False Capacity for Lane Drops

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16. Abstract

Lane drops downstream of signalized intersections are found on many urban and suburban streets and highways. Since drivers tend to avoid using the short lane due to the potential for stressful merges downstream of the signal, the short lane is typically under-utilized. Previous research indicates that the default lane utilization factors in the Highway Capacity Manual (HCM) appear to overestimate traffic in the short lane. The purpose of this project was to develop models to predict lane utilization factors for six defined intersection types and to assess how a lane drop affects safety near intersections.

Traffic, signal, and collision data were collected at 94 sites in North Carolina. Based on 15 candidate factors, multiple regression models were developed for the purpose of predicting the lane utilization factor. This study found that the downstream lane length and traffic intensity positively correlate with the lane utilization factor, and that some geometric variables at the approach may also influence lane utilization. Collision data analysis results show that collision rates downstream of an intersection decline as the distance to a lane drop increase. The results also show that the lane drop type does not affect collision rates upstream and downstream of intersections.

Many of the results derived from this study are consistent with previous research. The models developed should provide designers and traffic engineers with concrete methods to improve lane utilization when lane drops are contemplated.

A re-assessment of the effect of lane utilization on capacity is recommended, since models in the HCM consistently overestimate delay for all types of lane drop intersections with low lane utilization.

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SUMMARY

Lane drops downstream of signalized intersections are found on many urban and suburban streets and highways. Since drivers tend to avoid using the short lane due to the potential for stressful merges downstream of the signal, the short lane is typically under-utilized. Previous research indicates that the default lane utilization factors in the Highway Capacity Manual (HCM) appear to overestimate traffic in the short lane. The purpose of this project was to develop models to predict lane utilization factors for six defined intersection types and to assess how a lane drop affects safety near intersections.

Traffic, signal, and collision data were collected at 94 sites in North Carolina. Based on 15 candidate factors, multiple regression models were developed for the purpose of predicting the lane utilization factor. This study found that the downstream lane length and traffic intensity positively correlate with the lane utilization factor, and that some geometric variables at the approach may also influence lane utilization. Collision data analysis results show that collision rates downstream of intersections with the shortest distances to lane drops in each intersection category are about 14 times higher than the average lane drop site and the rates decline as the distance to a lane drop increase. The results also show that the lane drop type does not affect collision rates up to half mile upstream and downstream of intersections.

Many of the results derived from this study are consistent with previous research. The models developed should provide designers and traffic engineers with concrete methods to improve lane utilization when lane drops are contemplated. For example, designers could use the models provided in this report to estimate the performance of 500-foot and 1,000-foot lane drops and see whether the improved performance of the latter justifies the higher cost.

A re-assessment of the effect of lane utilization on capacity is recommended, since models in the HCM consistently overestimate delay for all types of lane drop intersections with low lane utilization.

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CHAPTER 1. INTRODUCTION

1.1. Background

Lane underutilization at a signalized intersection by through traffic can be attributed to many causes, including forced merges downstream of the signal due to short tapers/lanes, preponderance of turns in the lane, driveways, concentration of heavy trucks, vehicle queues, the presence of near side or far side bus stops, and the presence of on-street parking. Lane underutilization likely means a loss of capacity and extra delay, and the consequences of this could be large at some locations. The signal may be mistimed, resulting in extra delay for all users of the intersection, and needed capacity improvements may not be programmed. In addition, developments may be approved in the area that would not be if the roadway capacity were calculated correctly. Motorists may therefore experience considerable extra delay per year. Despite the potential large consequences, the *Highway Capacity Manual (HCM)* (1) does not provide any quantitative procedure considering downstream lane drops in assessing the capacity of signalized intersection. Instead, the HCM simply notes that lane utilization should be adjusted to account for the variation of traffic flow on the individual lanes in a lane group due to upstream or downstream roadway characteristics such as changes in the number of lanes available.

Previous research indicates that these lane drop effects are indeed significant, but also that they varied considerably among sites. Moreover, the evidence indicates that current *HCM* appears to overstate the value of the lane utilization factor (LUF). One previous study observed LUFs in the range of 0.73 to 0.82, compared to the 0.90 to 0.95 estimates provided in the HCM (1). This means that the saturation flow of the lane group will be over-estimated by around 15 percent, and that the capacity of the lane group will be over-estimated by about 5 to 10 percent. Thus, it appears that many intersections having short lanes in North Carolina may be under-designed, given the apparent over-estimated capacity that the HCM produces.

Designers also need better information on the safety consequences of lane drops and related features. Information available from the AASHTO "Green Book" (2) and other standard sources is confined to suggested taper rates and signs. There is no information or guidelines for the design of lane drop. For instance, designers could use guidance on the length of the additional lane to provide and whether to terminate the lane at an intersection as a turn-only lane or at a taper. Having these kinds of guidelines for the designers of an intersection could prevent many collisions with reasonable improvement costs.

1.2. Research Objective

There were two objectives for this research:

1. To develop a set of field-verified estimates for the lane utilization factor at a signalized intersection when there is a downstream lane drop.
2. To investigate the pattern of collision rates for lane drop areas around signalized intersections, including lane drops at the intersection.

Achieving these objectives might help designers, researchers, and practitioners regarding lane drop intersection design, operation and maintenance. For instance, the models can be used to determine the required downstream short lane length, given prevailing traffic volumes and a desired lane utilization factor.

1.3. Scope of Research

This research scope was limited to two-lane-to-one and three-lane-to-two lane drops. The research considered dual left turn lanes with downstream lane drops—those that terminate on surface streets and interchange ramps—as well as through lanes. The six categories of lane drop intersection we considered are listed below and shown in Figures 1 to 6.

- 2TS: Two through lanes, with no exclusive right turn lane at the signal approach (Figure 1)
- 2TE: Two through lanes, with an exclusive right turn lane at the signal approach (Figure 2)
- 2LR: Two left turn lanes onto a freeway on-ramp at the signal approach (Figure 3)
- 2LS: Two left turn lanes onto a surface street at the signal approach (Figure 4)
- 3TS: Three through lanes, with no exclusive right turn lane at the signal approach (Figure 5)
- 3TE: Three through lanes, with an exclusive right turn lane at the signal approach (Figure 6)

All data collection activities during the project were carried out in North Carolina.

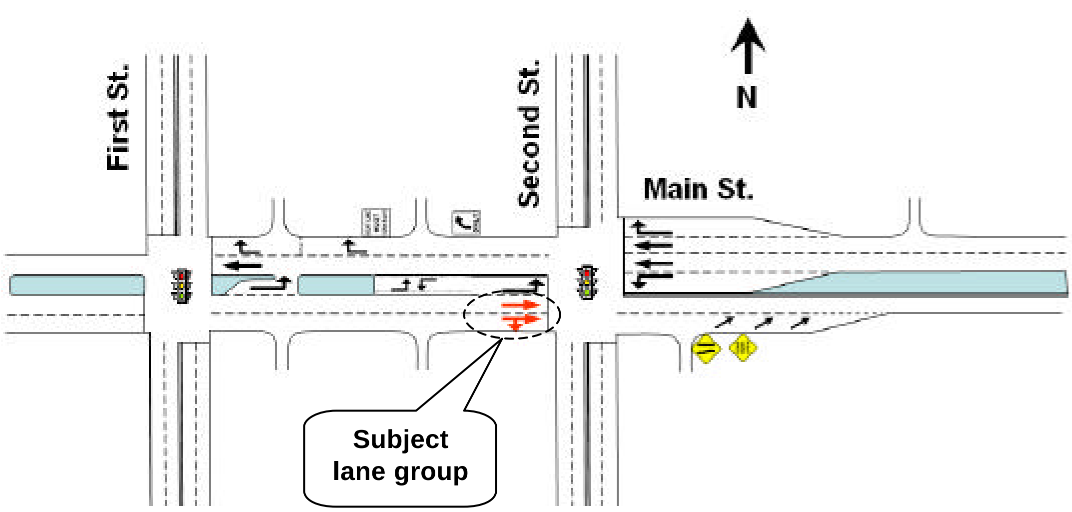


Figure 1. 2TS intersection geometry

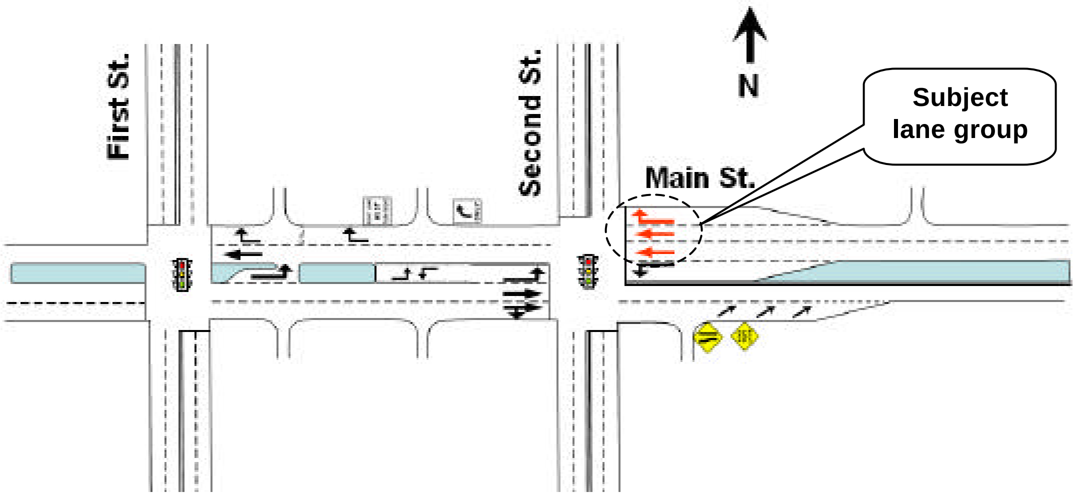


Figure 2. 2TE intersection geometry

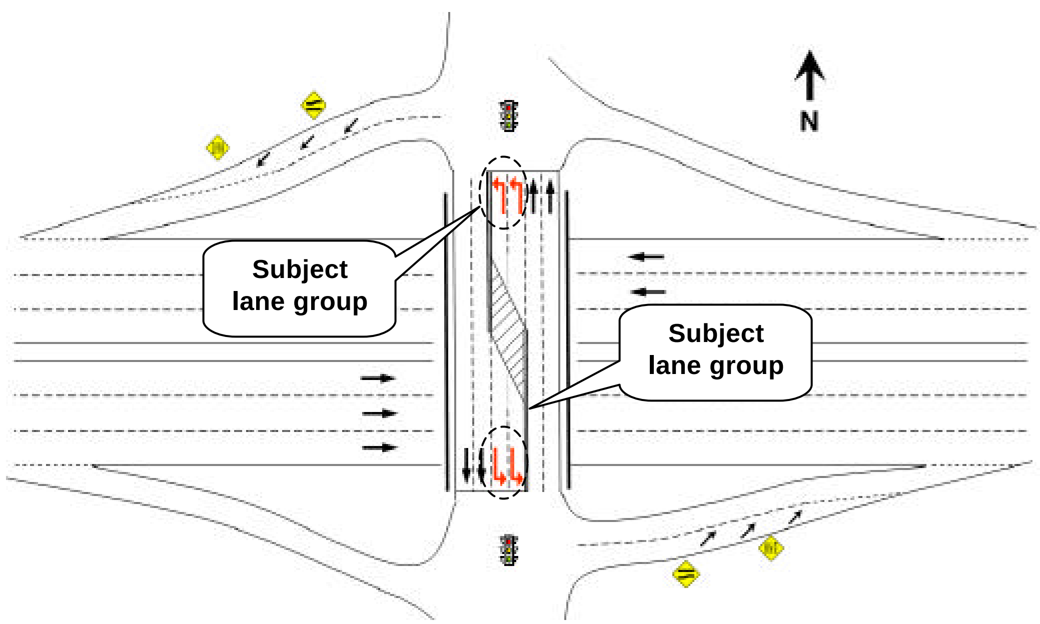


Figure 3. 2LR intersection geometry

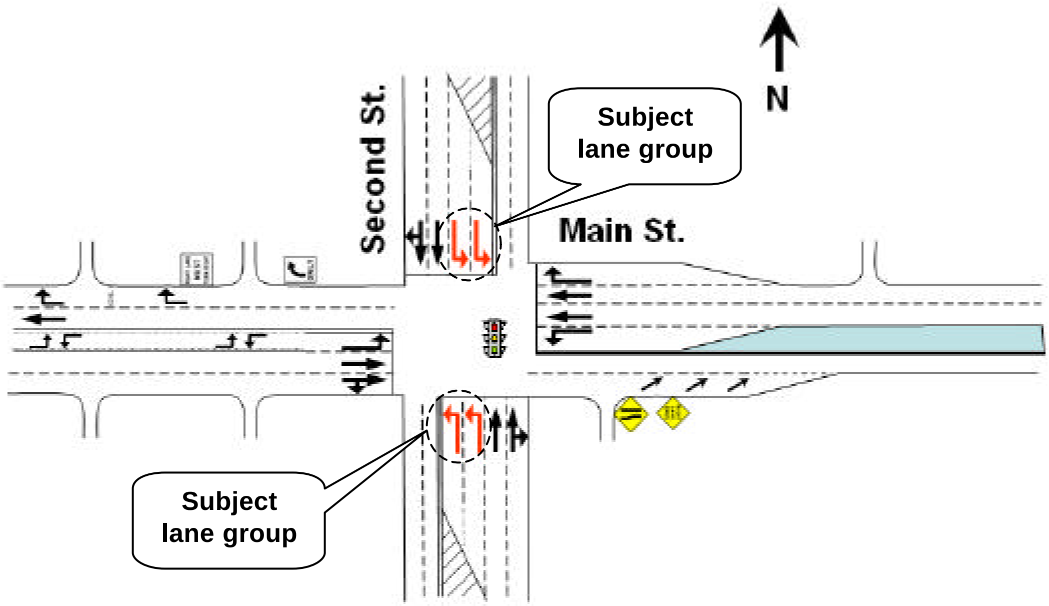


Figure 4. 2LS intersection geometry

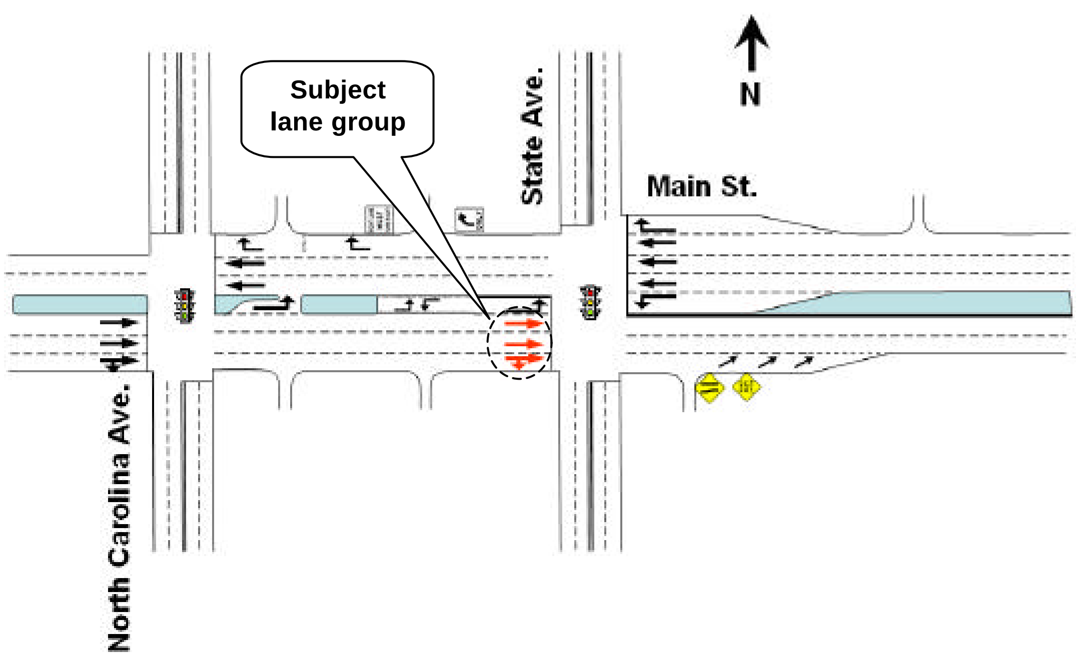


Figure 5. 3TS intersection geometry

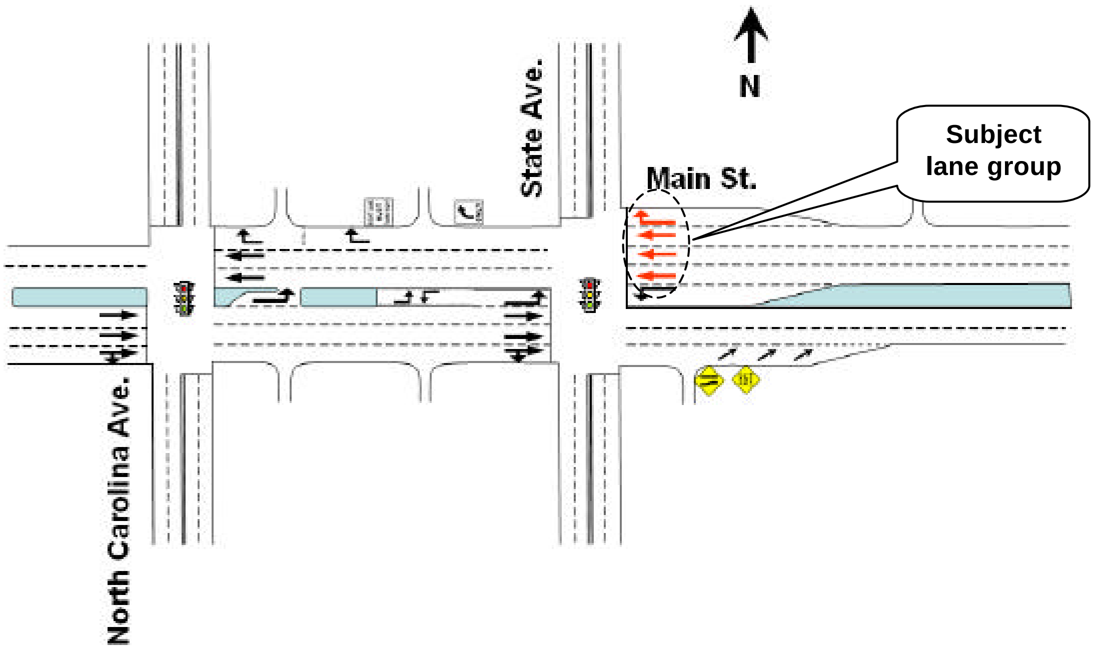


Figure 6. 3TE intersection geometry

For the purpose of data collection, only right lane drops were considered due to the lack of left lane drop sites. However, this limitation did not apply for the intersections at ramp junction areas. It was assumed that the presence of a lane drop influences traffic flow significantly no more than 1/2 mile upstream and downstream of the intersection. So we did not collect data beyond those limits.

This report is organized into the following sections:

- Literature review of previous research on lane utilization at lane drop intersections (chapter 2);
- Description of data collection and reduction procedure (chapter 3);
- Description of the development of lane utilization prediction models (chapter 3);
- Description of the methodology of collision data collection and analysis (chapter 3);
- Operational and collision data analysis results (chapters 4 and 5 respectively); and
- Conclusions and recommendations for future research (chapter 6).

CHAPTER 2. LITERATURE REVIEW

Designers in North Carolina and most states use the *Highway Capacity Manual* (1) to estimate capacity and level of service (LOS) at signalized intersections. The current guidance on lane utilization in the *HCM* is quite limited and often leads to poor estimates of capacity, delay, and level of service where a lane drops shortly after the signal. The HCM defines lane utilization as the ratio of the average lane volume to critical lane volume of the lane group.

$$f_{LU} = \frac{v_g}{(v_{g1}N)}$$

where f_{LU} = HCM lane utilization adjustment factor
v_g = unadjusted demand flow rate for the lane group
v_{g1} = unadjusted demand flow rate on lane with highest volume in lane group
N = number of lanes in lane group

The HCM method does suggest that the short lane, when added on the approaches to an intersection and/or dropped downstream, may not function as a full through lane. It does not, however, provide any guidance of the proper lane utilization factor for such lanes. Other traffic models used by designers and analysts contain lane utilization estimates that may or may not be more accurate than the *HCM* for lane drops.

Capacity guides in Australia, Canada and Sweden estimate and analyze signalized intersection approaches at the individual lane level. Due to the estimation method of lane flow, the lane utilization estimation is somewhat different than the HCM. The Canadian Capacity Guide (CCG) for signalized intersections estimates lane flow based on equal flow ratio (that is, the ratio of demand flow to saturation flow) (3). However, the CCG does not take into account the case of downstream lane drops. In the Australian and

Swedish guides, the lane flow is calculated based on the proportion of the lane saturation flow to approach saturation flow, and the procedure involves iterations (4,5). The Swedish capacity guide addresses the effects of short lanes on intersection capacity (6). With an existence of a short turning movement lane at the intersection, the access to the other lanes can be blocked when either the back of a queue from the short lane or adjacent full lane spills back beyond the length of the lane. The Swedish capacity guide considered this upstream short lane effect on capacity without addressing the downstream short lane impact.

The aaSIDRA model, which represents the methods in the Australian capacity guide, takes into account the downstream lane drop effect using the downstream short lane length as a factor (4). An equation is provided that describes the relationship between lane utilization of the auxiliary lane and the downstream auxiliary lane length, which includes the width of the cross street. The definition of lane utilization is somewhat different than the HCM lane utilization, since aaSIDRA analyzes signalized intersections at the individual lane level. The aaSIDRA lane utilization is defined as the ratio of the degree of saturation of a lane in a lane group to that for the lane with the highest degree of saturation in the lane group.

$$r = \frac{x_j}{x_c}$$

where r = lane utilization ratio
x_j = degree of saturation of a given lane j
x_c = degree of saturation of a critical lane (having highest x)

The lane utilization ratio of the auxiliary lane in aaSIDRA is calculated depending on the auxiliary lane length and other parameters such as minimum downstream short lane length, downstream short lane length for full lane utilization, and a model calibration parameter. The following equation shows the relationship.

$$R_{LU} = R_{LU_m} + \left(\frac{D_s - D_{sm}}{D_{sf} - D_{sm}} \right)^n \left(100 - R_{LU_m}\right)$$

where R_{LU} = $100 \times ? = 100 \times (x_j / x_c)$
R_{LUm} = Minimum downstream lane utilization ratio (default = 20 %)
D_s = Actual downstream short lane length
D_{sm} = Minimum downstream short lane length (default = 100 ft)
D_{sf} = downstream short lane length for full lane utilization (default = 660 ft), and
n = A model calibration parameter (default = 1.2)

Rouphail and Nevers introduced a new way to estimate saturation flow estimation using traffic subgroups (7,8). The method decomposes the traffic stream into individual components. Due to the flexibility of the model, saturation flows can be estimated by turning movement by lane or by vehicle type.

