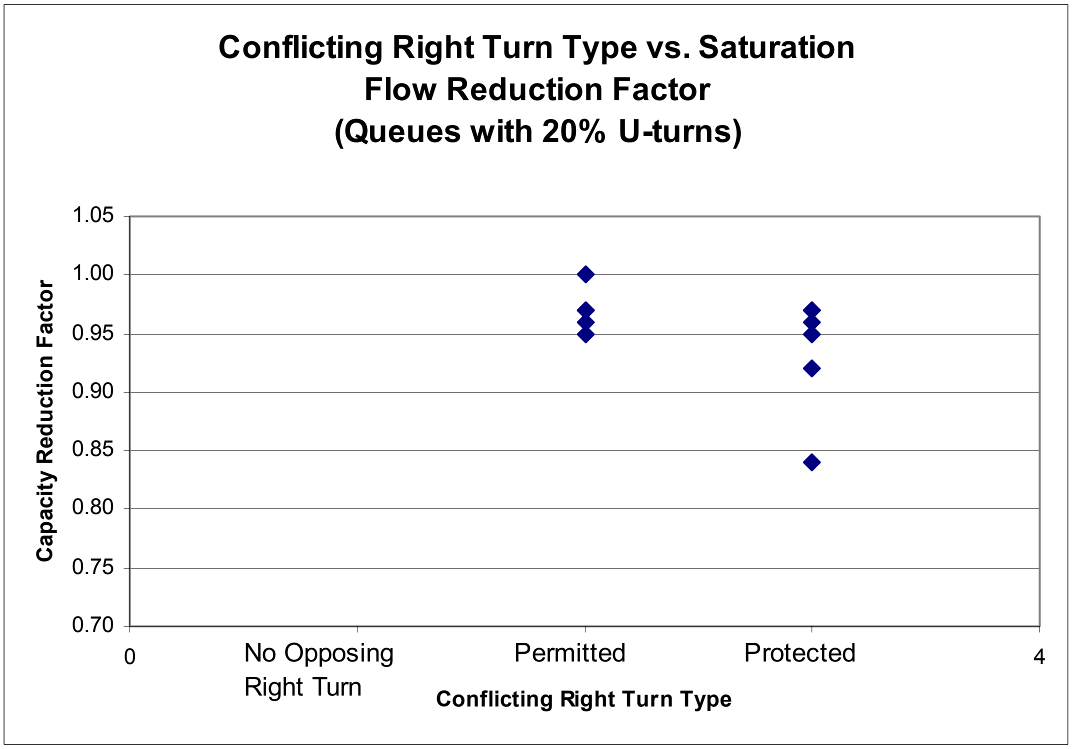

Total Receiving Width vs. Saturation
Flow Reduction Factor
(Queues with 50% U-turns)
Reduction Factor
Total Receiving Width (ft)
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
10
20
30
40
50
60
70
80
90

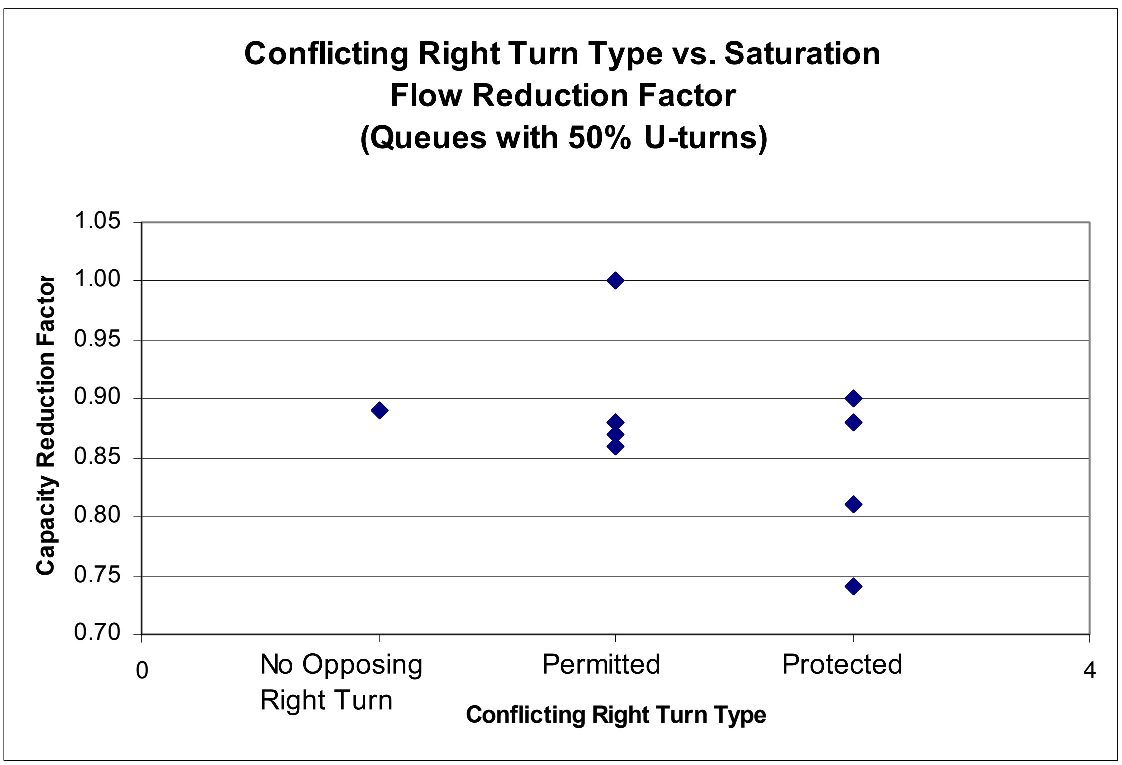

Number of Lanes Receiving vs. Saturation
Flow Reduction Factor
(Queues with 20% U-turns)
Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
1
2
3
4
Number Lanes Receiving

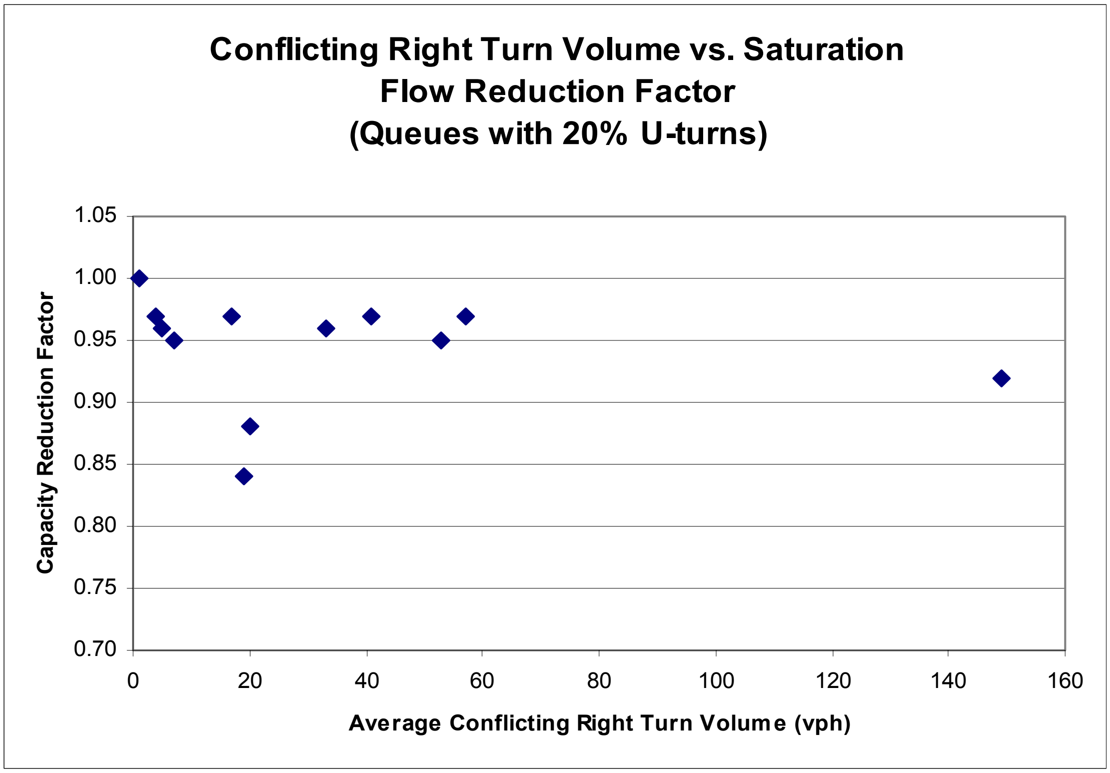

Number of Lanes Receiving vs. Saturation
Flow Reduction Factor
(Queues with 50% U-turns)
Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
1
2
3
4
Number Lanes Receiving