Hummer investigated the traffic operations at lane drops using the CORSIM simulation model (9,10). However, CORSIM lane utilization logic is not well documented and has not been validated for North Carolina conditions.

Another investigation using the CORSIM simulation model was performed by Shen (11). She determined the required minimum short lane lengths for an intersection with triple left-turn lanes and downstream lane drops, using CORSIM simulations to develop guidelines for minimum short lane lengths. The average delay was the selected measure of effectiveness (MOE) as a function of various operational parameters such as green times, percentage of heavy vehicle in the left-turning traffic stream and the design free-flow speed of the downstream roadway. Shen found that the merging section length increased, the average delay decreased and she suggested that the minimum required short lane length should be determined based on the abrupt transition point of the average delay vs short lane length curve. However, the simulation results have not been validated with field data. Also, the proposed minimum short lane lengths appeared to be somewhat low (for instance, the longest proposed minimum short lane length is only 520 ft).

Leisch (12) studied intersections with "widened approaches" and proposed a set of requirements for short lane lengths. These are shown in Table 1.

Table 1. Recommended length of auxiliary lane beyond intersection (12)

<table>
<tr><th colspan="3">LENGTH REQUIRED FOR</th><th rowspan="3">TAPER
(feet)</th></tr>
<tr><th colspan="2">ACCELERATION *</th><th rowspan="2">MERGING *</th></tr>
<tr><th>DESIGN SPEED
(mph)</th><th>SHORT LANE
LENGTH (feet)</th></tr>
<tr><td>40</td><td>200</td><td rowspan="3">$12 \times G$
(G, Green interval in seconds)</td><td>200</td></tr>
<tr><td>50</td><td>525</td><td>250</td></tr>
<tr><td>60</td><td>900</td><td>300</td></tr>
</table>

* Use the larger of two values but not less than 300 feet

This table gives two values for the short lane length; one is a fixed value based on the required acceleration lane length, depending on the design speed; the other is calculated by multiplying the green time by 12. The larger value between the two is the recommended length, subject to a minimum of 300 feet. The methods Leisch used to derive this table are not explained in his paper. Guell (13) pointed out that the values are close to the acceleration lane lengths given by American Association of State Highway Officials (AASHO) in 1965 and stated, "It is assumed that Leisch took the values from this source (AASHO) and rounded the value up to the next 25 ft increment."

Guell (13) examined the theoretical and practical aspects of short lane length. The theoretical considerations were deterministic and included: (1) all drivers apply the same acceleration vs. time relationship (2) all vehicles start from a stop and have the same delay time and vehicle space to follow the vehicles immediately ahead and (3) all vehicles accelerate to the same final speed and maintain the speed. Based on these assumptions, the vehicle space and headway at the final speed can be obtained. As a practical matter, cases with different final speeds and different delay times were examined. However, the vehicle space and headway for each case resulted in spaces and headways that were too short to merge vehicles. He noted that some vehicles in the short lane can merge into the adjacent full lane when the acceleration rates vary. The short lane length was determined under the assumption that most vehicles in the short lane merge behind the vehicles in the full lane. It was concluded that the minimum short lane length must be sufficient to store the number of vehicles that could enter the intersection during green time. Although the assumption of vehicle merge order is unrealistic, the study provides insight on vehicle space and headway during the merging process.

Research on the lane utilization of auxiliary lanes which are added upstream and dropped downstream near the intersection (also known as a "New Jersey style" intersection) was carried out by McCoy and Tobin (14). A multiple linear regression model was developed to estimate the usage of the auxiliary lane by through vehicles as a function of the total length of the auxiliary lane and green time. The relationship found as a result of regression analysis for data collected at five sites in Lincoln, Nebraska is:

$$STR = 1.24 + 0.00058\,(Da + Db) - 0.021G$$

where

STR = mean number of through vehicles (passenger cars) discharging from the additional through lane, per cycle,

Da = length of additional through lane in advance of stop line (ft),

Db = length of additional through lane from the stop line to the start point of the taper (ft), and

G = green time for through and right-turn movement on approach (s).

McCoy and Tobin concluded that the use of additional through lanes is positively correlated with lane length and negatively correlated with green time. In addition, they found that the lane-use factor in the HCM generally overestimated the use of through lanes. Further research with a wider range of data was suggested because the small sample size they used did not provide strong confidence in the results.

A series of papers by Hurley (15-17) and research by Tarawneh (18,19) are examples of recent significant research on signalized intersections with lane drops. Hurley studied the effect of downstream lane reductions on left-turn lanes and through lanes. He categorized drivers as "captive" users and "choice" users in his research. Captive users were defined as "drivers who must use the auxiliary lane because of a need to turn right downstream of the intersection." Choice users were defined as "drivers traveling continuously through the intersection and downstream auxiliary lane, who have made an unforced decision to use the auxiliary lane." This categorization helps to analyze the driver lane choice behavior. Hurley's papers summarize the development of mathematical models to predict the utilization of auxiliary through lanes and double left-turn lanes with downstream lane drops. The model was generated via stepwise regression. A linear model structure proved unrealistic, so a hyperbolic tangent curve was applied in the model that represented lane choice behavior effectively. The general form of the model was:

$$P = a + b \tanh (b_0 + b_1X_1 + \ldots + b_nX_n)$$

where, P = the portion of choice users in auxiliary lane
a, b = model coefficients
$X_1 \ldots X_n$ = independent variables
$\beta_0 \ldots \beta_n$ = coefficients of variables

Hurley also found that the lane utilization factors of the HCM are overestimated. Hurley identified some significant factors in estimating the LUF for through lanes near a lane drop and he noted that:

- LUF increases with through flow rate, downstream auxiliary lane length, and the existence of a two-way-left-turn-lane (TWLTL) downstream; and
- Turn volumes to and from the subject roadway in auxiliary lane negatively affect LUF.

The distinct positive factors to LUF prediction at intersections with double left-turn lanes were:

- The length of outside left turn lane, and
- Product of area size and total left turn flow rate.

Tarawneh conducted research to observe and identify auxiliary lane utilization at intersections (18). Three factors – the auxiliary (typically a shared through and right) lane length, right turn volume and through/right turn lane group delay – were investigated. A 3-factor ANOVA was applied to test the significance of the factors. The research results indicate that the auxiliary lane use by through vehicles significantly increases as the auxiliary lane length is increased. Conversely, the auxiliary lane use decreased with an increase in right turns beyond the intersection signal. One important contribution by Tarawneh is evidence that the utilization of an auxiliary lane varies with the congestion level at the intersection. This appears logical: at an uncongested intersection drivers may use an auxiliary lane without worrying about the merge, at a congested intersection drivers are just satisfied to get through the signal, and with moderate congestion drivers likely shy away from an auxiliary lane.

Table 2 summarizes the finding of previous research. The research by Hurley and Tarawneh provided a good start in estimating the lane utilization at signalized intersections with lane drops, but these previous investigations could not be directly used in the present study for several reasons. First, the findings were never incorporated into the *Highway Capacity Manual* framework. Hurley's models need detailed estimates of turning movements upstream and downstream of the signal that are usually not available to designers. Second, the use of Tarawneh's findings in North Carolina is limited by the fact that his data were collected in the country of Jordan. Third, the sample sizes for these studies were very small: five sites in each of Hurley's studies and eight sites in Tarawneh's study. Fourth, Hurley did not look at three-lanes-to-two lane drops. Finally, neither researcher examined collision rates. Thus, despite the good contributions of previous research in this area there is a need for a statistically valid study that focus on the variety of lane drop intersections in North Carolina.

Table 2. Summary finding of literature review on lane utilization

Category	Source	*Notes*
Lane flow estimation without lane drops	Australian (1984)	• Equal degree of saturation
	Canadian (1985)	• Equal flow ratio
	Swedish (1978)	• Equal degree of saturation
	Rouphail and Nevers (2001)	• Equal back of queue
Lane flow estimation with lane drops	Swedish (1978)	• Only considers upstream short lane such as turning bays
	aaSIDRA	• Downstream lane drop effect is considered. • (Australian) Lane utilization of the auxiliary lane is determined depending on downstream short lane length and other parameters, which need calibration.
	Hummer (1999, 2000)	• Simulated lane drop with CORSIM
	McCoy and Tobin (1982)	• Auxiliary lane usage is positively correlated with auxiliary lane length and negatively correlated with green time.
	Hurley (1995,1997,1998)	• Drivers are categorized as captive and choice drivers. • Auxiliary lane length, right turn volumes to and from the driveways in auxiliary lane and the existence of TWLTL downstream are found factors. • Small sample size.
	Tarawneh (2000, 2001)	• Auxiliary lane length, right turn volume are the factors affecting auxiliary lane utilization. • No effect of lane group delay. • Small sample size. • Data from other country (Jordan)
Applicable Software	HCS	• Default LUF – no effect of downstream lane drops
	SIDRA	• Can replicate lane drop effect but not calibrated with US conditions
	CORSIM	• Merging algorithm murky

CHAPTER 3. FIELD DATA COLLECTION AND METHODOLOGY

This chapter explains the procedure for data collection conducted by the research team including the site identification task. The chapter has seven subsections as listed below.

- Site selection criteria
- Site selection
- Field data collection
- Data reduction
- Data statistics
- Operational data analysis methodology
- Collision data collection and analysis methodology

3.1. Site Selection Criteria

Identification of appropriate sites and data collection began in October 2002 and lasted to December 2003. At the beginning of this period, NCDOT provided a list of 59 intersections that may have lane drops. A survey was conducted by the research team to identify additional sites to fulfill the target of 100 data collection sites. One hundred sites would allow for significant samples in all six categories shown in Chapter 1. Locations in the Raleigh, Durham, Chapel Hill, Research Triangle Park, Winston-Salem, Greensboro, High Point, and Charlotte areas were investigated. Most of major roadways examined were in the Triangle area (Raleigh, Durham, Chapel-Hill and RTP area). For other areas, candidate sites were selected based on the North Carolina state transportation map and city maps. Intersections where the number of lanes varied from an upstream to a downstream approach on the maps were chosen as candidate sites.

The criteria for identifying a site were as follows. If a downstream lane drop existed or a lane discipline changed from “through lane” to “turning-only lane” within a half mile from an intersection, the intersection was identified as a candidate intersection. The distances from the stop bar of the subject intersection to the end of the downstream short lane, which is equivalent to the beginning point of the taper, were approximately measured by the trip odometer in the survey vehicle. When dual left turn lanes were observed during the examination of the arterials, downstream conditions were inspected within a half mile or until the next signalized intersection was encountered, whichever occurred first. In the case of dual left turns, the downstream receiving lanes had to be two lanes with lane drops to qualify. If the number of downstream receiving lanes was more than two, the intersection was not included in our list even if there was a lane drop.

3.2. Site Selection

For the North Raleigh area, most of major corridors were examined. This included Capital Boulevard, Atlantic Avenue, Falls of Neuse Road, Six Forks Road, Lead Mine Road, Creedmoor Road, Ray Road, Sawmill Road, Lynn Road, Spring Forest Road, Millbrook Road, New Hope Church Road and Louisburg Road. For the Research Triangle Park (RTP) area, the major investigated arterials were Alexander Drive, Cornwallis Road, Jenkins Road, and NC 54. Many sites were studied in the Durham and Chapel-Hill area, including Chapel Hill-Durham Boulevard, NC 751, University Drive, Cornwallis Road, Erwin Road, Fayetteville Street, Fayetteville Road, Raleigh Road, Franklin Street, NC 86, and Jones Ferry Road.

After the initial identification of sites, it was found that the number of intersections with “New-Jersey-Style” auxiliary lanes was too low to provide meaningful sample sizes and this category was dropped from consideration. Some sites were dropped due to construction that changed the geometry and traffic patterns while some were excluded because of evolution of the site selection criteria. For instance, any intersections having lane drops from the leftmost lane were eliminated from the list to avoid bias because there were not enough left lane drops to provide adequate sample sizes.

After visiting over 170 candidate intersections, the site selection process yielded a total of 94 sites. Table 3 shows the number of sites by category.

Table 3. Number of study sites selected by geometric category.

Category	Number of sites						
	2 through lanes		2 left turn lanes		3 through lanes		Total
	Shared lane (2TS)	Exclusive turn lane (2TE)	Onto ramp (2LR)	Onto surface street (2LS)	Shared lane (3TS)	Exclusive turn lane (3TE)	
Collision only	12	10	11	6	4	4	47
Operational only	-	-	-	2	1	-	3
Collision and operational	12	10	8	5	5	4	44
Total	24	20	19	13	10	8	94

Only sites that satisfied several criteria were selected for analysis. All intersections were checked for recent changes in geometry. Intersections where we knew of any changes or upgrades within the last three years were excluded, due to the maturation problem for collision analysis. Intersection history was verified with intersection signal plan and discussion with NCDOT personnel. Second, high-volume intersections were given higher priority. Intersections on major arterials were assumed to have high traffic volumes and those intersections were included in the list. Finally, intersections having unusual geometric configuration were excluded (e.g., downstream lane drop of the left most lane). However, this last exclusion was not applied to the "dual left turn onto ramp" category, where we did collect some data on sites with left lane drops.

For operational analysis, intersections located outside the Triangle, Triad and Mecklenburg County areas were given lower priority than those in the Triangle area to economize on the data collection costs.

Table 4 shows the number of selected sites and distribution by region. Nearly 60 percent of the sites are in the Triangle area. However, the collision sites are well distributed across all the major cities in North Carolina.

Table 4. Selected sites distribution by region

Category	Number of sites								Total
	Triangle					Triad	Charlotte	Other	
	Raleigh	Cary	Chapel Hill	Durham	RTP				
Collision only	2	1	9	7	0	8	12	8	47
Operational only	3	0	0	0	0	0	0	0	3
Collision and operational	12	12	3	3	9	3	0	2	44
Total	17	13	12	10	9	11	12	10	94

3.3. Field Data Collection

Three types of field data were considered for the research: geometric, traffic count and signal control data. The geometric data collection procedure was applied to all selected intersections. However, traffic count and signal data collection procedures only applied to intersections for which operational analysis were carried out. Detailed data collection procedures are described below.

3.3.1. Geometric Data Collection

NCDOT provided the intersection signal plan which includes geometric drawings, and the information in the plan was confirmed and corrected through measurements taken during the site visit. The geometric data collection form is provided in Appendix A. The geometric data collection mainly consists of three parts: general information, geometric characteristics and schematic drawings. First, general information includes date, time, weather, intersection location, the intersection category, intersection ID number, subject movement approach and posted speed limit of each approach. Second, detailed geometric and other relevant data were recorded. There are two types of lane drops which are defined in this study. One category is the typical lane drop: one lane merges onto the adjacent lane via an acceleration lane and taper (Type I lane drop). The other type is a lane drop due to a lane discipline change (e.g., the through lane becomes a turning-only lane - Type II lane drop). The Type II lane drop does not apply to “dual left turn onto ramp” category which has only Type I lane drops. Figure 7 illustrates the lane drop types. After recording the type, the analyst measured the short lane length. The definition of short lane length is different depending on the type of short lane. For a Type I lane drop,

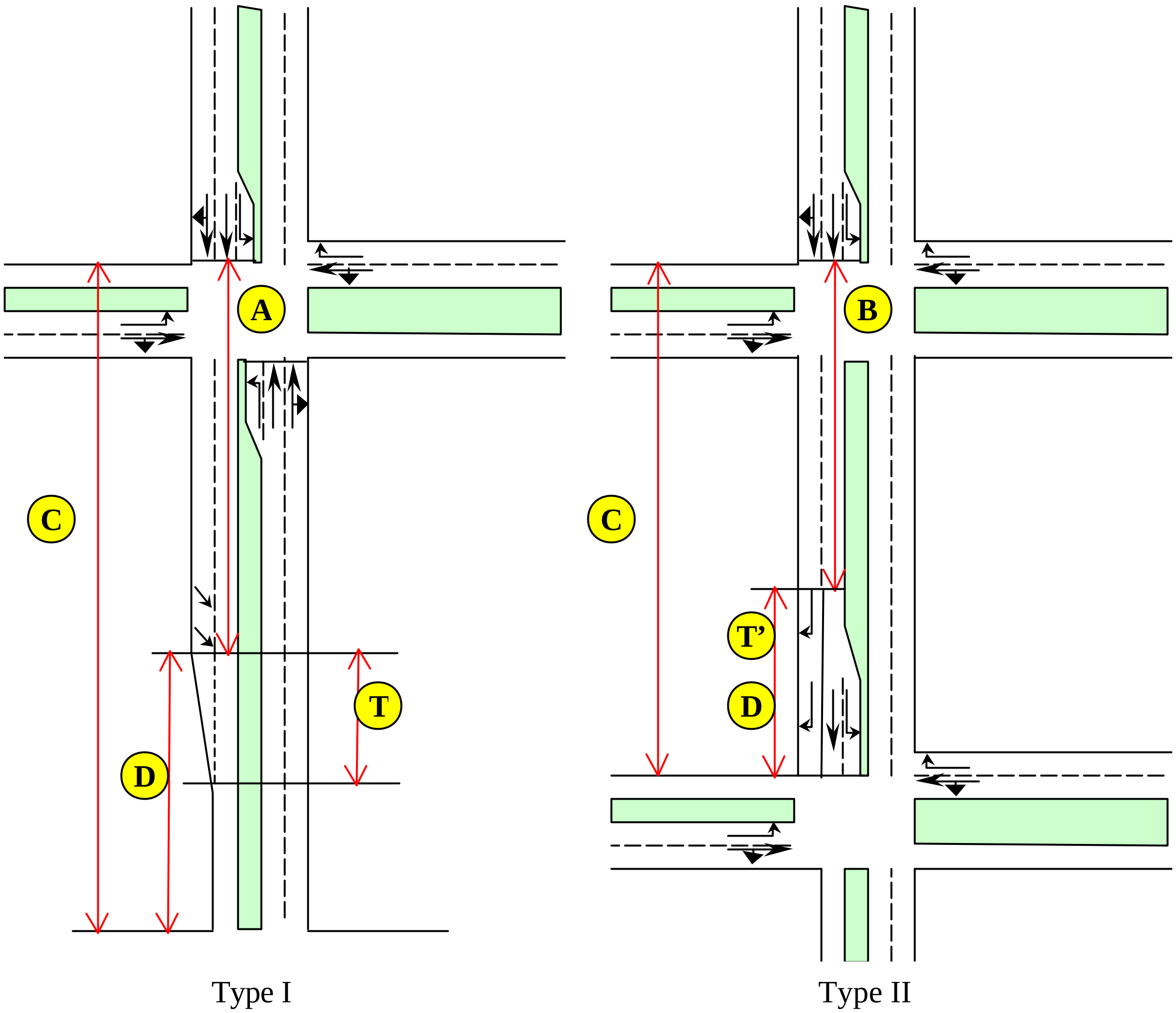


Figure 7. Lane drop types and short lane length definitions

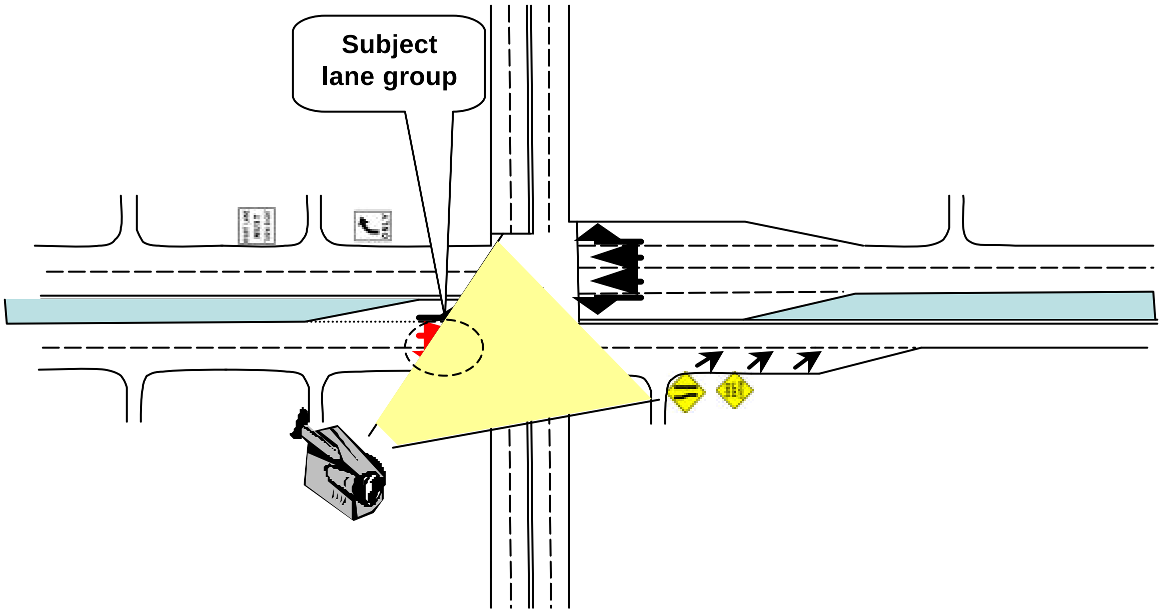
the short lane length was measured from the stop bar at the upstream intersection approach to the beginning of the taper (distance A). For Type II lane drop, the short lane length was measured from the stop bar at the intersection to the first pavement marking that informs lane usage change (distance B). If the short lane length was shorter than 1,500 feet, a data collector measured the length (distance A or B) with a measuring wheel. Otherwise, first the distance between the subject intersection and the next downstream intersection (distance C) was measured using the odometer of a vehicle and then the distance between the ending point of the short lane and the stop bar of the next downstream intersection (distance D) was measured with a measuring wheel so that the short lane length could be calculated. The taper lengths were measured as well. The taper length measurements are also different depending on the lane drop type. For a Type I lane drop, the taper length was measured from the beginning of the taper to the end of the taper (distance T). In the field, it was easy to distinguish the beginning point of a taper for most intersections. Typically, the broken pattern of lane line changes to a dotted pattern at the beginning of taper. When this pattern was not discernable, observers measured lane widths of the several locations near the taper begin point and the point where has significantly different lane width was regarded as the taper begin point. For a Type II lane drop, the length from the first pavement marking to the stop bar of the end of the lane was regarded as a taper and was measured (distance T'). The cross street width was measured from the stop bar to the far side curb-to-curb line. The analyst measured storage lengths of turning lanes and taper lengths in feet and observed whether and how often the lanes overflowed during the typical signal cycles. In addition, the distance to upstream signalized intersection was measured from stop bar of the subject intersection to the stop bar of the upstream signalized intersection.

Signing information near the lane drop was noted. The types and location of signs were identified. The types of signs were broadly classified into two categories, warning signs and regulatory signs. Then, each sign type was specified exactly. Distances from the stop bar to each sign were measured. If any sign was located upstream of the subject intersection a negative value for distance was recorded. Existence, location and number of pavement markings that inform drivers of the presence of the lane drop such as "merge left" or "right turn only" were recorded. The analyst recorded the number and activities of driveways both upstream and downstream. Instead of counting all entrance and exit volumes of each driveway the intensities of driveway traffic volume were observed for a short time and recorded qualitatively. The intensity of each driveway was rated in three levels: high, medium and low. In addition, the presence of a two-way-left-turn-lane or a left turn bay in the mid-block was recorded. Intersections where the subject movement is a left turn needed additional data such as left-turn lane length and taper length for each left turn lane. A supplemental data collection form for dual left turn intersections was used and is attached in Appendix A. Finally, the analyst developed intersection schematics that include street names, posted speed limits, land uses, lane widths, approach grade, north arrow, and other relevant information.

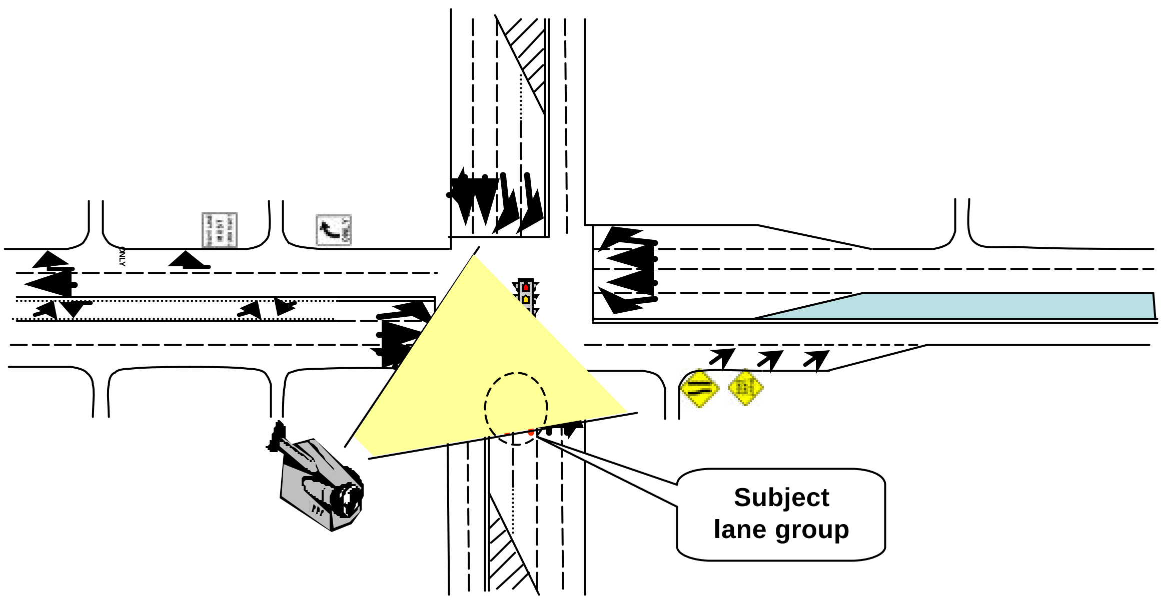
3.3.2. Traffic Data Collection

Traffic data were collected at operational intersections using the forms shown in Appendix B. Traffic flow data were collected for three hours at each intersection. We collected data from 7 am to 10 am or 4 pm to 7 pm; these are periods that cover both off-peak and peak hours. A video camera was set at the intersection for later data retrieval. The camera was aligned to capture as much information as possible, such as the movements of the non-lane drop approaches and signal indications of the cross street. If it was not possible to see all other movements, the camera was aligned to capture at least the subject movement and traffic signal indication for the subject movement. Video camera time and time of the analysts' watches were synchronized for consistent results. Two analysts counted traffic volumes for movements on the subject movement's approach and the opposing approach. Right turn volumes on the subject approach (when the subject movement is a through movement) were counted and recorded every cycle by watching recorded videotapes. The movement volumes of opposing approaches were counted and recorded every cycle as well to verify which approach is the critical one. Lane volumes for the subject movement were counted in the field if the lane volume counting task did not affect the data quality; otherwise, lane volumes were counted by watching recorded videotapes.

The location of the video camera was critical. It was located as close as possible to the subject movement at the intersection. The camera was also hidden as much as possible from drivers in order to avoid distraction. Figure 8 shows the locations of the camera for through (a) and left turn (b) movements.



(a) Through movement camera location



(b) Left-turn movement camera location

Figure 8. Camera location by subject movement

Two observers counted lane volumes, pedestrian volumes and the number of queued vehicles for the subject movement with manual counters. Figure 9 shows the locations of the observers. Assuming that WB traffic is the subject movement, Observer 1 counted westbound lane volumes and recorded the number of queued vehicles at the start of the green for the subject movement. The observer also monitored whether there was a

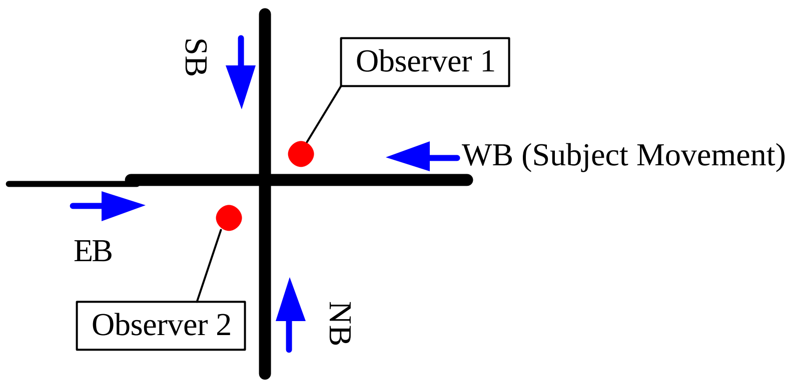


Figure 9. Location of traffic counters

left turn movement spillback each cycle and recorded when/if it happened. In addition, the observer observed upstream driveway activities and recorded the results if the vantage point allows driveway observations. The other observer counted all traffic volumes on the eastbound approach and recorded the start of green time and the green ending time of the movements. Saturation headway by lane and lost times were measured by extraction from the videotapes.

Intersections in the dual left turn (2LS and 2LR) categories have unique characteristics. Therefore, the data collection procedure was somewhat different from intersections with a through subject movement. For 2LR intersections, the left turn movement does not have any conflicting movements which affect the lanre volume distribution so only the subject left turn lane volume was collected. Observations of driveways activity were not required either. As a result, a single observer could accomplish the counting and queue length measurement tasks. In case of a 2LR intersection having a channelized right turn movement from the opposing direction which merges with the subject left turn movement, the right turn volume was needed to be counted as well. The opposing right turn volume and the traffic signal timings of the subject left turn movement were recorded by watching videotapes.

In the 2LS category (dual left turns on surface streets), the subject left turn shares its signal phase with the opposing left turn movement. The opposing left turn traffic volumes were counted for the purpose of verifying the critical movement. The opposing left turn volume-counting task was performed by a second observer who also recorded the start and end time of the left turn phase. Simultaneously, the second observer monitored and recorded downstream driveway activity. The primary observer counted traffic volumes of the subject left turn movement and measured the queue length at the beginning of the green.

3.3.3. Signal Data Collection

Since most of intersections were operated under semi-actuated or full-actuated control, an observer needed to measure green, yellow, all-red and red time for the subject movement every cycle. However, if the recording task of the signal data degraded data quality the signal data were not recorded. In this case, the signal data were retrieved from the recorded videotapes.

3.4. Data Reduction

All collected data were coded in a spreadsheet for easy data retrieval and analysis. Data were reduced by watching the recorded video images from the field. Data reduction involved:

- Confirmation of green time;
- Saturation headway measurement;
- Lane volume counts for some intersections;
- Right turn volume counting;
- Conversion of 15-minute volume to hourly flow rate; and
- Truck volume counts.

The start of green and end of yellow times were recorded in the field for most of the sites. Yellow time was measured separately in order to calculate green time. However, it was found that the green times recorded in the field tended to be about two seconds shorter for each cycle than times recorded from the videotape. This is likely caused by differences in the start and stop points. In other words, the observer could be prepared to push the stop watch button when the signal turns yellow. However, he or she could not predict the start of green time. This caused one or two seconds of reaction time and resulted in shorter green times. For this reason, all recorded cycles were examined from the videotape and corrected if there was more than one second of green time difference.

The method to measure saturation headway in the field followed the method described in Appendix H of the HCM 2000 (1). Cycles with more than five queued vehicles in both the full lane and short lane were selected to measure saturation headway. The saturation headway period started when the fourth queued vehicle passed the stop line and ended when the last queued vehicle passed the stop line. Then, the saturation period was divided by the number of vehicles involved in the saturation period. The average headway by lane was calculated as the total measured saturation period divided by the number of vehicles involved.

Lane volumes were counted in the field for the most of intersections that had two lanes for the subject movement. When the traffic demand was heavy the lane volume counting task was not performed in the field to ensure high data qualit. For the intersections with three lanes, the lane volumes of one or two lanes were counted at the site. The remaining lane volume counting was done by watching videotapes. After finishing the lane volume counts, the average lane volume and the lane utilization factor were calculated.

The intersection categories of 2TS (two through lanes with shared right turn lane) and 3TS (three through lanes with shared right turn lane) needed right turn volume counting. This was done from the videotapes.

For operational data analysis, 15-minute traffic counts were converted to one-hour flow rates. Every cycle has different length because most of the intersections operated under actuated signal control and most 15-minute periods did not contain an integer number of cycles. For this reason, flow rates had to be calculated carefully. First, we found the number of cycles that came close to the 15-minute time period in duration. Then, we calculated the flow rate accounting for the duration of the cycles. For instance, say the duration of eight cycles was 953 seconds and we counted 300 vehicles during these eight cycles. The flow rate was then calculated as:

$$300 \times \frac{3600}{953} = 1133 \text{ vph}$$

Truck volumes were also counted with the recorded videotape. Vehicles with more than six tires were recorded as heavy vehicles. Bus traffic volumes were counted separately when a bus made a stop downstream or upstream of the subject intersection. Any buses that did not stop near the intersection were classified as heavy vehicles.

3.5. Overall Statistics

It was found that the collected data covered a very wide range of variables, which gives us confidence that the analysis will be robust. The observed lane utilization factors ranged from 0.511 to 1.000 with an average of 0.733 and a standard deviation of 0.112. Table 5 shows the minimum and maximum values of lane utilization that are theoretically possible, that were observed in our sample of intersections, and that are provided by the HCM.

Table 5. Ranges of lane utilization factor

<table>
<tr><th rowspan="2"></th><th colspan="2">2-lane</th><th colspan="2">3-lane</th></tr>
<tr><th>MIN</th><th>MAX</th><th>MIN</th><th>MAX</th></tr>
<tr><td>Thoretical</td><td>0.500</td><td>1.000</td><td>0.333</td><td>1.000</td></tr>
<tr><td>Observed</td><td>0.511</td><td>1.000</td><td>0.529</td><td>0.962</td></tr>
<tr><td rowspan="2">HCM Defaults</td><td>TH</td><td>LT</td><td colspan="2" rowspan="2">0.908</td></tr>
<tr><td>0.952</td><td>0.971</td></tr>
</table>

The short lane lengths of the 94 studied intersections ranged from 73 feet to 2370 feet. Table 6 shows the minima, maxima, averages and standard deviations of some key variables gathered during the data collection.

Table 6. Ranges, mean and standard deviation of some key operational variables

	Lane utilization factor (f_{LU})*	Average lane volume (vphpl)*	Short lane length (ft)	Taper length (ft)	Number of lane drop information signs	Number of lane drop information markings
Minimum	0.51	27	73	92	0	0
Maximum	1.00	836	2370	1121	6	11
Average	0.73	265	757	390	1.57	4.03
Standard deviation	0.11	173	453	205	1.08	1.79

* From 46 operational analysis sites

3.6. Operational data analysis methodology

The HCM notes that the lane utilization factor (f_{LU}) "accounts for the unequal distribution of traffic among the lanes in a lane group with more than one lane." The relevant equation was provided earlier and is repeated here for convenience:

$$f_{LU} = \frac{v_g}{(v_{g1}N)} = \frac{\text{average lane volume}}{\text{highest lane volume in a lane group}}$$

where

v_g = lane group flow rate (vph)

v_{g1} = highest lane volume in a lane group (vph)

N = number of lanes in lane group

This factor indicates the skewness degree of the lane volume distribution. High lane utilization represents a more even lane volume distribution over the approach lanes and a low value indicates an uneven lane volume distribution.

The lane utilization factor plays an important role in the calculation of signalized intersection capacity. The signalized intersection capacity is the product of the saturation flow rate and the green ratio. The saturation flow rate is obtained by multiplying the ideal saturation flow rate by several adjustment factors. The lane utilization factor is one of those adjustment factors. Therefore, in the HCM, low lane utilization leads directly to a low saturation flow rate which leads to low capacity. The equation is:

$$capacity = s \times \frac{g}{C}$$

where $\mathrm{s} = \text{saturation flow rate}$

$= \mathrm{s_o} f_{LU} \prod f_i$

$\mathrm{s_o} = \text{ideal saturation flow rate}$

$f_{LU} = \text{lane utilization adjustment factor}$

$f_i = \text{other saturation flow adjustment factors (e.g. lane widths, parking, etc)}$

$\mathrm{g} = \text{green time}$

$\mathrm{C} = \text{cycle length}$

Since capacity is an important indicator of intersection performance in its own right and is a key factor in predicting delay, upon which level of service is based, the importance of the lane utilization factor is clear. This section describes how we developed our lane utilization models.

3.6.1. Candidate factors affecting lane utilization

Based on the collected geometric and operational data, the analyst identified 15 possible candidate factors affecting lane utilization. The factors are listed and explained below. Figure 10 illustrates the factors schematically. The label preceding each factor can be referenced to the Figure.

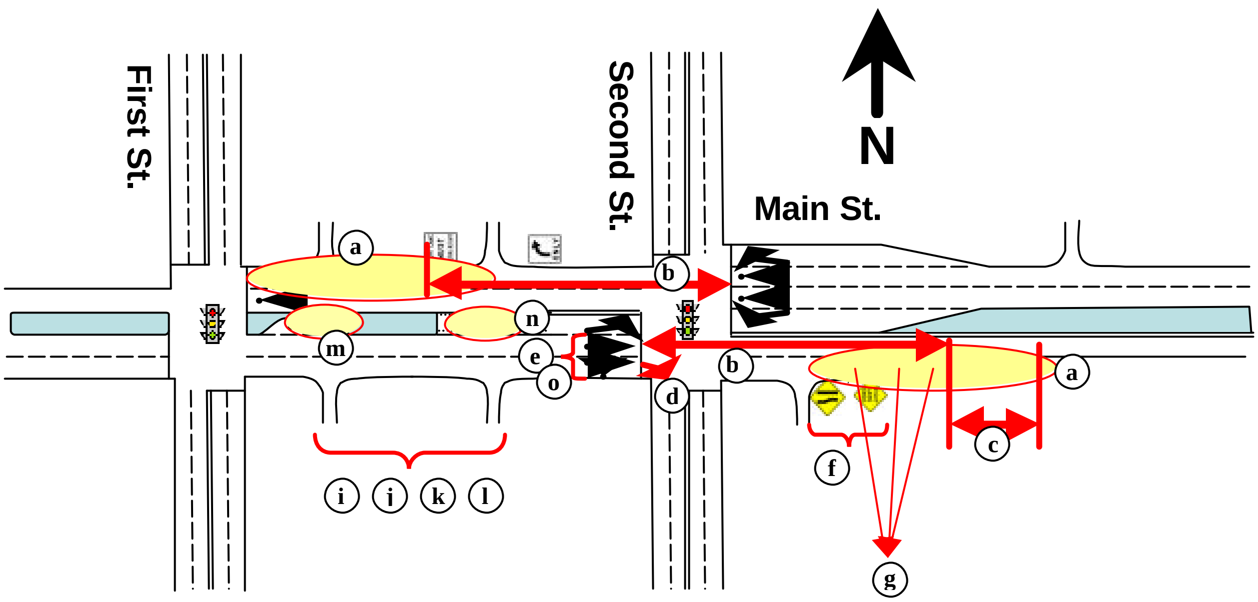


Figure 10. Pictorial representation of candidate factors

a) Lane drop type - ***drp_type***

As described previously there are two types of lane drops:

i. Type I lane drop: Mid-block physical lane drop (e.g., the eastbound Main St. movement downstream of the Main St./Second St. intersection in Figure 10.)

ii. Type II lane drop: Changes of lane discipline from through lane to right turn only lane (e.g., the westbound movement downstream of the Main St./Second St. intersection in Figure 10.)

Note that in case of 2LR (dual left turn onto freeway junction), the definition of lane drop type is different. Since 2LR intersections only have physical merging lane drop "right side lane drop" is defined as Type I lane drop and "left side lane drop" is represented with Type II lane drop.

b) Short lane length (ft) – ***short***

   Short lane lengths were measured from the stop bar of the relevant approach to the beginning of the taper for Type I lane drop. The length was measured from the stop bar of the relevant approach to the first pavement marking that informs lane discipline change for Type II lane drop.

c) Taper length (ft) – ***taper***

   Taper lengths were measured from the beginning of the taper to the end of the taper for a Type I lane drop. The length was measured from the first pavement marking that informs drivers of a lane discipline change to the end of the lane for a Type II lane drop.

d) Right turn volume on shared lane (vph) - ***rtvol***

   Flow rate of right turn movement in the shared through/RT lane for 2TS and 3TS categories.

e) Heavy vehicle percentage (%) – ***hvpct***

   Percentage of heavy vehicles for the subject movement

f) Number of signs – ***n_sign***

   Number of signs that inform drivers of the lane drop or change of lane discipline ahead

g) Number of markings – ***n_mark***

   Number of pavement markings that inform drivers of the lane drop or change of lane discipline ahead

h) Location of the first lane drop information (ft) - ***fstinfo***

   Location of first lane drop information (either pavement marking or sign) measured from the stop bar (negative for upstream and positive for downstream)

i) Density of driveway upstream left side (#/mile) - ***DdrwyupL***

   Number of driveways per mile upstream of the subject intersection on the left side of the subject traffic

j) Density of driveways upstream right side (#/mile) - ***DdrwyupR***

Number of driveways per mile upstream of the subject intersection on the right side of the subject traffic

k) Density of driveways downstream left side (#/mile) - ***DdrwydnL***

Number of driveways per mile downstream of the subject intersection on the left side of the subject traffic

l) Density of driveways downstream right side (#/mile) - ***DdrwydnR***

Number of driveways per mile downstream of the subject intersection on the right side of the subject traffic

m) Existence of upstream mid-block left-turning accessibility (yes/no) - ***MBAup***

Existence of a two-way-left-turn-lane or mid-block left-turn bay upstream of the subject intersection

n) Existence of downstream mid-block left-turning accessibility (yes/no) - ***MBAdn***

Existence of a two-way-left-turn-lane or mid-block left-turn bay downstream of the subject intersection

o) Average lane volume – ***Avg_lnvol***

Average lane volume of the relevant lane group in vehicles per hour per lane (vphpl)

3.6.2. Regression Analysis Methodology

Lane utilization prediction models for each intersection type were developed based on the model development steps. The analyst applied six steps to develop the lane utilization factor prediction model:

1. Scatter plot examination
2. Multicollinearity tests
3. Stepwise regression analysis
4. Residual plot examination
5. Model transformation
6. Model explanation

First, scatter plots between the response variable (lane utilization factor) and explanatory variables were produced to examine patterns of explanatory variables and to check if any data needed transformation. Figure 11 shows the examples of scatter plots. Figure 11(a) is the scatter plot of lane utilization and short lane length. It reveals a positive and strong relation. Figure 11(b) shows a scatter plot between lane utilization and merging lane taper length, with no evident pattern between the two.

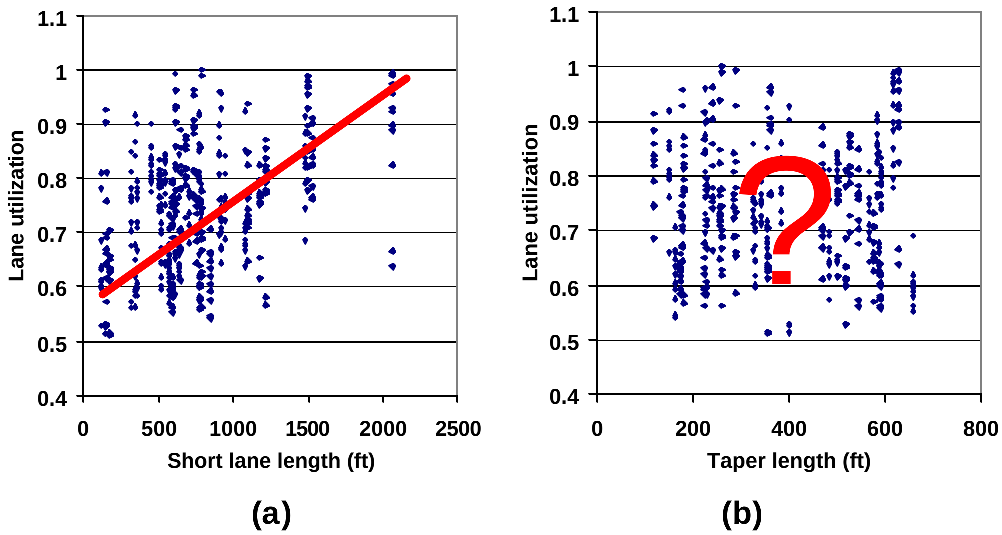


Figure 11. Examples of scatter plots

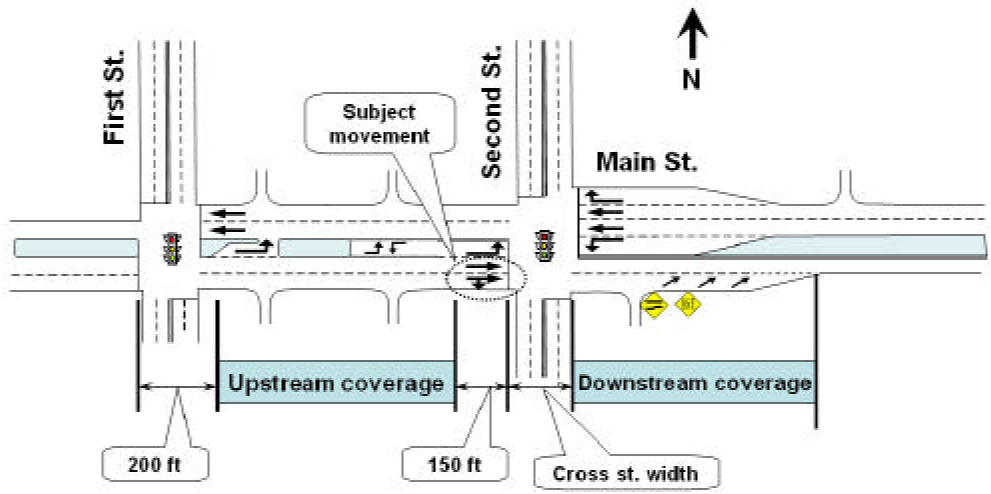
Second, multicollinearity tests were conducted among the independent variables using the SAS® (20) statistics software package. Multicollinearity is a serious problem in regression. When two independent variables are highly correlated, they contain virtually identical information. If the two variables are included in the model they do contribute a lot to the model even when the variables are not individually significant. When a model exhibits a collinearity problem, it might have a low overall goodness of fit while the individual variables appear to fit very well. Using SAS® the Variance Inflation Factors (VIF) were examined and the explanatory variable which has the highest VIF value was excluded step by step until the VIF value of all the variables were less than a standard value.