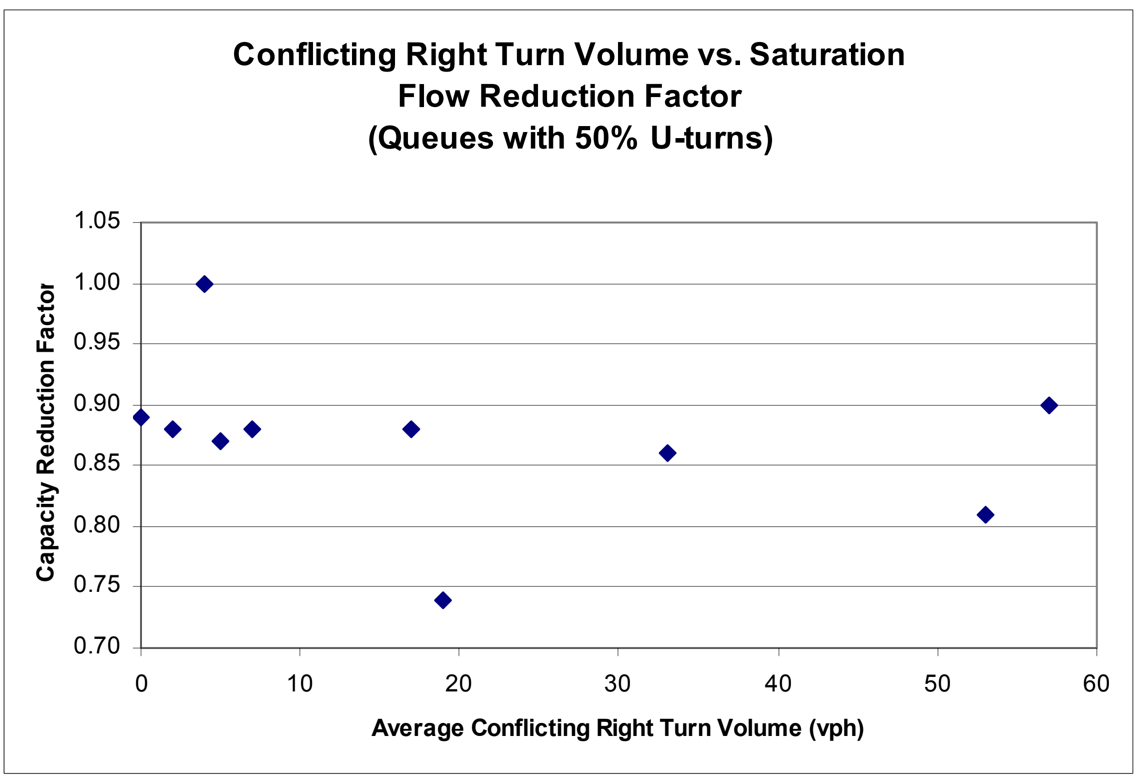

Number of Left Turn Lanes vs. Saturation Flow Reduction Factor (Queues with 20% U-turns)
Reduction Factor
1.05
1.00
0.95
0.90
0.85
0.80
0.75
0.70
0
1
2
3
Number of Left Turn Lanes


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		Intersection Information		
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

	Site and Approach Location					Collisions				Intersection Information: 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

                           Sum of      Expected       Std Dev          Mean
      group        N       Scores      Under H0      Under H0         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 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.

APPENDIX N. SAS® OUTPUT FOR COLLISION MODELS

Raised Median Residuals 1

The GENMOD Procedure

Model Information

Data Set	WORK.RESULTS
Distribution	Poisson
Link Function	Log
Dependent Variable	Crashes
Observations Used	50

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	44	111.4858	2.5338
Scaled Deviance	44	44.9495	1.0216
Pearson Chi-Square	44	109.1308	2.4802
Scaled Pearson X2	44	44.0000	1.0000
Log Likelihood		356.8048	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square
Intercept	1	-16.6814	2.6147	-21.8061	-11.5566	40.70
LADT	1	1.3270	0.2788	0.7806	1.8733	22.66
LLength	1	0.7233	0.1813	0.3680	1.0786	15.92
BO	1	-0.8463	0.3011	-1.4365	-0.2561	7.90
RI	1	-0.6968	0.2873	-1.2598	-0.1337	5.88
BO*AD	1	0.0132	0.0050	0.0035	0.0229	7.07
Scale	0	1.5749	0.0000	1.5749	1.5749	

Analysis Of Parameter Estimates

Parameter	Pr > ChiSq
Intercept	<.0001
LADT	<.0001
LLength	<.0001
BO	0.0050
RI	0.0153
BO*AD	0.0078
Scale	

The GENMOD Procedure

Model Information

Data Set	WORK.RESULTS
Distribution	Poisson
Link Function	Log
Dependent Variable	Crashes
Observations Used	65

Criteria For Assessing Goodness Of Fit

Criterion	DF	Value	Value/DF
Deviance	61	318.3084	5.2182
Scaled Deviance	61	60.8126	0.9969
Pearson Chi-Square	61	319.2893	5.2343
Scaled Pearson X2	61	61.0000	1.0000
Log Likelihood		204.8032	

Algorithm converged.

Analysis Of Parameter Estimates

Parameter	DF	Estimate	Standard Error	Wald 95% Confidence Limits		Chi-Square
Intercept	1	-21.2535	4.7668	-30.5962	-11.9108	19.88
LADT	1	1.5829	0.4162	0.7673	2.3986	14.47
LLength	1	0.8902	0.2025	0.4934	1.2870	19.33
AD*BO	1	0.0080	0.0025	0.0031	0.0130	10.02
Scale	0	2.2878	0.0000	2.2878	2.2878	

Analysis Of Parameter
Estimates