Third, based on the candidate variables that qualified after the previous step, sequential variable selection techniques were applied. For each intersection category, forward selection, backward elimination and stepwise regression were conducted to identify those factors affecting lane utilization. The forward selection procedure starts only with a constant mean and adds one explanatory variable at a time. All models with a single variable are compared and the model with the best fit is chosen. Then, one explanatory variable is added at a time until the fit does not improve any more. The backward elimination starts with all possible variables and eliminates one variable at a time. This process continues until a model has all significant variables and no further elimination significantly changes the fit. In stepwise regression, a constant mean model without any explanatory variable is the starting model. Each step consists of a forward selection and a backward elimination. Variables are removed and added one by one until all selected variables are significant with a 95% confidence level. The three methods to identify the significant variables can result in different final models. If this happens, the candidate models are examined against each other and the final model is selected. After finding the significant variables, the model is examined using residual plots and the need for model transformation is determined. Next, the logic of the final model is examined. For instance, if there are any unexplainable coefficient signs (for example, longer short lane lengths resulting in lower lane utilization) the model and data are investigated further. Finally, the implications of the model were examined and data ranges of model application were noted. Overall, this thorough process results in an optimal model to explain lane utilization, given the data that were collected.

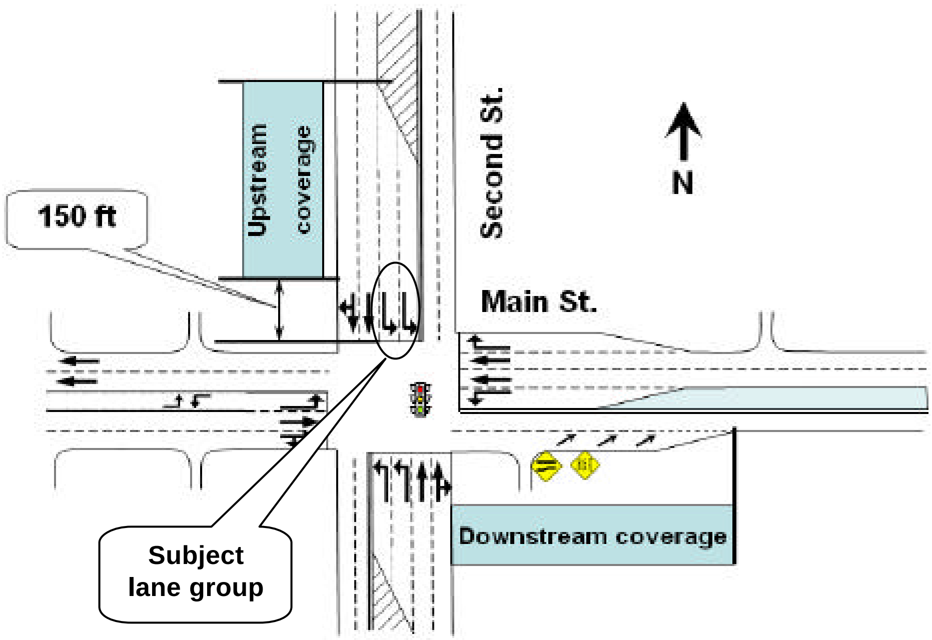
3.7. Collision data collection and analysis methodology

Collision data were analyzed to find whether lane drop-related geometric variables affect safety in the vicinity of the intersection. Collision data were retrieved by the NCDOT Traffic Safety Systems Management Units using NCDOT's Traffic Engineering Accident Analysis System (TEAAS) for the time period from June 1, 2000 to May 31, 2003. Collision frequencies are usually used for collision data analysis. In this study, collision rates with the units of collisions per 100 million mile of travel were used for this collision data analysis because the sections of the subject intersections involved various section lengths and traffic volumes which would have biased our findings if not accounted for. The error introduced in used collision rate for this analysis is likely a small one due to the homogeneity of the sites and is outweighed by the convenience of viewing and using the results in terms of rates.

All sites were divided into three sections: upstream, intersection and downstream. An upstream section is defined in a different way depending on the subject movement. For intersections with dual left turns, the upstream section was defined from the stop bar to the end of the taper of the left turn bay. The upstream section length for a site involving a through movement was measured as the distance between a point 150 feet upstream of the stop bar at the subject intersection to a point 200 feet downstream of the prior intersection. This definition effectively excluded collisions occurring at the intersections. The downstream length was defined for all intersection types as the sum of the short lane length and the taper length minus the cross street width. Figure 12 depicts the definitions of upstream and downstream sections for the two broad categories of intersections.



(a) Through movement lane group



(b) Left turn movement lane group

Figure 12. Pictorial representation of upstream and downstream coverage for collision analysis purposes

Collision counts during the three years in each section were obtained from the collision strip map provided by NCDOT Traffic Safety Systems Management Units. Average daily traffic (ADT) estimates were obtained from the website of the NCDOT Traffic Survey Unit. Based on the collision counts, section lengths, and ADT data, the collision rate for each section was calculated from the following equation in units of collisions per million vehicle miles traveled (acc/mvmt).

$$Collision\ rate = \frac{Collision\ count \times 1{,}000{,}000}{365 \times 3yr \times ADT \times section\ length}$$

Since collisions at intersection involve many factors other than the lane drop effects, intersection collisions were excluded in this study. The pattern of collision rates were examined and reported in the results chapter.

CHAPTER 4. OPERATIONAL DATA ANALYSIS RESULTS

Based on the model development procedure as described in the previous chapter, models for each category were produced. This chapter will discuss significant variables, degree of fit and sensitivity analysis, and the recommended models will be provided. A summary table of the models is included and comparisons between the field measured delay and HCM derived delay will be discussed in the last section of this chapter.

4.1. Lane Utilization Prediction Models

In this section, the model development procedure will be explained. The candidate variables after each step are listed. An explanation of the recommended models and the meaning of the significant variables will be provided. Sensitivity analyses of the significant factors were performed for each model and will be explained. The sensitivity of each factor was calculated with the following equation:

Sensitivity $_{i}$ ***(% per unit) = (base*** f_{LU} ***-*** f_{LU} ***with one unit change in variable i) / base*** f_{LU}

where

base f_{LU} = f_{LU} calculated with average value of each variable and 0 for indicator variables

4.1.1. 2TE intersections

Scatter plots of explanatory variables for the 2TE intersection category did not originally indicate any need for data transformation. The average lane volume (avg_lnvol), short lane length (short), lane drop type (drp_type), number of marking (n_mark), and first lane drop information (fstinfo) variables exhibited a positive relation to lane utilization while the existence of downstream mid-block accessibility (MBAdn) variable was negatively correlated with lane utilization. The multicollinearity test screened the candidate variables which were reduced to eight: Avg_lnvol, Short, N_sign, N_mark, DdrwyupR, Drp_type, MBAup, MBAdn. After screening the data for quality, a total of 88 data points were available for the 2TE model development.

The multiple linear regression equation for the 2TE intersection had five significant explanatory variables with an R^2 = 0.717 and standard error = 0.066. The proposed model is:

f_{LU} = 0.572 – 0.164 Drp_type + 0.110 MBAdn + 0.138 ShortK + 0.436 Avg_lnvolK – 0.074 * N_sign

where	f_{LU}	=	Lane utilization factor	
	Drp_type	=	1	If Physical lane drop (Type I)
			0	If Lane usage change lane drop (Type II)
	MBAdn	=	1	If downstream mid-block left-turn accessible
			0	Otherwise
	ShortK	=	Short lane length (ft) ÷ 1000	
	Avg_lnvolK	=	Average lane volume (vphpl) ÷ 1000	
	N_sign	=	Number of signs	

However, a residual plot of the model (Figure 13) indicated a fan shape of the relationship which means that model transformation is required. A lognormal transformation was applied and the final model form is shown below. The transformed model had a slightly higher R^2 and standard error than the previous model with values of 0.727 and 0.0875, respectively.

f_{LU} = exp(- 0.539 – 0.218 Drp_type + 0.148 MBAdn + 0.178 ShortK + 0.627 Avg_lnvolK – 0.105 * N_sign)

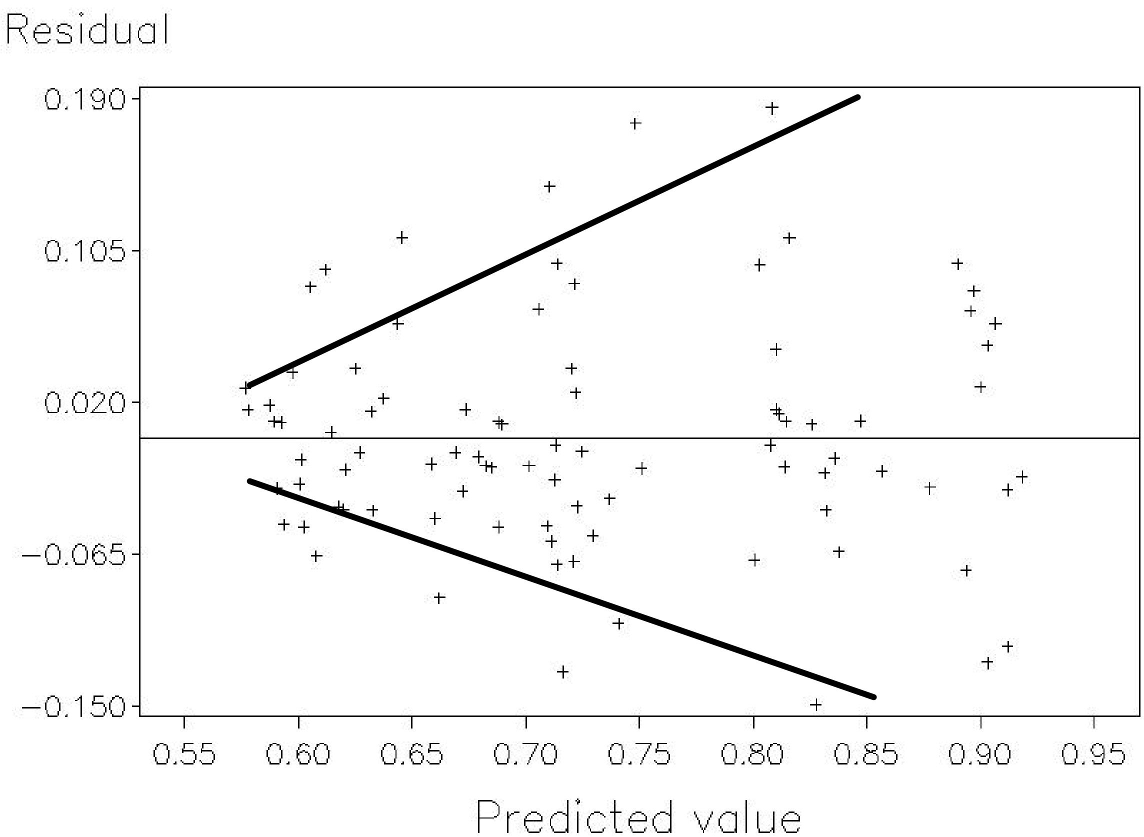


Figure 13. Residual plots of 2TE linear model

This model implies that the short lane length, lane volume and left turn availability at the downstream intersection affect lane utilization positively. In other words, a long lane length before the drop, high traffic demand and existence of mid-block left turn bay or TWLTL downstream of the intersection of interest mean higher utilization of the short lane at the signal. By contrast, the physical mid-block lane drop and number of signs variables showed negative effects on lane utilization. The positive effects may be because:

- Longer lanes before the drop might relieve stress for drivers.

- When an intersection gets congested the lane change stress might be compensated for by getting through the signal.
- Drivers may be avoiding the impedance caused by decelerating left-turning vehicles ahead.

The negative effects may be because:

- Downstream right turn vehicles are captive users in the short lane when the lane discipline changes.
- Information on a lane drop might encourage or remind drivers to use the full lane.

The relationship between the observed lane utilization factor and the predicted factor is shown in Figure 14.

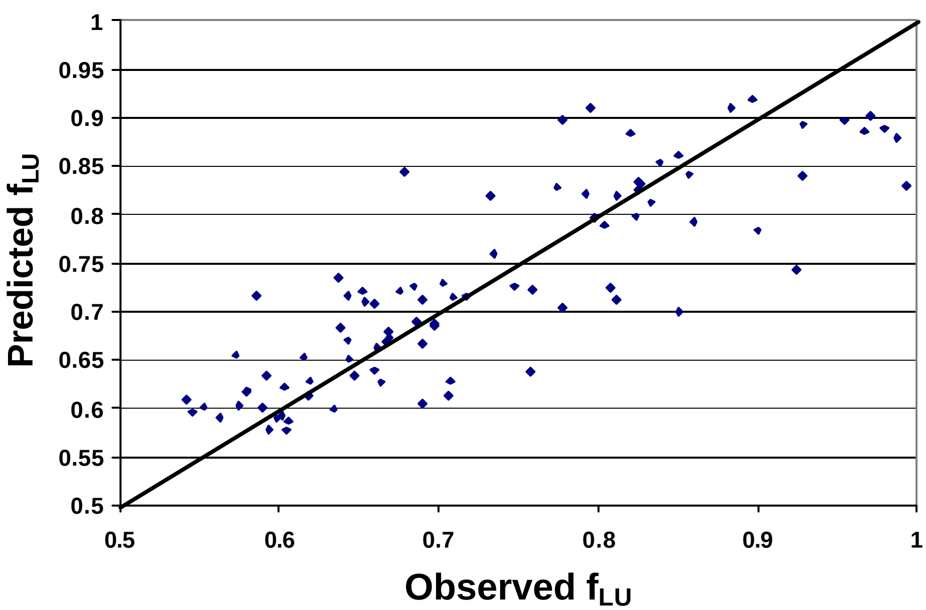


Figure 14. Observed and predicted lane utilization factor for 2TE

Figure 15 shows the effect of short lane length and average lane volume on lane utilization factor prediction with a physical mid-block lane drop, a mid-block left turn lane downstream of the intersection of interest and one lane drop sign. The steep slopes of the lines implies that the average lane volume has a large affect on the lane utilization factor. The gaps between the lines indicate the sensitivity of the short lane length factor on the lane utilization factor: wider gaps would show that the short lane factor was more sensitive.

The ranges of the 2TE model's application and average values of each factor are listed in Table 7. Analysts should be very cautious about extrapolating beyond the data range. Nevertheless, the data collected cover a wide range of potential future applications.

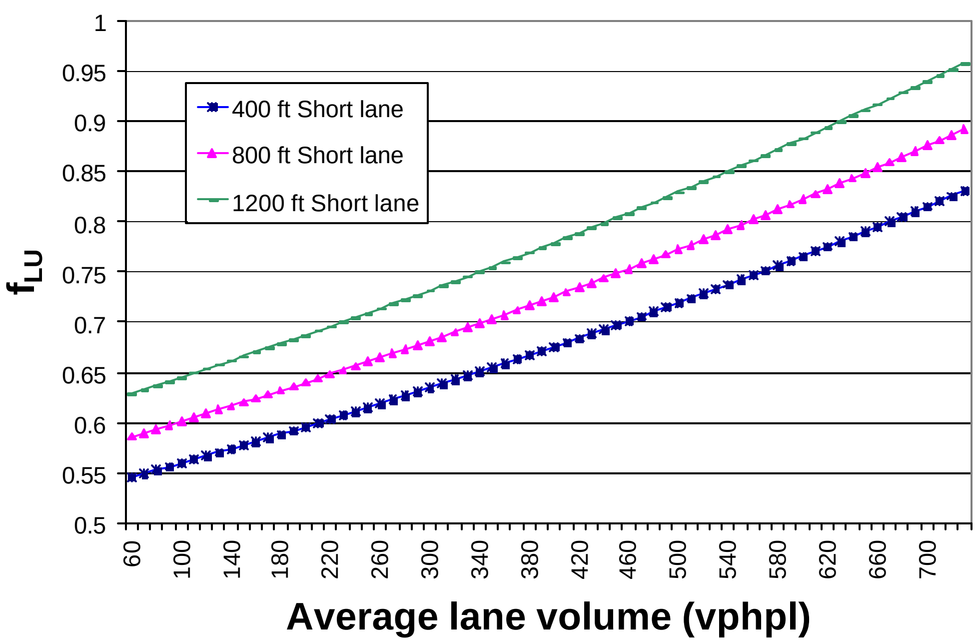


Figure 15. Effect of volume and short lane length on lane utilization factor (2TE)

Table 7. Ranges of the 2TE model application

Factor / Range	Average lane volume (vphpl)	Short lane length (ft)	Number of signs
Minimum	60	150	0

Maximum	730	1500	2
Average	242	748	1.1

Table 8 shows how each factor is sensitive to lane utilization for the 2TE model.

Table 8. Sensitivity of significant factors in 2TE model

Factor / Base f_{LU}*	Drp type (0 → 1)	MBAdn (0 → 1)	Short (per 100 ft)	Avg_lnvol (per 100 vphpl)	N_sign (per 1 sign)
0.698	0.562	0.810	0.686	0.656	0.776
Sensitivity	-19.6%	16.0%	1.8%	6.1%	11.1%

* Base f_{LU} was calculated with average values of each factor (except "N_sign" = 1) and 0 was applied for the indicator variables.

Two indicator variables, "Drp_type" and "MBAdn", have sensitivities of -19.6 % and 16.0 %, respectively. Every 100 ft of short lane length difference brings approximately 2% of lane utilization variation. The difference between the shortest lane to be dropped and the longest lane to be dropped for the 2TE group is over 1300 ft which can result ± 11.7 % difference in the dependent variable, f_{LU}. Average lane volume also shows high sensitivity of 6.1% per 100 vphpl. One sign affects lane utilization prediction by 11 %.

4.1.2. 2TS intersections

Scatter plots of explanatory variables in the 2TS intersection category did not indicate any need for data transformation. After the multicollinearity test, the screened candidate factors for predicting 2TS intersection lane utilization prediction were: Avg_lnvol, Short, RTpct, N_sign, N_mark, DdrwyupR, Drp_type, MBAup, MBAdn. Among the candidate factors, the final 2TS model included significant factors for lane drop type, short lane length and average lane volume. Similar to the 2TE model, the short lane length and average lane volume factors were positively related to the lane utilization factor and the model implies that physical mid-block lane drop lowers the f_{LU} by 0.123 compared to a lane discipline change lane drop type. The model yields an $R^2 = 0.75$ and a standard error of 0.0589. The model is:

$$\mathbf{f_{LU} = 0.588 - 0.123\ Drp\_type + 0.141\ ShortK + 0.121\ Avg\_lnvolK}$$

where	Drp_type	=	1	if physical lane drop
			0	if lane usage change lane drop
	ShortK	=	Short lane length (ft) ÷ 1000	
	Avg_lnvolK	=	Average lane volume (vphpl) ÷ 1000	

Figure 16 shows a plot of the observed and the predicted lane utilization factors and confirms that the model was predicting effectively. This model did not require a transformation.

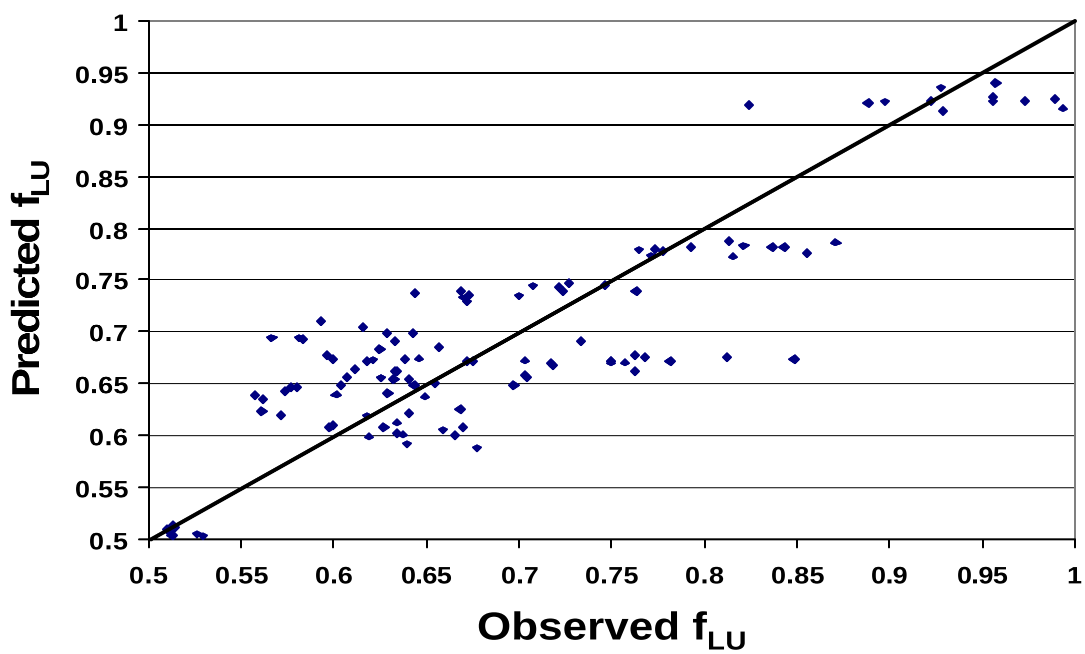


Figure 16. Observed and predicted lane utilization factor for 2TS intersection

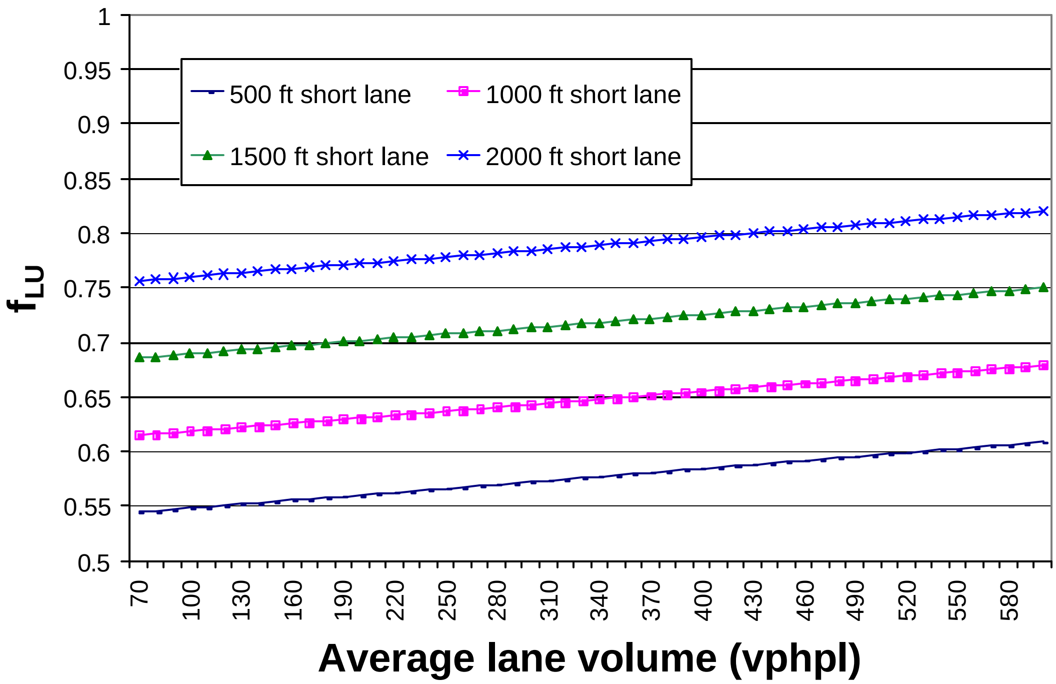


Figure 17. Effect of significant variables on lane utilization factor for 2TS intersections

Figure 17 shows the sensitivity of the lane utilization to the short lane length and average lane volume factors where there was a physical mid-block lane drop. The milder slopes compared to the 2TE slope in Figure 15 indicates that f_{LU} is less sensitive to the volume in this case. By contrast, the gaps between the lines are wider than those for the 2TE model indicating a stronger effect of short lane length. Table 8 summarizes the applicable data range of the 2TS model and average values of each factor. This 2TS model appears to be applicable for a wide range of volume and short lane lengths.

Table 9. Ranges of the 2TS model application

Factor / Range	Average lane volume (vphpl)	Short lane length (ft)
Minimum	66	148
Maximum	608	2061
Average	272	735

Table 10 represents the sensitivity of lane utilization to each factor for the 2TS model.

Table 10. Sensitivity of significant factors in 2TS model

Factor Base f_{LU}*	Drp type (0 → 1)	Short (per 100 ft)	Avg_lnvol (per 100 vphpl)
0.725	0.602	0.739	0.737
Sensitivity	-17.0%	2.0%	1.7%

The f_{LU} is as sensitive to short lane length is as in 2TE model while it is much less sensitive to average lane volume than in the 2TE model.

4.1.3. 2LS intersections

Data from two 2LS type intersections were excluded from the model development due to extremely low traffic volumes. From the original 15 factors, three factors (MBAup, DdrwyupL, and DdrwyupR) were not considered because left turn maneuvers within the left turn bay are not allowed and it was assumed that driveways upstream of the intersection would not affect the driver's lane selection. A multicollinearity test filtered six other factors and the candidate factors for the 2LS model development were: Avg_lnvol, Short, Taper, HVpct, Drp_type, and MBAdn. Applying the model selection procedure described previously, the final equation including MBAdn and Avg_lnvol was developed with an R^2 = 0.471 and a standard error of 0.0718. The model is:

$$\mathbf{f_{LU} = 0.616 + 0.105\ MBAdn + 0.864\ Avg\_lnvolK}$$

where	MBAdn	=	1	if downstream mid-block left-turn accessible
			0	otherwise
	Avg_lnvolK	=	Average lane volume (vphpl) ÷ 1000	

This 2LS model includes only two significant variables, MBAdn and Avg_lnvol. As with other models, these two variables have positive signs. However, the coefficient of the

Avg_lnvol variable has a tremendously large value, showing that f_{LU} is very sensitive to this factor. This might be caused by the narrow range of average lane volume, only 24 vphpl to 174 vphpl. Figures 18 shows plots of the observed and predicted lane utilization factors and Figure 19 shows the sensitivity of the average lane volume and the lane drop type. Traffic intensity, represented by the average lane volume, moderately affects the prediction of the lane utilization factor while approximately 0.1 of difference in f_{LU} depends on whether there is a downstream mid-block left turn lane or two way left turn lane.

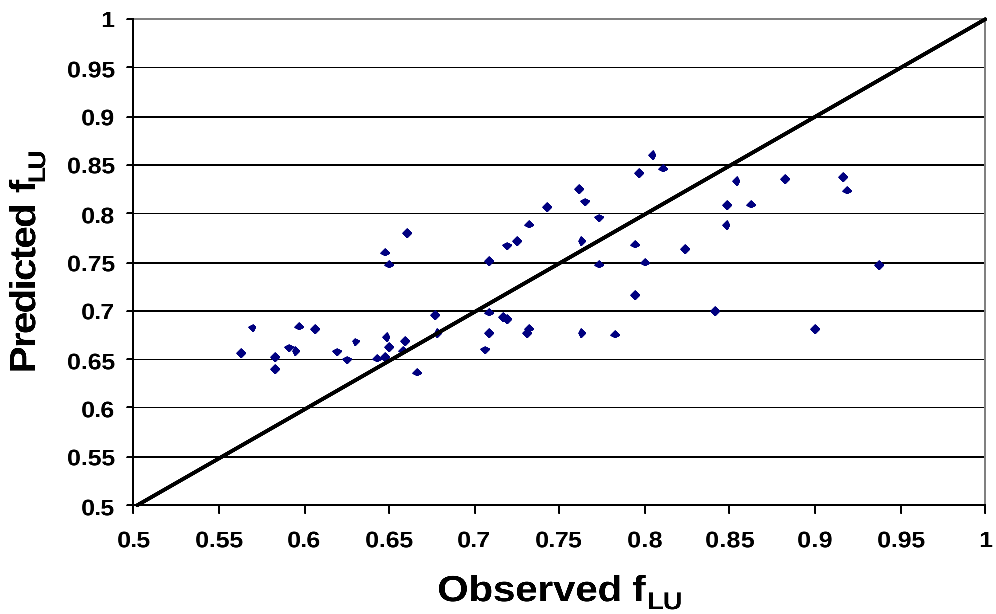


Figure 18. Observed and predicted lane utilization factor for 2LS

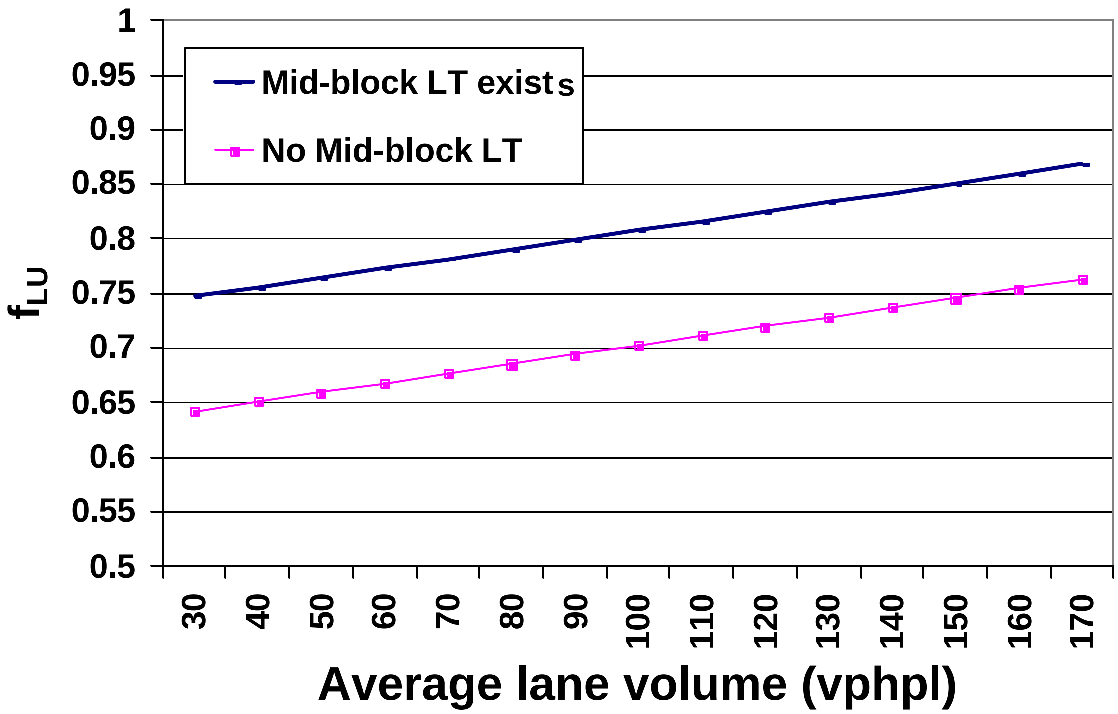


Figure 19. Effect of significant variables on lane utilization factor for 2LS intersections

Table 11 shows the sensitivity of f_{LU} to the significant factors for the 2LS model.

Table 11. Sensitivity of significant factors in 2LS model

Factor / Base f_{LU}*	MBAdn (0 → 1)	Avg_lnvol (per 100 vphpl)
0.685	0.790	0.772
Sensitivity	15.3%	12.6%

4.1.4. 2LR intersections

Intersections with dual left turns onto a freeway junction (2LR) have distinct characteristics. First, mid-lock left turn maneuvers upstream or downstream of the intersection do not matter. Second, there are no driveways downstream of the intersection. Third, a lane drop due to a change in lane discipline does not apply at this kind of intersection. Therefore, the factors MBAup, MBAdn, DdrwyupL, DdrwyupR, DdrwydnL, and DdrwydnR were not considered and the lane drop type was re-defined. For this intersection, a Type I lane drop is defined as "right side lane drop" and a Type II lane drop is a "left side lane drop". Data transformation was not required for this category. The multicollinearity test step condensed the list of candidate variables to: Avg_lnvol, Short, N_sign, Fstinfo, Drp_type and Taper. The final model is shown below, with an R^2 = 0.687 and a standard error of 0.0554. The model is:

$$\mathbf{f_{LU} = 0.323 + 0.176\ Drp\_type + 0.453\ Avg\_lnvolK + 0.237\ shortK + 0.397\ taperK}$$

where	Drp_type	=	1	if Left lane drops
			0	otherwise
	Avg_lnvolK	=	Average lane volume (vphpl) ÷ 1000	
	ShortK	=	Short lane length (ft) ÷ 1000	
	TaperK	=	Taper length (ft) ÷ 1000	

Since there are no negative factors in the model the intercept is lower than in the other models. The lane utilization factor rises by about 0.18 when there is a left lane drop at the ramp junction area. Most 2LR intersections in our sample had a right lane drop. Thus, driver expectation may cause higher f_{LU} value under left lane drop conditions. Consistently, average lane volume and short lane length are positive related to lane utilization. Taper length is a distinctive significant factor affecting lane utilization for this model. This might be due by the relatively high final speed at the ramp area: merging maneuvers in high speed conditions needs more space than under the conditions experienced on surface streets. A long taper might make the merging maneuver easier, inducing more usage of the short lane. Figure 20 shows the plots of the observed and the

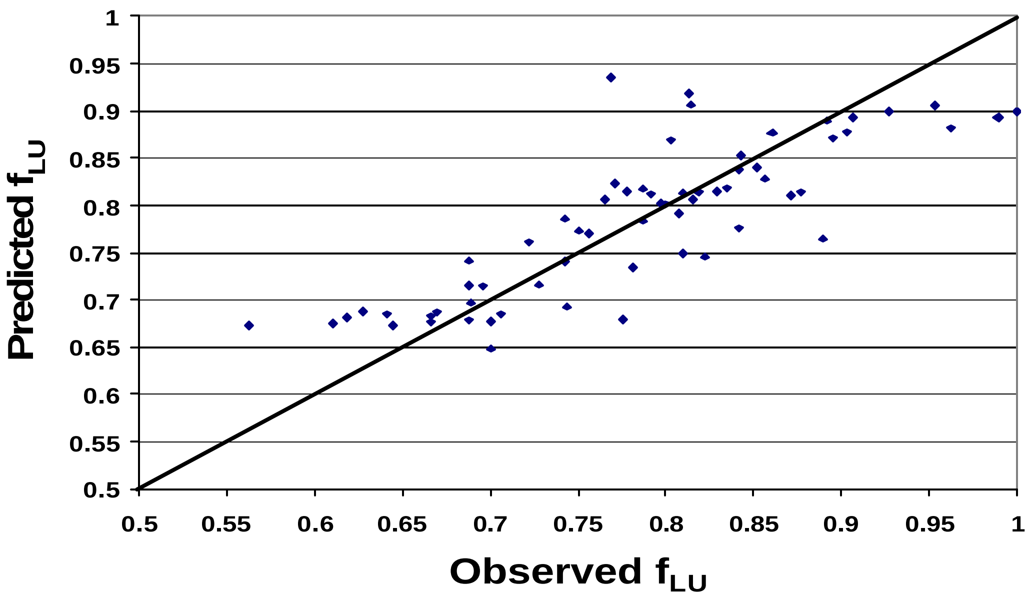


Figure 20. Observed and predicted lane utilization factor for 2LR intersection

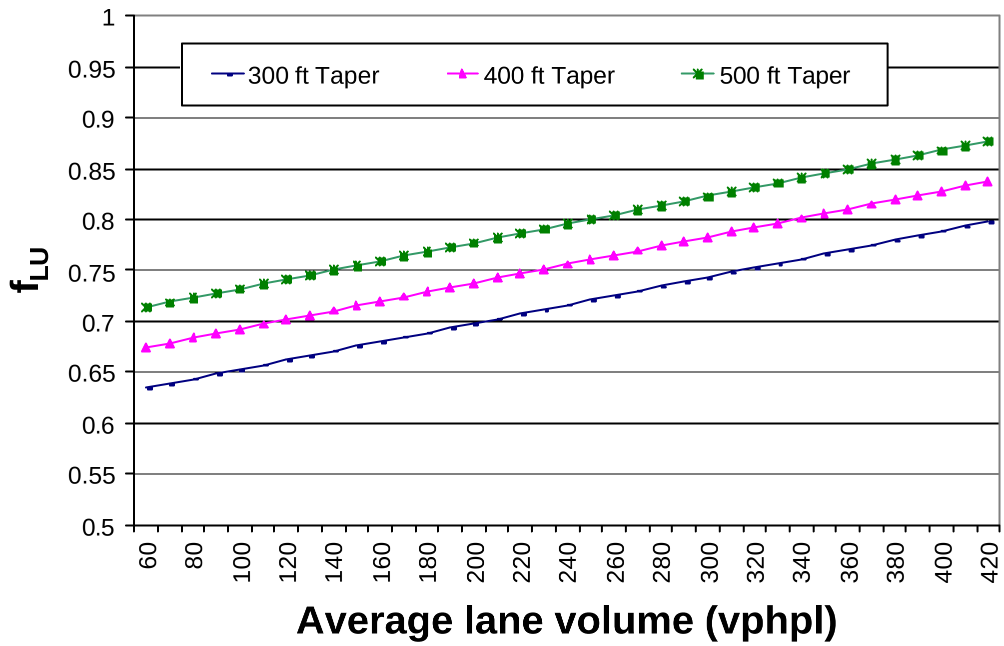


Figure 21. Effect of lane volume and taper length on lane utilization factor (2LR)

predicted lane utilization factors and Figure 21 represents the sensitivity of f_{LU} to the average lane volume and taper length factor with a right lane drop and 700 ft of short lane length. Average lane volume moderately affects the lane utilization factor while f_{LU} is quite sensitive to taper length. The model predicts an approximately 0.1 difference in lane utilization when there is 200 ft taper length difference.

Table 12 demonstrates the applicable data range for the 2LR model and average values of each factor. The applicable data range of short lane length is narrower than that in other models due to the more consistent design of the short lane length at freeway ramp junction areas.

Table 12. Ranges of the 2LR model application

Factor / Range	Average lane volume (vphpl)	Short lane length (ft)	Taper length (ft)
Minimum	58	548	260
Maximum	424	944	527
Average	227	725	401

Table 13 shows the sensitivities of f_{LU} to changes in each factor in the 2LR model.

Table 13. Sensitivity of significant factors in 2LR model

Factor / Base f_{LU}*	Drp type (0 → 1)	Short (per 100 ft)	Avg_Invol (per 100 vphpl)	Taper (per 100 ft)
0.756	0.932	0.732	0.711	0.716
Sensitivity	23.2%	3.1%	6.0%	5.2%

Lane drop type determines roughly 0.18 of the f_{LU}. The f_{LU} value was also quite sensitive to taper length, with a difference of 5.2 % per 100 ft of taper.

4.1.5. 3TE intersections

The candidate factors for 3TE intersection lane utilization prediction after the multicollinearity test were Avg_lnvol, Short, N_sign, Drp_type, MBAup, and MBAdn. The following model was developed with an $R^2 = 0.879$ and a standard error of 0.0345:

$$\mathbf{f_{LU} = 0.403 + 0.162\ MBAup + 0.281\ ShortK + 0.058\ Avg\_lnvolK}$$

where	MBAup	=	1	if upstream mid-block left-turn accessible
			0	otherwise
	ShortK	=	Short lane length (ft) ÷ 1000	
	Avg_lnvolK	=	Average lane volume (vphpl) ÷ 1000	

The significant factors in the 3TE model have all positive signs. The unique characteristic of the model is that the MBAup variable is significant instead of the MBAdn variable as in other models. Figure 22 shows the plots of the observed and predicted lane utilization factors. As shown in the Figure 23, f_{LU} is not very sensitive to the average lane volume for this particular intersection type. In contrast, the wide gaps between the lines indicate a high sensitivity of f_{LU} to short lane length. When a lane to be dropped is increased by 400 ft the lane utilization factor increases by approximately 0.12.

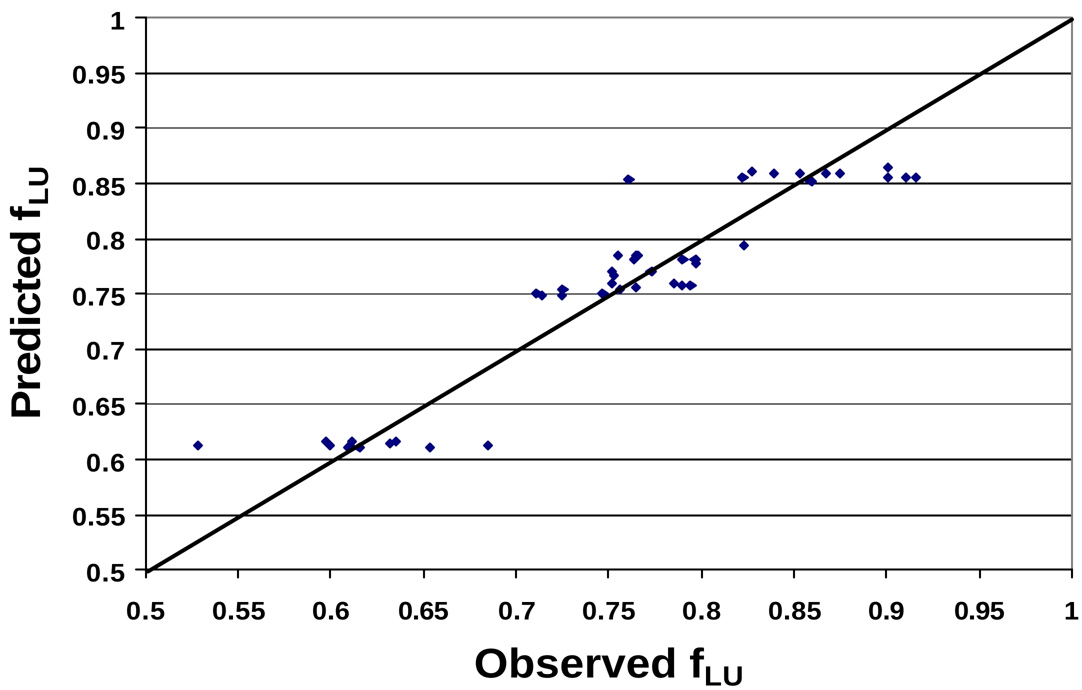


Figure 22. Observed and predicted lane utilization factor for 3TE intersections

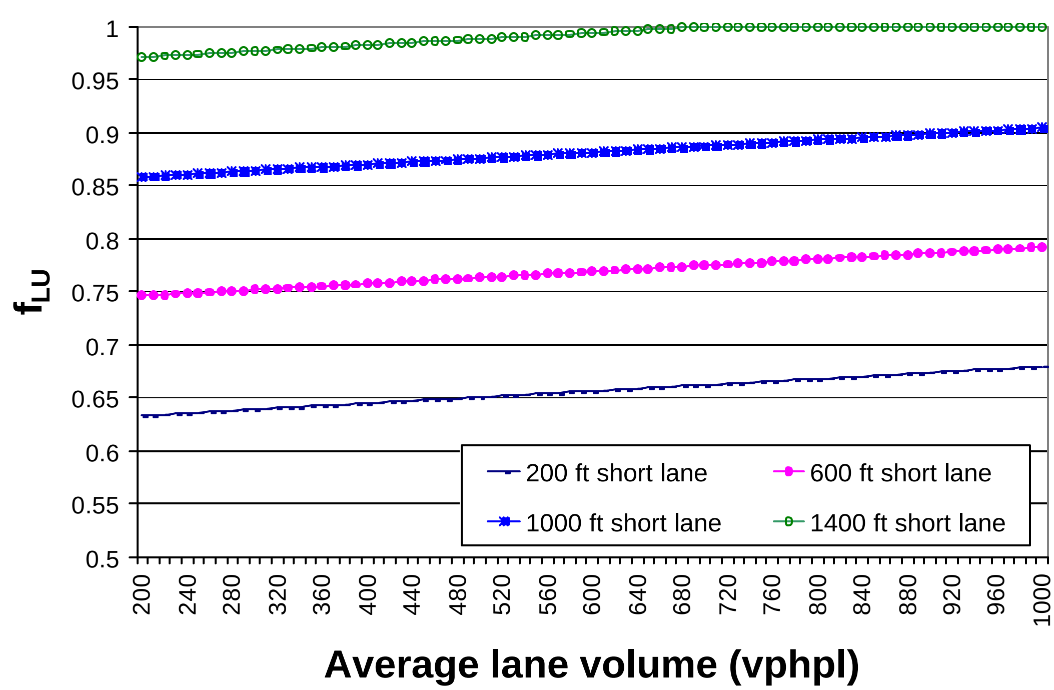


Figure 23. Effect of significant variables on lane utilization factor for 3TE

Table 14 shows the applicable data ranges for the 3TS model and average values of each factor. The average lane volume variable ranged from approximately 200 to 1000 vehicles per lane per hour and the short lane length ranged from 120 to 1500 ft.

Table 14. Ranges of the 3TE model application

Factor / Range	Average lane volume (vphpl)	Short lane length (ft)
Minimum	193	120
Maximum	1028	1529
Average	454	855

Table 15 shows the sensitivity of f_{LU} to the significant factors in the 3TE model.

Table 15. Sensitivity of significant factors in 3TE model

Factor / Base f_{LU}*	MBAup (0 → 1)	Short (per 100 ft)	Avg_lnvol (per 100 vphpl)
0.670	0.832	0.642	0.664
Sensitivity	24.2%	4.2%	0.9%

Compared to the other models, the sensitivity of the "Short" variable in this model is higher at 4.2 % per 100 ft. On the other hand, the Avg_lnvol sensitivity is the lowest in this model compared to the other models, at 0.9 % per 100 vphpl.

4.1.6. 3TS intersections

Data at two intersections were excluded from the model development due to a very heavy downstream right turn volume. The through and right turn shared lane operated as a de-facto downstream right-turn lane at these sites.

There was no need for data transformation. The candidate factors for 3TS intersection lane utilization, after the multicollinearity test, were: Avg_lnvol, Short, RTvol, HVpct, N_sign, N_mark, Drp_type, MBAup, and MBAdn. The final model to predict lane utilization factor for a 3TS intersection was:

$$\mathbf{f_{LU} = 0.682 + 0.079MBAdn + 0.115\ RTvolK + 0.017\ HVpct}$$

where	MBAdn	= 1	if the downstream mid-block left-turn was accessible
		0	otherwise
	RTvolK	=	Right turn volume in the TH/RT shared lane (vphpl) ÷ 1000
	HVpct	=	Percentage of heavy vehicle of the lane group

The 3TS model produced the lowest R^2 value of all six models at 0.45 and the standard error was 0.0429. This low R^2 might be caused by the low sample size (43 points). This model yielded distinctive significant variables, RTvol and HVpct, which affected lane utilization positively. In case of approach with a through/right shared lane, right turn vehicles are the captive user of the short lane. Heavy right turn volumes make for balanced lane utilization. For this reason, it makes sense that right turn volume was a significant factor. Heavy vehicle percentage is included in the model as well. Based on the field observation, trucks have tendency to use full lanes to avoid last minute lane changing due to their low acceleration rates. The higher proportion of heavy vehicles in the full lane sends more passenger vehicles to the short lane because passenger cars can accelerate faster and change lanes in front of the trucks.

Figure 24 represents the plots of the observed and the predicted lane utilization factors and Figure 25 shows the sensitivity of the lane utilization factor to the right turn volume and heavy vehicle factors when there were mid-block left turns downstream of the intersection. The slope and spacing of the lines indicate low sensitivity of f_{LU} to the variables. Table 16 summarizes the applicable data ranges of the 3TS model and average values of each factor. The data ranges of the 2LR model are fairly wide.

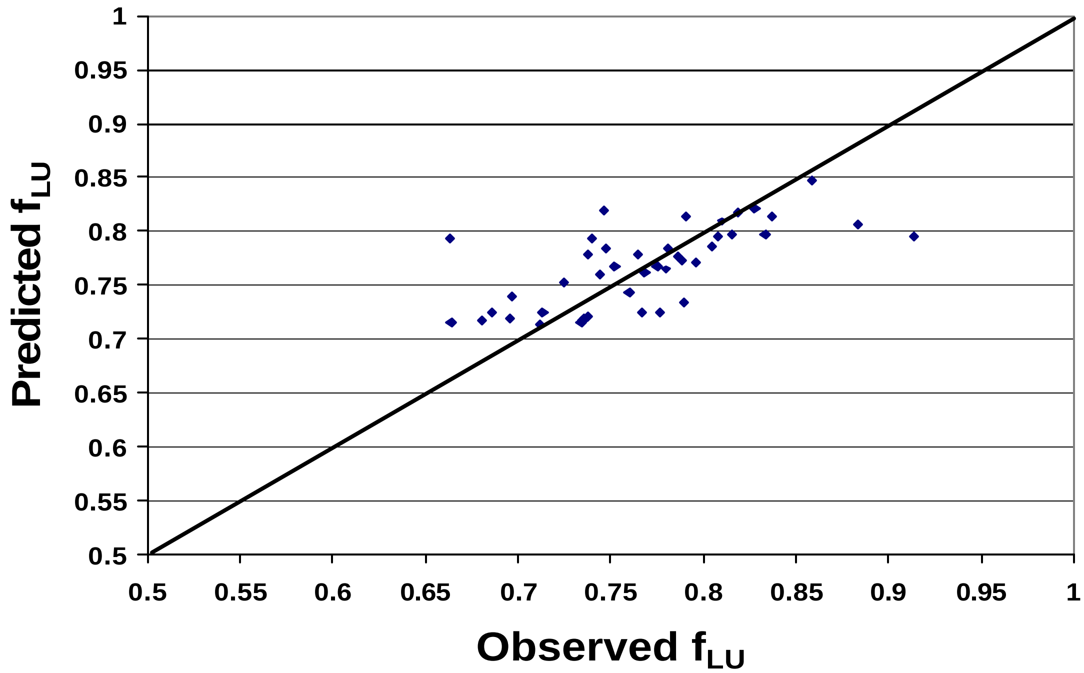


Figure 24. Observed and predicted lane utilization factor for 3TS intersection

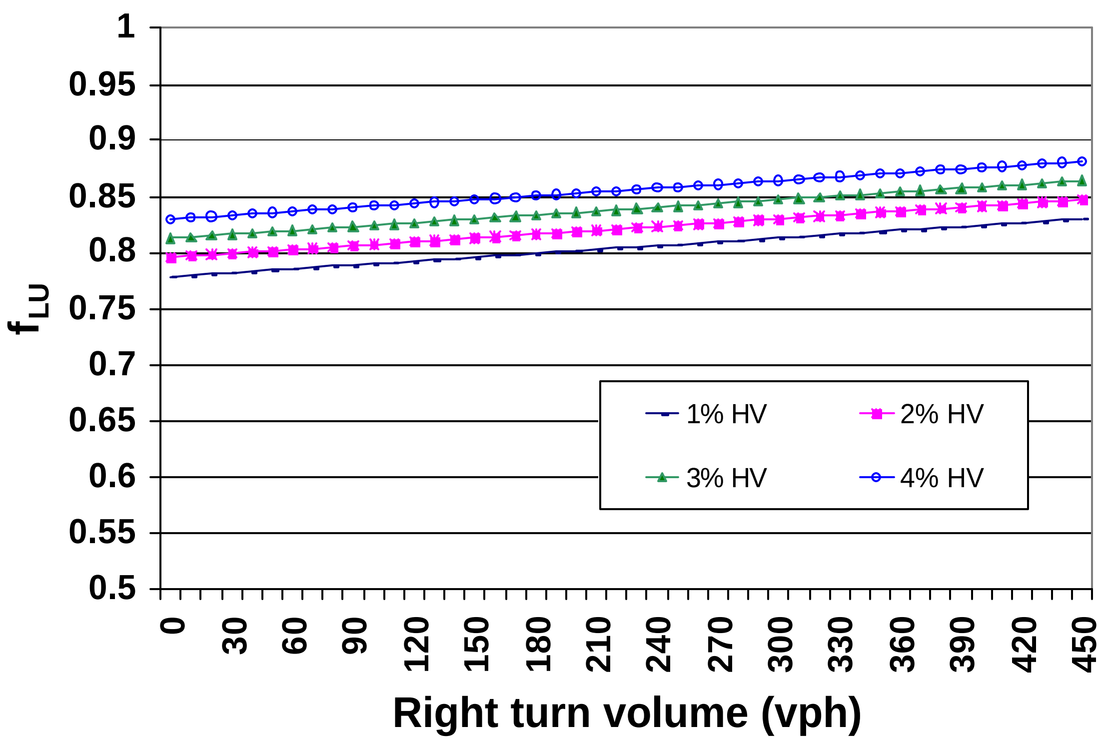


Figure 25. Effect of significant variables on lane utilization factor for 3TS intersection

Table 16. Ranges of the 3TS model application

Range \ Factor	Right turn volume (vph)	Heavy vehicle percentage (%)
Minimum	0	0.26
Maximum	453	4.68
Average	130	1.71

Table 17 shows the sensitivities of f_{LU} to the significant factors in the 3TS model.

Table 17. Sensitivity of significant factors in 3TS model

Base f_{LU}* \ Factor	MBAdn (0 → 1)	RTvol (per 100 vphpl)	HVpct (per 1%)
0.726	0.806	0.715	0.744
Sensitivity	10.9%	1.6%	2.4%

The overall sensitivities of f_{LU} to the significant variables in this model are not very high. Generally the sensitivity of f_{LU} to 0 → 1 variables was quite high for other models, ranging from 16.0 % to 24.2 %. However, changing the MBAdn variable from 0 to 1 only meant a change of 10.9% in f_{LU} for the 3TS model. In addition, the 1.6 % and 2.4 % sensitivities for the RTvol and HVpct variables, respectively, were quite low considering the data ranges.

4.1.7. Summary of lane utilization prediction models

Average lane volume was an important explanatory variable in five of the six prediction models. The coefficient of the average lane volume variable was consistently positive. This means that a higher average lane volume (i.e., more congestion) attracts more users to the short lane. Short lane length was revealed to be significant in four prediction models. Like the average lane volume variable, all short lane length variables in the four models have a positive coefficient, which means that longer lanes to be dropped yield more equal lane volume distributions. The existence of mid-block left-turn bays or TWLTLs downstream or upstream of the intersection increased the short lane utilization in four intersection types. This implies that drivers may be avoiding the impedance caused by decelerating left turning vehicles ahead. It was also found that the lane drops associated with lane discipline changes downstream short lane turn into an exclusive right turn lane attract more drivers to the short lane than the physical mid-block lane drops for the two-to-one-lane-drop. Taper length was a significant factor affecting lane utilization for dual left turn lane groups onto freeway ramps. The final speed at the freeway ramp is generally high, causing a need for more space to merge. A long taper might make the merging maneuver easier, inducing more usage of the short lane. The variables for right turn volume and heavy vehicle percentage are only utilized in the 3TS model. Right turn vehicles are captive users of the shared right-turn/through lane. Consequently, more right turn volume in the shared lane appears to increase lane utilization for that intersection type.

Table 18 summarizes the six models and shows the R^2 of each model. The six models were simplified for easy usage by adopting α, which is a single variable representing the indicator variables.

Table 18. Lane utilization factor prediction models for lane drop intersection by intersection type

Lane drop geometry	Lane discipline	Drop type	MBAdn	MBAup	a	R^2	Lane utilization prediction equation
Two through lanes to one through lane	With exclusive right turn lane (2TE)	Mid-block taper	1*	All	0.5435	0.73	$f_{LU} = a\, e^{(0.1782 ShortK + 0.6273 Avg\_lnvolK - 0.1047 N\_sign)}$
			0*		0.4688		
		Terminates in exclusive right turn lane	1*	All	0.6760		
			0*		0.5832		
	Shared through and right turn lane (2TS)	Mid-block taper	All	All	0.4651	0.75	$f_{LU} = a + 0.1414\ ShortK + 0.1210\ Avg\_lnvolK$
		Terminates in exclusive right turn lane			0.5882		
Two left turn lanes to one lane	At intersection (2LS)	All	1*	All	0.7210	0.47	$f_{LU} = a + 0.8636\ Avg\_lnvolK$
			0*		0.6161		
	Onto ramp (2LR)	Left lane drop	All	All	0.4984	0.69	$f_{LU} = a + 0.4527\ Avg\_lnvolK + 0.2367\ ShortK + 0.3966\ TaperK$
		Right lane drop			0.3228		
Three lanes to two lane	With exclusive right turn lane (3TE)	All	All	1*	0.5654	0.88	$f_{LU} = a + 0.2814\ ShortK + 0.0576\ Avg\_lnvolK$
				0*	0.4033		
	Shared through and right turn lane (3TS)	All	1*	All	0.7614	0.45	$f_{LU} = a + 0.1145\ RTvolK + 0.0171\ HVpct$
			0*		0.6823		

MBAdn: Left turns can access downstream driveways ***MBAup***: Left turns can access upstream driveways
ShortK: Short lane length (ft) ÷ 1000 ***Avg_lnvolK***: Average lane volume (vphpl) ÷ 1000 ***N_sign***: Number of signs informing drivers of lane drop
TaperK: Taper length (ft) ÷ 1000 ***RTvolK***: Right-turn volume (vph) ÷ 1000 ***HVpct***: Heavy vehicle percentage in approach volume

* 1 means a mid-block left turn bay or Two-Way-Left-Turn Lane (TWLTL) exists, zero if neither exists.

4.2. Delay comparison between HCM default and model predicted value

The HCM signalized intersection procedure specifies that the adjusted lane group saturation flow rate is directly proportional to the lane utilization factor. For instance, a lane utilization factor of 0.75 for a lane group with two-through lanes leads to approximately a 20 percent loss in saturation flow (and consequently, capacity) compared to the default HCM value of 0.95. The use of a low value of lane utilization will therefore yield a lower capacity, higher delays and a worsening level-of-service. To see whether this logic matches the conditions observed in the field, field-measured delays and the delay predicted by the HCM method using the model-predicted lane utilization factors were compared.

Signalized intersection delay was estimated in the field based on the arrival rate, saturation flow rate and queue length at the beginning of green. Figure 26 shows a delay profile and a set of queue accumulation polygons (QAPs) at a signalized intersection. For our intersection sample, the total areas of the QAPs during 15-minute periods were calculated from field observations and the values were divided by demand volumes during the periods to obtain average delays. The average delays were estimated by lane and then the lane group average delays were calculated. Using the HCM method, the saturation flow rate of the full lane was applied along with default saturation flow adjustment factors except for the lane utilization factor, which was estimated by the prediction models developed earlier in the chapter.

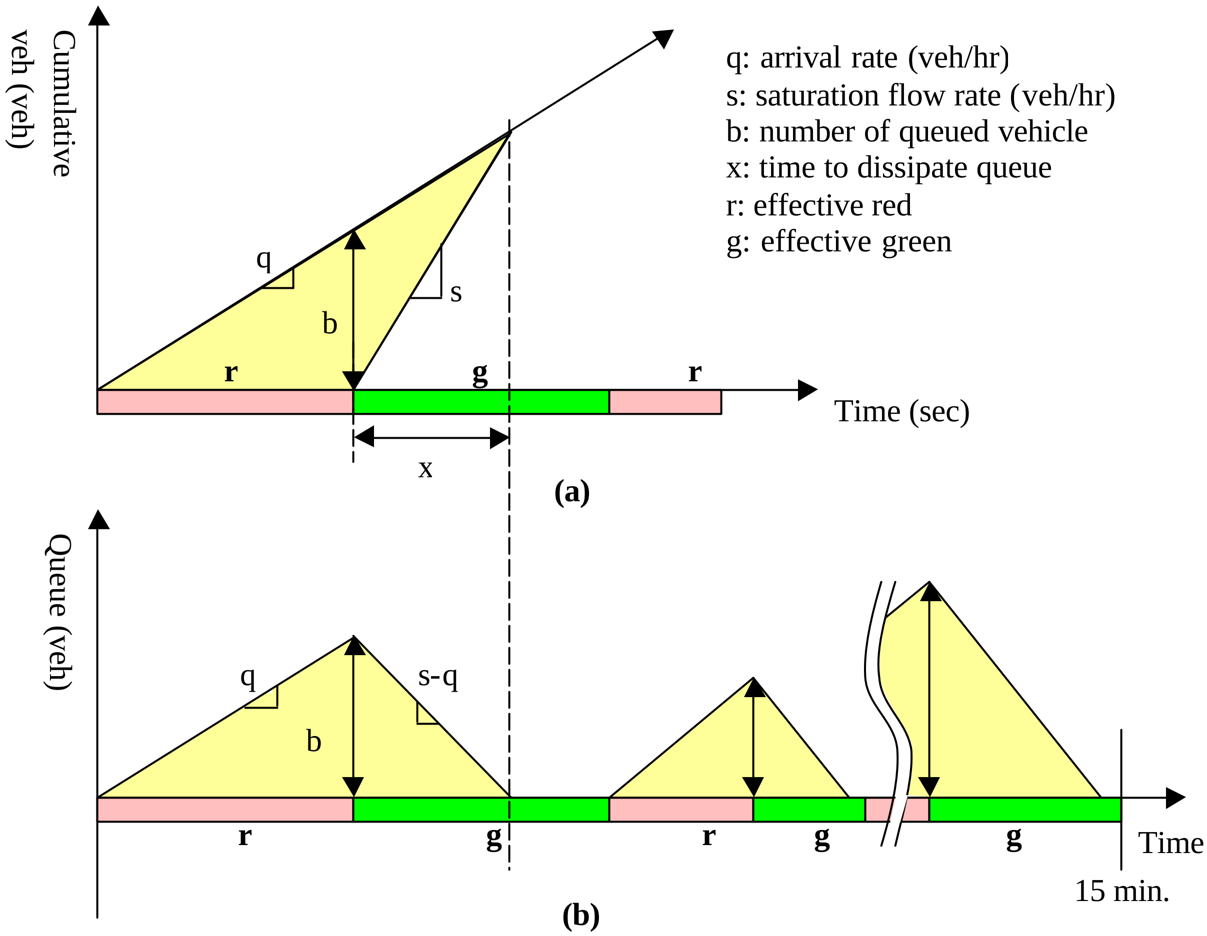


Figure 26. Intersection delay: (a) delay profile (b) queue accumulation polygons

Figure 27 shows the comparison between the field and HCM when applied to one typical intersection approach for each of the six intersection types considered in this study. The delay discrepancies ranged from 8 seconds per vehicle to 30 seconds per vehicle. Since the results from each site are independent of the others, a paired t-test was conducted for the collection of sites, yielding an overall mean and standard deviation of delay difference of 17.6 s and 8.83 s, respectively. The null hypothesis was that the average difference between the observed delay and HCM delay is zero. The calculated t-

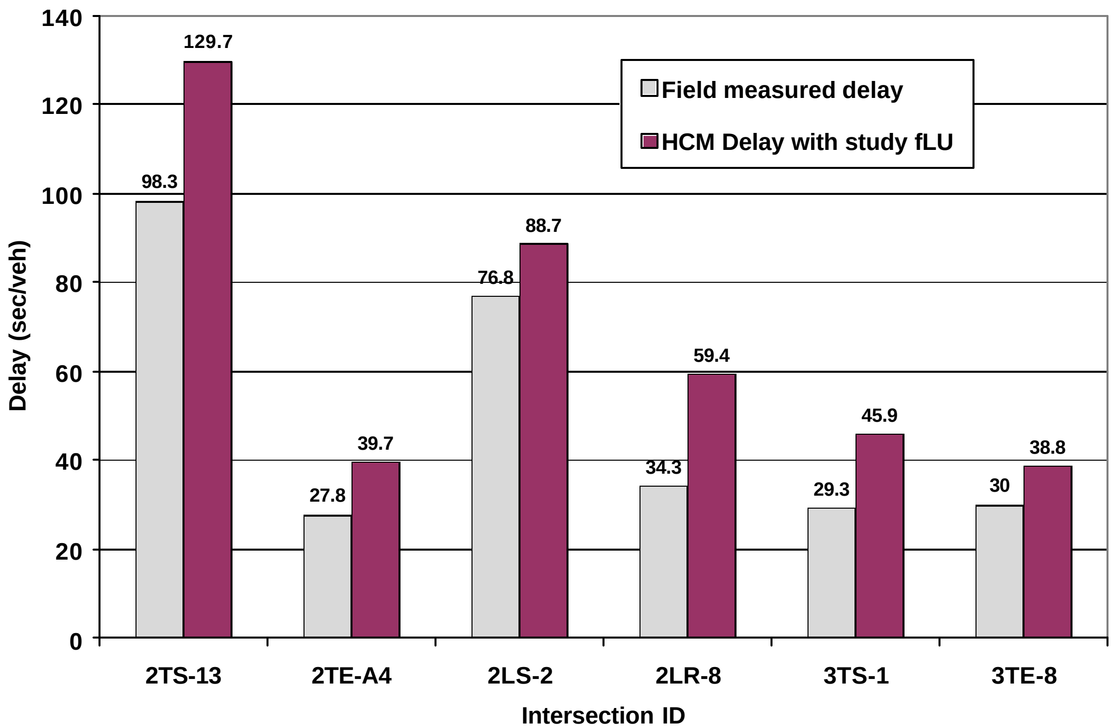


Figure 27. Field and HCM delay comparisons with model predicted lane utilization factors

value of 4.88 ($p < 0.0025$), led us to conclude that the HCM delay is significantly higher than the field-measured delay.

The result is shown in Figure 27 and implies that lane utilization effects on saturation flow in the HCM need to be re-assessed in the context of estimating capacity and LOS for signalized intersection lane groups. One possible explanation lies in the nature of actuated intersection control that was prevalent at the study sites. Since short lane utilization was found to increase with average lane volume, low lane utilization factors will tend to occur under lighter traffic conditions. In these cases, unless the volume in the full lane(s) triggers an extension of the green time, the approach capacity will likely be unaffected by the uneven lane utilization. Furthermore, green time extensions on an approach may be triggered by the heavier traffic demand on the opposing approach which calls the same phase (Phase 2+6 in NEMA convention), again masking any effect the uneven lane distribution on the subject approach may have. In general, our study suggests that while lane utilization factors are much lower than the HCM, their impact on delay appears to be much less than what the HCM predicts. Continuing research is focused on developing a better understanding of the relationship of signal control parameters and lane utilization using both field data and simulation models.

CHAPTER 5. COLLISION DATA ANALYSIS RESULTS

Safety near lane drops is a critical issue. Even if a short lane does not have a large effect on delay and capacity, a highway agency may not want to use it due to an increase in the expected number of collisions. To treat this issue, we examined the pattern of collision rates along with the short lane length for all intersection types previously analyzed and examined the relationship between the taper length and collisions at ramp areas. We compared the numbers of collisions recorded on the study sections by lane drop type. In addition, we looked at the distributions of collision types and severities occurring at or near lane drops. This chapter describes those analyses.

5.1. Collision rates versus short lane length

After calculating the collision rates for each section and site, plots of collision rate vs. short lane length were produced for each intersection category. Before the analysis, the data were reduced to improve data quality. Some collision data were excluded due to the lack of ADT data. We also examined and discarded outlying data points when those had a tendency to affect the findings and when they were unrelated to lane drops. An excessive number of collisions at stop-controlled intersections downstream or upstream of the intersection of interest was the most common type of outlier.

Figures 28 and 29 show the plots of collision rate versus short lane length of each intersection category for road sections upstream and downstream of the intersection of interest, respectively. Approximate trend lines and their equations are embedded in the plots. We hypothesized that collision rates would be negatively related to the short lane length for downstream sections. That proved to be true. However, for upstream sections,

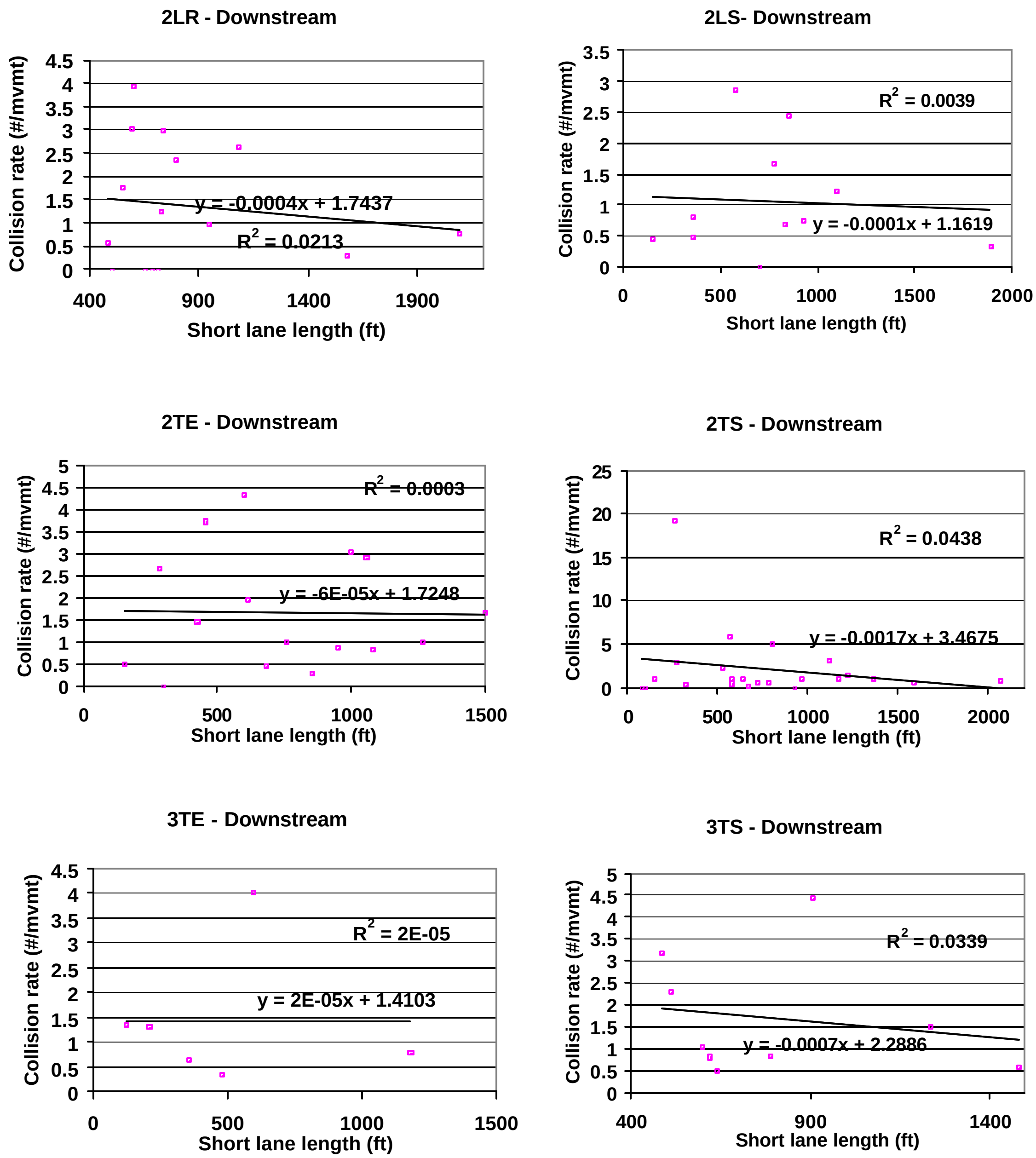


Figure 28. Short lane length vs. collision rate on roadway sections downstream of the intersections of interest.

half of the six intersection categories have positive relationships between collision rates and short lane length. Overall, it appears that shorter lane drops mean generally higher collision rates, but the effect of short lane length is not strong in this sample of intersections.

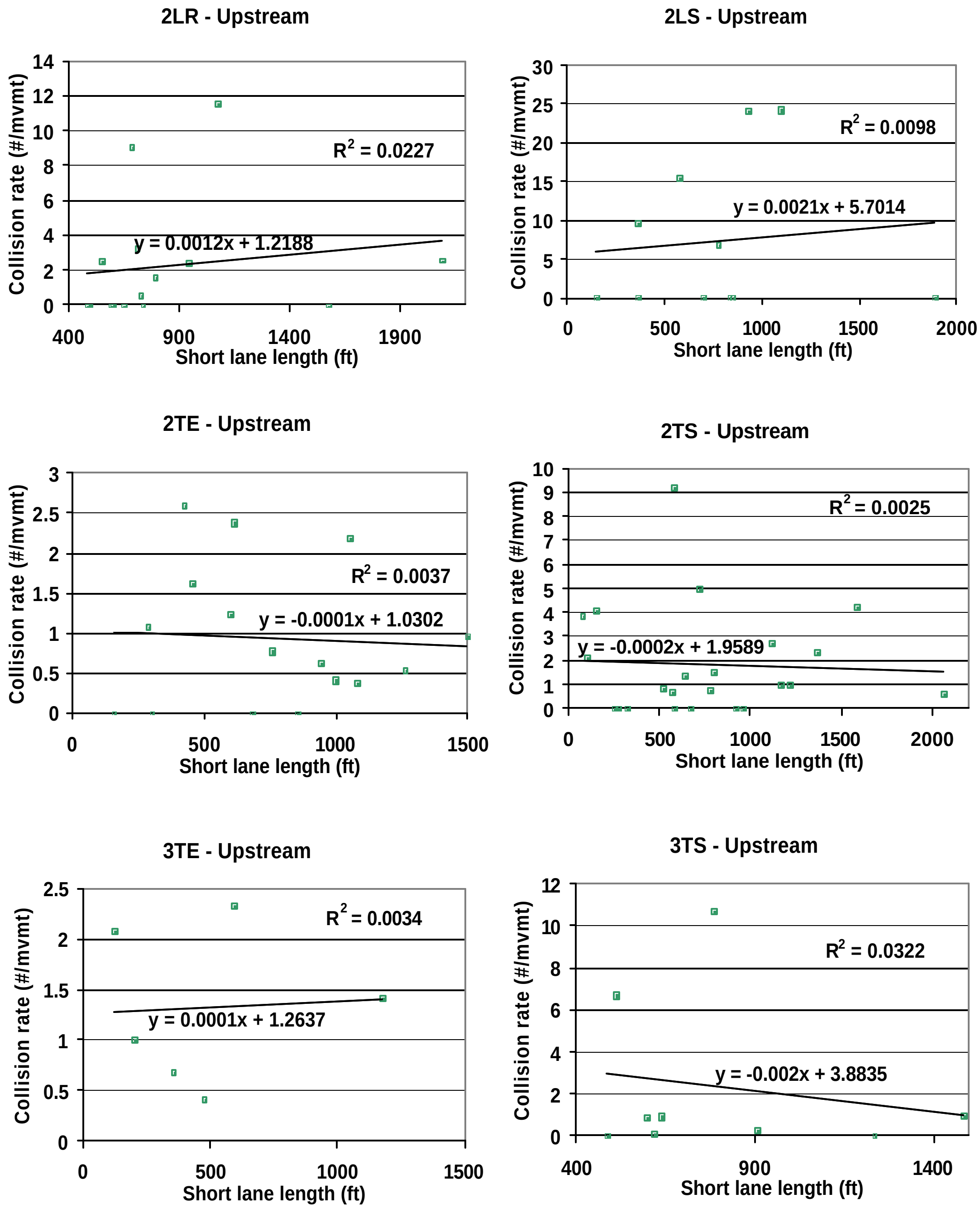


Figure 29. Short lane length vs. collision rate on roadway sections upstream of the intersections of interest.

5.2. Collision rates versus taper length for freeway on-ramps

In the previous chapter we found that the taper length on an on-ramp affects lane utilization significantly. Consequently, we also examined the relationship between collision rate and taper length for on-ramps. The collision rates on the on-ramp downstream of the intersection of interest (with the dual left turn lane, the 2LR case) were plotted against taper lengths as shown in Figure 30. The plot shows a negative relationship: collision rates decrease as taper lengths increase. This result suggests that long tapers should be provided at on-ramps where possible when there is a lane drop.

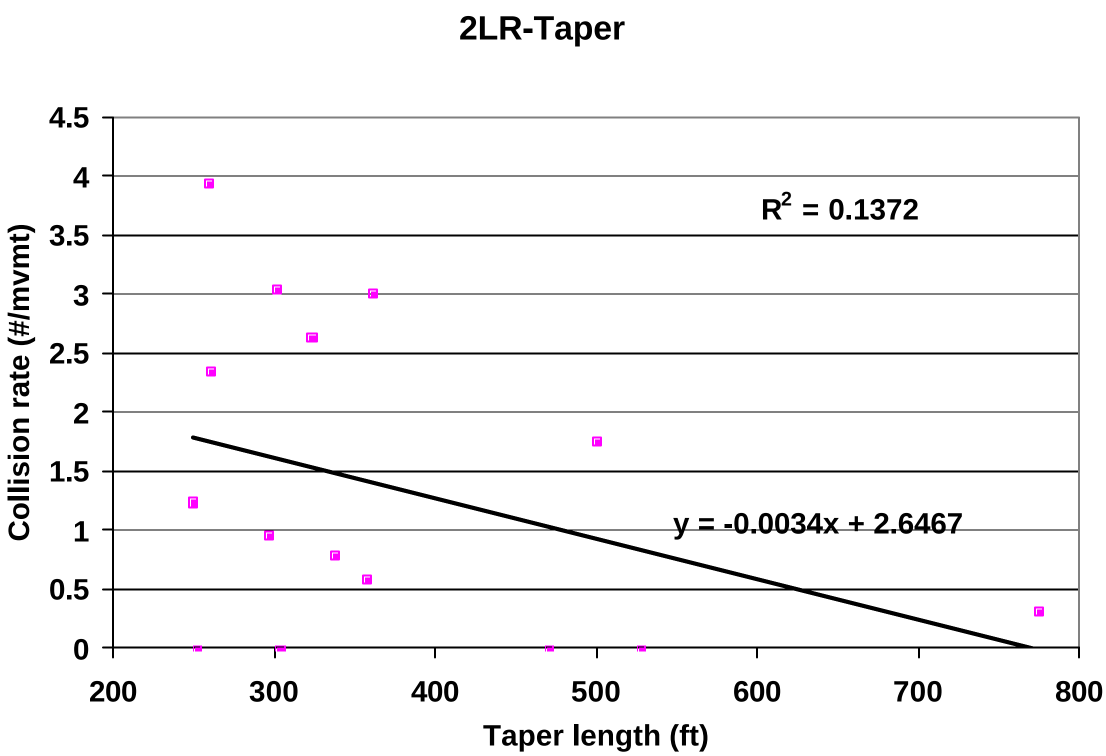


Figure 30. Taper length vs. collision rate for on-ramp lane drops.

5.3. Collision rates vs. drop type

This question involves two types of lane drops--Type 1and Type 2--which are the physical lane drop at a mid-block and the change of lane discipline from through lane to right turn only lane, respectively. The collision rates between the two types of lane drop were compared statistically using T-tests. The data from 2LR sites were excluded from this comparison because the definition of lane drop type in the 2LR category is different. It was assumed that the population distribution of the collision rates is normal. The null hypothesis was that the difference between the collision rates for the two lane drop types is zero. The standardized variable T and the degrees of freedom were calculated with the following equations:

$$T = \frac{\overline{X} - \overline{Y}}{\sqrt{\frac{S_1^2}{m} + \frac{S_2^2}{n}}}$$

where $\overline{X}$ = Mean of collision rates for Type 1 lane drop
$\overline{Y}$ = Mean of collision rates for Type 2 lane drop
$S^2{}_1$ = Variance of collision rates for Type 1 lane drop
$S^2{}_2$ = Variance of collision rates for Type 2 lane drop
m = number of Type 1 lane drop intersections
n = number of Type 2 lane drop intersections

$$n = \frac{\left(\frac{S_1^2}{m} + \frac{S_2^2}{n}\right)^2}{\frac{\left(S_1^2/m\right)^2}{m-1} + \frac{\left(S_2^2/n\right)^2}{n-1}}$$

where n = degree of freedom

The calculated T-value and the T-table value with α = 0.025 were compared; the null hypothesis would be rejected if the calculated T-value was greater than the T-table value.

Table 19 shows the comparison results and some statistical parameters such as mean, standard deviation, number of samples and degree of freedom. The mean rates are similar to each other both upstream and downstream of the intersection of interest. T-test results confirm that there was no significant difference in collision rates between Type 1 and Type 2 intersections upstream or downstream.

Table 19. Statistic parameters and comparison results between Type 1 and Type 2 lane drop.

Section		**Upstream**	**Downstream**
Type 1	Mean (collision / mvmt)	2.26	2.06
	Std. dev. (collision / mvmt)	4.75	3.54
	# of sample	31	31
Type 2	Mean (collision / mvmt)	2.78	1.46
	Std. dev. (collision / mvmt)	5.02	1.29
	# of sample	35	35
Degrees of freedom		62.3	35.6
t-test statistic table value		2.00	2.02
t-table value with α = 0.025		0.43	-0.88
Conclusion		No significant difference	No significant difference

5.4. Percentage of sideswipe and rear-end collisions

Another safety question we examined was whether the percentages of sideswipe and rear-end collisions changed due to the lane drops. Lane drops require lane changing, so sideswipe and/or rear end collisions might increase due to that feature. Figure 31 shows the distribution of sideswipe, rear-end and other types of collisions for each intersection category we studied. Collisions at the signalized intersections were excluded from this analysis; Figure 31 just contains collisions upstream or downstream of the signal. Near intersections at ramp junction areas (category 2LR) sideswipe collisions were a relatively high portion (nearly 30%) of all collisions while rear end collisions were a relatively small portion (just over 20 %). Rear end collisions were the major type of collision at the 3TE intersections. The proportion of collisions that were sideswipe was relatively low (below 10 percent) for the 2TE and 2TS intersection types, perhaps showing that lane changes are not so difficult without complications from left turns and third lanes.

5.5. Percentage of injury-involved collisions

Collisions in our sample were categorized as property damage only (PDO) or injury-involved collisions. Figure 32 shows the distribution of the two categories for each intersection type. About 30 % of all collisions involved an injury. The left turn lane drop intersections had the lowest percentages of injury collisions, while intersections with two through lanes had the highest percentages.

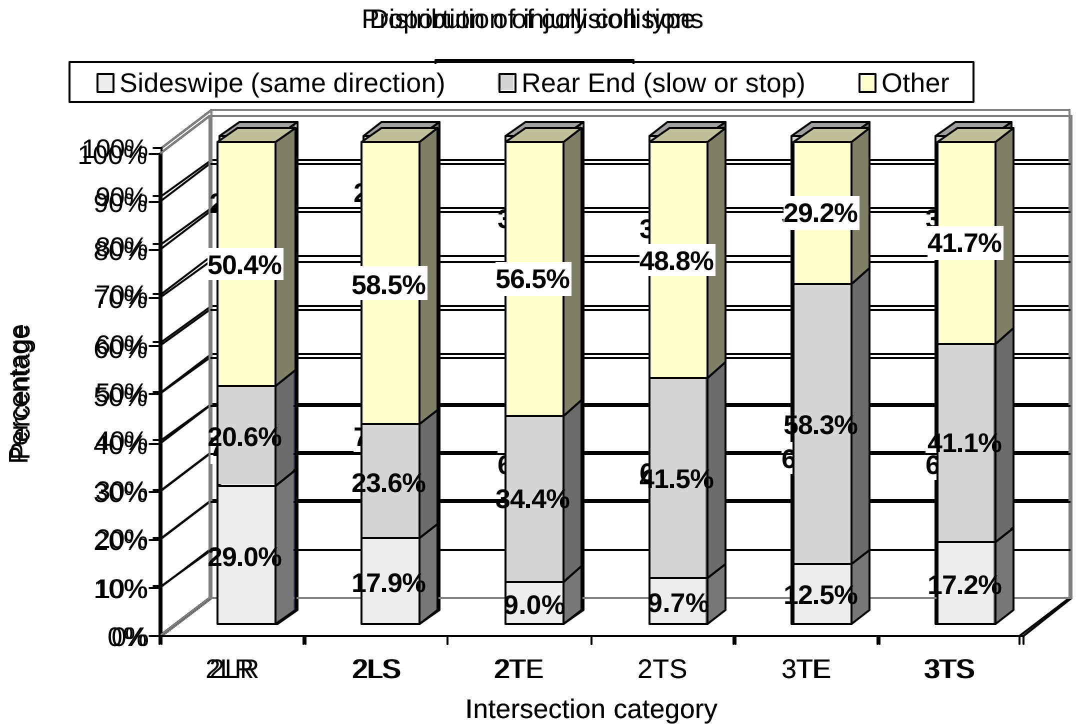


Figure 31. Distribution of collision types.

CHPATER 6. CONCLUSIONS AND RECOMMENDATIONS

This project was aimed at the development of lane utilization prediction models for signalized intersections with lane drops, and investigated the pattern of collision rates associated with the lane drop. The lane utilization prediction models, summarized in Table 18, were developed based on extensive field data collection. The developed models illustrate how conditions at and near intersections affect lane utilization of the subject approaches.

For the purpose of information dissemination, a portion of the results from this research were submitted to the Transportation Research Board for consideration for publication and presentation at the 84th annual meeting of TRB. The findings of this project might also be presented at a future NCDOT traffic engineering conference and at an NCSITE annual meeting in the future. In addition, the results of this study along with further investigation might be published as a part of the doctoral dissertation of the lead author.

The key findings and conclusions related to the operational aspects of this study include:

- Traffic intensity has a positive relationship with lane utilization. When traffic volumes increase there is a greater tendency for drivers to use the short lane.
- Lane utilization at most types of intersections is affected by the length of the short lane. A long distance to the lane drop leads to higher lane utilization.
- The existence of a TWLTL or a mid-block left turn bay tends to increase the lane utilization factor.

- Lane drops due to a lane usage change (Type 2 lane drop) have more equal lane volume distribution than the mid-block taper lane drop (Type 1 lane drop).
- Taper length significantly affects lane utilization at freeway ramp junction areas.

Collision rates were also examined to identify whether geometric characteristics such as short lane length, taper length, and lane drop type affect collision rates positively or negatively. The findings of collision data analysis include:

- Collision rates downstream of intersections with lane drops have a tendency to decline as the distance to the lane drop gets longer although the sample size did not allow for a robust statistical analysis of its significance.
- Collision rates upstream of intersections with lane drops did not show a consistent trend with short lane length. This might be because traffic signals dilute the effects of the short lanes on collision rates in these cases.
- The collision rates for on-ramps downstream of 2LR (dual left turn onto freeway ramp) intersections tend to decrease as the taper lengths increased.
- The collision rates of Type 1 and Type 2 lane drops are not statistically different upstream or downstream of the intersections of interest.

Based on the analysis results and more than 150 hours of field observations, recommendations for short lane design and future research include:

- Apply lane drops on approaches that have higher demands than the opposite direction when the approach involving lane drop and the opposite approach operate in same signal phase. This might weaken the effect of biased lane utilization on the approach.

- A Type 2 lane drop is recommended where a high demand for right turn movements at the downstream intersection exists.
- Long tapers at freeway ramp junction areas are highly recommended to reduce collision rates and induce more even lane utilizations at the signals.
- Careful application of a low lane utilization factor is advised when evaluating the capacity, delay or level of service of a signalized intersection following the HCM methods. Applying a low lane utilization factor directly in an HCM signalized intersection evaluation might result in underestimated capacity and overestimated delay.
- Further research is recommended on how low lane utilization affects signalized intersection capacity and delay.

Further research is also recommended into the safety effects of lane drops. In particular, more sophisticated modeling techniques should be used on a larger database to produce an estimate of the number of collisions saved when designer choose longer lanes and longer tapers.

Many of the results derived from this study are consistent with previous research. The models developed should provide designers and traffic engineers with concrete methods to improve lane utilization estimates when lane drops are contemplated.

7. REFERENCES

1. *Highway Capacity Manual 2000.* TRB, National Research Council, Washington, D.C., 2000.

2. *A Policy on Geometric Design of Highways and Streets.* Fourth Edition, AASHTO, Washington, D.C., 2001.

3. Teply, S., D. I. Allingham, D. B. Richardson, and B.W. Stephenson. *Canadian Capacity Guide for Signalized Intersections.* Second Edition, ITE District 7, Canada, June 1995.

4. Akcelik, R. SIDRA-2 Does It Lane By Lane. *Proceedings of 12th ARRB Conference*, Vol. 12, Part 4, Victoria, Australia, August 1984, pp.137–149.

5. Perterson, B.E., A. Hansson, and K.L. Bang. Swedish Capacity Manual. In *Transportation Research Record 667*. TRB, National Research Council, Washington, D.C., 1978, pp. 1–28.

6. Stenberg, L. and T. Bergh. *CAPCAL-2 Model Description of Intersection with Signal Control.* Swedish National Road Administration, Sweden, 1995.

7. Rouphail, N. and B. Nevers. Saturation Flow Estimation Using Traffic Subgroups. In *Transportation Research Record 1776,* TRB, National Research Council, Washington, D.C., 2001, pp. 114–122.

8. Nevers, B.L. A Saturation Flow and Lane Distribution Model for Signalized Intersections. Thesis for North Carolina State University, January 2001.

9. Hummer, J.E. Operational Effects of New 'Double Wide' Intersection Design on Suburban Arterials, Paper 00-1535, Transportation Research Board 79th Annual Meeting CD-ROM, Washington, D.C., 2000.

10. Ledbetter, J.D. and J.E. Hummer. A Preliminary Study of the Congestion Relief Potential of Auxiliary Through Lanes at Signalized Intersections, ITE Compendium of Technical Papers, 1991, pp. 38-42.

11. Qiong, S. Minimum Merging Section Lengths for Triple Left-turn Lanes with Downstream Lane Reductions, In *ITE Journal*, Volume 71, No. 3, March 2001, pp.40 – 45.

12. Leisch, J.E. Capacity Analysis Techniques for Design of Signalized Intersections, In *Public Roads*, Volume 34, No. 9, 1967, August, pp. 171 – 210.

13. Guell, L.D. Additional Through Lanes at Signalized Intersections, In *Journal of Transportation Engineering*, Volume 109, No. 4, July 1993, pp. 499-505.

14. McCoy, T.P. and J.R. Tobin. Use of Additional Through Lanes at Signalized Intersections. In *Transportation Research Record 869*, TRB, National Research Council, Washington, D.C., 1982, pp. 1 – 5.

15. Hurley, J.W. Utilization of Double Left-turn Lanes with Downstream Lane Reductions, In *Journal of Transportation Engineering*, Volume 124, Number 3, 1998, May, pp. 235-239.

16. Hurley, J.W. Utilization of Auxiliary Through Lanes at Signalized Intersections with Downstream Lane Reductions, In *Transportation Research Record 1572*, TRB, National Research Council, Washington, D.C., 1997, pp 167-173.

17. Hurley, J.W. Utilization of Auxiliary Through Lanes at Signalized Intersections, In *Transportation Research Record 1484*, TRB, National Research Council, Washington, D.C., 1995, pp. 50-57.

18. Tarawneh, S.M. and T.M. Tarawneh, Effect of Utilization of Auxiliary Through Lanes of Downstream Right-Turn Activity, Transportation Research Board 80th Annual Meeting CD-ROM, Washington, D.C., 2001.

19. Tarawneh, S.M. Utilization of Auxiliary Through Lanes at Intersections of Four-Lane, Two-Way Roadways, In *Transportation Research Record 1737*, TRB, National Research Council, Washington, D.C., 2000, pp. 26-33.

20. SAS User's Guide. Release 8.02 edition. Cary, N.C., SAS Institute, Inc., 2001.

Appendices

APPENDIX A:

Geometric data collection form

Intersection Geometric Data Collection Form

General Information

Date: ____________________ Surveyor: ________________________

Time: ____________________ Weather: ________________________

Location

Intersection: ________________________ @ ___________________________

City, County: ____

Intersection

Type* #: ________________________ ID: _______________

Subject Movement Approach (circle one): EB WB NB SB

Posted Speed Limit (mph): EB _____, WB _____, NB _____, SB _____

* 2TS: Two through lanes, no exclusive right turn lane at signal
2TE: Two through lanes, exclusive right turn lane at signal
2LR: Two left lanes, turn onto a freeway on-ramp
2LS: Two left lanes, turn onto a surface street
3TS: Three through lanes, no exclusive right turn lane at signal
3TE: Three through lanes, exclusive right turn lane at signal
\# Use Supplemental intersection geometric data collection form if the intersection type is “2LS” or “2LR”

Geometric Characteristics

- Number of lanes of the subject movement (circle one): 2 3
- Type of lane drop (check one):

  ____ (Type I) Typical lane drop

  ____ (Type II) Changes of lane usage (e.g. through lane becomes right turn only lane)

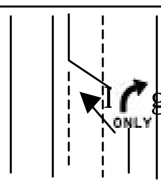
- Distance to upstream signal intersection: ______________ ft
- Approximate storage length of turning lane
  - o Left turn exclusive lane: _______ ft + _______ ft (taper)
  - o Right turn exclusive lane: _______ ft + _______ ft (taper)
- Cross street width (measure from stop bar to the other side curb-to-curb line):

  ________ ft

- Short lane length: ________ ft
    - Measure from the stop bar to the beginning of a taper for typical lane drop
    - Measure from the stop bar to the first arrow marking on the pavement for lane usage change lane drop
- Taper length: ________ ft
    - Measure the length from the beginning of the taper to the end of the taper
    - Measure the length from the beginning of the first marking to the end of the lane
- Signs

 signs		Warning Sign			Regulatory Sign		Other
Exist? (Y or N)							
Location	Subject movement Upstream (U) or Downstream (D)						
	Distance from stop bar (ft)						

- Pavement marking

	Merge Left	Right Turn Only
 gram		
Exist? (Y or N)		
Location of the first marking (distance from stop bar, in ft)		
Number of markings		

- Driveway (No. & activity)

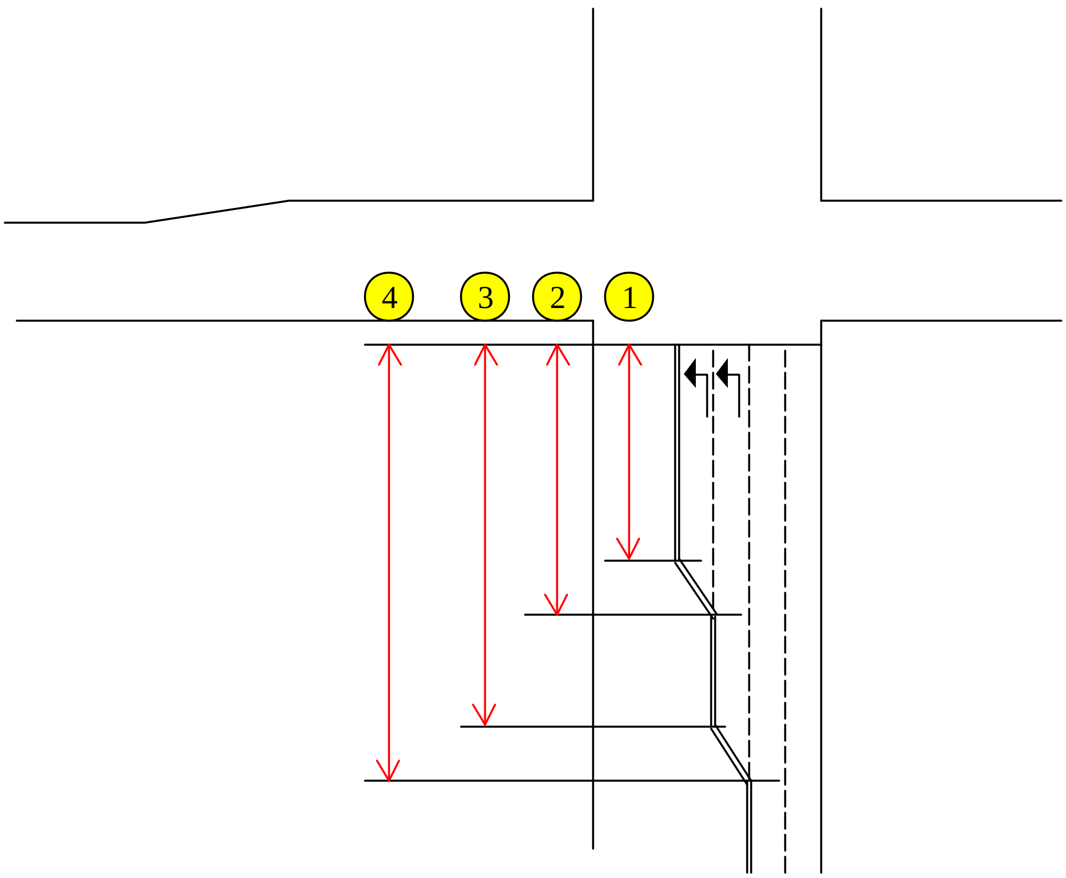
	Upstream		Downstream	
	Left side	Right side	Left side	Right side
No. of driveways				
Activity (high, medium, low)				

- TWLTL (Y or N)? _____ Upstream _____ Downstream
- Left turn bays (Y or N)? _____ Upstream _____ Downstream
- Land uses: Describe in intersection schematic drawing

<u>Supplemental Intersection Geometric Data Collection Form</u>

(for Dual Left-Turn intersections only)

Intersection Type (circle one): ***2LS*** ***2LR***

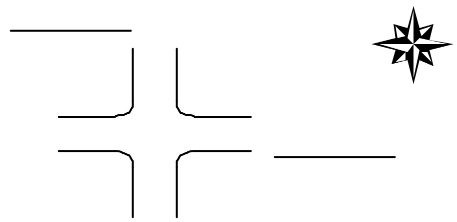


1. Inside left turn lane length (1): ________ ft
2. Inside left turn lane taper length (2-1): ________ ft
3. Outside left turn lane length (3): ________ ft
4. Outside left turn lane taper length(4-3): ________ ft

APPENDIX B:

Volume data collection form

Turning Volume Data Collection Form
Surveyor: ______________________ Date: - - (day)
Time Period: ____________________ ID: ________________
Intersection: ____________________@_______________________
Lane group of interest: ____________________ Page: of



Time Period	NB / SB / EB / WB					Time Period	NB / SB / EB / WB				
	LT	TH	RT	HV	Ped		LT	TH	RT	HV	Ped
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Queue Length Data Collection Form (for 2TS,2TE, 2LR and 2LS)
Surveyor: ______________________ Date: - - (day)
Time Period: ____________________ ID: ________________
Intersection: ____________________@_______________________
Lane group of interest: ____________________ Page: of

Green Start Time	Leftmost lane	HV	Rightmost lane	HV	HV Location	Green End Time	Note	Lane Volume
: :					/	: :		/
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: :					/	: :		/
: :					/	: :		/
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Queue Length Data Collection Form (for 3TE or 3TS)

Surveyor: ______________________ Date: - - (day)
Time Period: ____________________ ID: ________________
Intersection: ____________________@_______________________
Lane group of interest: ____________________ Page: of

Green Start Time	Left-most lane	HV	Middle lane	HV	Right-most lane	HV	HV Location	Green End Time	Note	Lane Volume
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: :							/ /	: :		/ /
: :							/ /	: :		/ /
: :							/ /	: :		/ /
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APPENDIX C:

Data for lane utilization prediction modeling

2TE intersection data

ID	f_{LU}	Avg_Invol (veh/h/l)	Short (ft)	drp-type	N-sign	MBAup	MBAdn
2TE-3	0.850	129	1496	Usage change	1	Exist	Exist
2TE-3	0.820	170	1496	Usage change	1	Exist	Exist
2TE-3	0.778	194	1496	Usage change	1	Exist	Exist
2TE-3	0.795	216	1496	Usage change	1	Exist	Exist
2TE-3	0.897	232	1496	Usage change	1	Exist	Exist
2TE-3	0.883	216	1496	Usage change	1	Exist	Exist
2TE-3	0.971	202	1496	Usage change	1	Exist	Exist
2TE-3	0.967	174	1496	Usage change	1	Exist	Exist
2TE-3	0.988	160	1496	Usage change	1	Exist	Exist
2TE-3	0.979	178	1496	Usage change	1	Exist	Exist
2TE-3	0.929	185	1496	Usage change	1	Exist	Exist
2TE-3	0.955	193	1496	Usage change	1	Exist	Exist
2TE-5	0.615	137	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.669	201	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.690	171	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.669	186	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.638	208	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.811	274	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.697	212	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.661	160	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.573	142	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.758	100	153	Usage change	0	Non_exst	Non_exst
2TE-5	0.644	132	153	Usage change	0	Non_exst	Non_exst
2TE-12	0.643	280	753	Usage change	1	Non_exst	Non_exst
2TE-12	0.748	301	753	Usage change	1	Non_exst	Non_exst
2TE-12	0.807	299	753	Usage change	1	Non_exst	Non_exst
2TE-12	0.734	375	753	Usage change	1	Non_exst	Non_exst
2TE-12	0.759	296	753	Usage change	1	Non_exst	Non_exst
2TE-12	0.684	302	753	Usage change	1	Non_exst	Non_exst
2TE-12	0.709	278	753	Usage change	1	Non_exst	Non_exst
2TE-12	0.697	214	753	Usage change	1	Non_exst	Non_exst
2TE-12	0.643	174	753	Usage change	1	Non_exst	Non_exst
2TE-13	0.583	220	284	Physical	2	Non_exst	Non_exst
2TE-13	0.561	192	284	Physical	2	Non_exst	Non_exst
2TE-13	0.563	211	284	Physical	2	Non_exst	Non_exst
2TE-13	0.606	171	284	Physical	2	Non_exst	Non_exst
2TE-13	0.646	218	284	Physical	2	Non_exst	Non_exst
2TE-13	0.612	220	284	Physical	2	Non_exst	Non_exst
2TE-13	0.605	140	284	Physical	2	Non_exst	Non_exst
2TE-13	0.577	192	284	Physical	2	Non_exst	Non_exst
2TE-13	0.582	225	284	Physical	2	Non_exst	Non_exst
2TE-13	0.579	205	284	Physical	2	Non_exst	Non_exst
2TE-13	0.571	262	284	Physical	2	Non_exst	Non_exst
2TE-A1	0.679	729	1079	Physical	2	Non_exst	Exist
2TE-A1	0.924	526	1079	Physical	2	Non_exst	Exist
2TE-A1	0.851	429	1079	Physical	2	Non_exst	Exist
2TE-A1	0.637	507	1079	Physical	2	Non_exst	Exist
2TE-A1	0.675	479	1079	Physical	2	Non_exst	Exist
2TE-A1	0.703	497	1079	Physical	2	Non_exst	Exist
2TE-A1	0.717	466	1079	Physical	2	Non_exst	Exist
2TE-A1	0.686	407	1079	Physical	2	Non_exst	Exist
2TE-A1	0.667	358	1079	Physical	2	Non_exst	Exist
2TE-A1	0.708	259	1079	Physical	2	Non_exst	Exist
2TE-A3	0.545	73	850	Physical	1	Exist	Exist

2TE-A3	0.542	109	850	Physical	1	Exist	Exist
2TE-A3	0.575	91	850	Physical	1	Exist	Exist
2TE-A3	0.603	142	850	Physical	1	Exist	Exist
2TE-A3	0.592	171	850	Physical	1	Exist	Exist
2TE-A3	0.664	153	850	Physical	1	Exist	Exist
2TE-A3	0.659	184	850	Physical	1	Exist	Exist
2TE-A3	0.647	170	850	Physical	1	Exist	Exist
2TE-A3	0.619	157	850	Physical	1	Exist	Exist
2TE-A3	0.706	119	850	Physical	1	Exist	Exist
2TE-A3	0.634	82	850	Physical	1	Exist	Exist
2TE-A4	0.860	292	450	Usage change	1	Non_exst	Exist
2TE-A4	0.833	332	450	Usage change	1	Non_exst	Exist
2TE-A4	0.857	388	450	Usage change	1	Non_exst	Exist
2TE-A4	0.792	349	450	Usage change	1	Non_exst	Exist
2TE-A4	0.774	363	450	Usage change	1	Non_exst	Exist
2TE-A4	0.838	411	450	Usage change	1	Non_exst	Exist
2TE-A4	0.812	347	450	Usage change	1	Non_exst	Exist
2TE-A4	0.900	274	450	Usage change	1	Non_exst	Exist
2TE-A4	0.825	358	450	Usage change	1	Non_exst	Exist
2TE-A4	0.798	302	450	Usage change	1	Non_exst	Exist
2TE-A4	0.804	286	450	Usage change	1	Non_exst	Exist
2TE-A4	0.824	303	450	Usage change	1	Non_exst	Exist
2TE-A5	0.660	67	611	Usage change	1	Exist	Exist
2TE-A5	0.778	57	611	Usage change	1	Exist	Exist
2TE-A5	0.654	72	611	Usage change	1	Exist	Exist
2TE-A5	0.586	85	611	Usage change	1	Exist	Exist
2TE-A5	0.690	76	611	Usage change	1	Exist	Exist
2TE-A5	0.652	97	611	Usage change	1	Exist	Exist
2TE-A5	0.826	323	611	Usage change	1	Exist	Exist
2TE-A5	0.825	327	611	Usage change	1	Exist	Exist
2TE-A5	0.993	320	611	Usage change	1	Exist	Exist
2TE-A5	0.928	339	611	Usage change	1	Exist	Exist
2TE-A5	0.732	300	611	Usage change	1	Exist	Exist
2TE-A9	0.594	152	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.606	176	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.563	184	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.579	254	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.580	259	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.618	246	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.599	182	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.553	215	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.690	222	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.589	212	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.605	149	595	Usage change	2	Non_exst	Non_exst
2TE-A9	0.601	190	595	Usage change	2	Non_exst	Non_exst

2TS intersection data

ID	fLU	Avg_Invol	Short	drp-type
2TS-3	0.672	88	918	Usage change
2TS-3	0.700	137	918	Usage change
2TS-3	0.670	121	918	Usage change
2TS-3	0.763	170	918	Usage change
2TS-3	0.669	179	918	Usage change
2TS-3	0.708	217	918	Usage change
2TS-3	0.724	177	918	Usage change
2TS-3	0.644	156	918	Usage change
2TS-3	0.673	144	918	Usage change
2TS-3	0.727	239	918	Usage change
2TS-3	0.721	207	918	Usage change
2TS-3	0.747	218	918	Usage change
2TS-4	0.928	459	2061	Usage change
2TS-4	0.957	501	2061	Usage change
2TS-4	0.956	347	2061	Usage change
2TS-4	0.824	317	2061	Usage change
2TS-4	0.898	349	2061	Usage change
2TS-4	0.922	356	2061	Usage change
2TS-4	0.956	383	2061	Usage change
2TS-4	0.973	355	2061	Usage change
2TS-4	0.888	334	2061	Usage change
2TS-4	0.989	365	2061	Usage change
2TS-4	0.994	298	2061	Usage change
2TS-4	0.929	276	2061	Usage change
2TS-7	0.665	366	638	Physical
2TS-7	0.638	379	638	Physical
2TS-7	0.634	382	638	Physical
2TS-7	0.670	436	638	Physical
2TS-7	0.619	362	638	Physical
2TS-7	0.641	551	638	Physical
2TS-7	0.668	576	638	Physical
2TS-7	0.634	474	638	Physical
2TS-7	0.627	435	638	Physical
2TS-7	0.659	411	638	Physical
2TS-7	0.640	300	638	Physical
2TS-7	0.677	268	638	Physical
2TS-13	0.765	154	1216	Usage change
2TS-13	0.843	175	1216	Usage change
2TS-13	0.778	145	1216	Usage change
2TS-13	0.871	217	1216	Usage change
2TS-13	0.813	226	1216	Usage change
2TS-13	0.792	173	1216	Usage change
2TS-13	0.837	176	1216	Usage change
2TS-13	0.774	168	1216	Usage change
2TS-13	0.821	188	1216	Usage change
2TS-13	0.771	108	1216	Usage change
2TS-13	0.855	130	1216	Usage change
2TS-13	0.816	104	1216	Usage change
2TS-15	0.567	203	574	Usage change
2TS-15	0.581	203	574	Usage change
2TS-15	0.593	338	574	Usage change
2TS-15	0.629	243	574	Usage change
2TS-15	0.643	236	574	Usage change

2TS-15	0.616	291	574	Usage change
2TS-15	0.583	199	574	Usage change
2TS-15	0.633	182	574	Usage change
2TS-15	0.733	176	574	Usage change
2TS-15	0.657	130	574	Usage change
2TS-15	0.625	115	574	Usage change
2TS-19	0.608	188	319	Usage change
2TS-19	0.641	177	319	Usage change
2TS-19	0.763	238	319	Usage change
2TS-19	0.675	316	319	Usage change
2TS-19	0.719	282	319	Usage change
2TS-19	0.763	360	319	Usage change
2TS-19	0.812	351	319	Usage change
2TS-19	0.782	320	319	Usage change
2TS-19	0.848	336	319	Usage change
2TS-19	0.768	344	319	Usage change
2TS-19	0.757	308	319	Usage change
2TS-19	0.703	210	319	Usage change
2TS-28	0.572	362	778	Physical
2TS-28	0.561	396	778	Physical
2TS-28	0.558	527	778	Physical
2TS-28	0.604	608	778	Physical
2TS-28	0.601	534	778	Physical
2TS-28	0.577	592	778	Physical
2TS-28	0.574	563	778	Physical
2TS-28	0.562	495	778	Physical
2TS-28	0.580	588	778	Physical
2TS-28	0.618	361	778	Physical
2TS-28	0.600	287	778	Physical
2TS-28	0.598	270	778	Physical
2TS-37	0.639	94	520	Usage change
2TS-37	0.622	92	520	Usage change
2TS-37	0.703	90	520	Usage change
2TS-37	0.618	84	520	Usage change
2TS-37	0.596	124	520	Usage change
2TS-37	0.600	92	520	Usage change
2TS-37	0.750	90	520	Usage change
2TS-37	0.646	106	520	Usage change
2TS-37	0.750	72	520	Usage change
2TS-37	0.717	66	520	Usage change
2TS-37	0.750	78	520	Usage change
2TS-37	0.672	86	520	Usage change
2TS-A4	0.697	289	178	Usage change
2TS-A4	0.643	296	178	Usage change
2TS-A4	0.626	348	178	Usage change
2TS-A4	0.635	396	178	Usage change
2TS-A4	0.626	348	178	Usage change
2TS-A4	0.612	416	178	Usage change
2TS-A4	0.633	403	178	Usage change
2TS-A4	0.633	334	178	Usage change
2TS-A4	0.704	352	178	Usage change
2TS-A4	0.655	300	178	Usage change
2TS-A4	0.629	226	178	Usage change
2TS-A4	0.649	201	178	Usage change
2TS-A5	0.511	193	148	Physical
2TS-A5	0.514	216	148	Physical
2TS-A5	0.514	225	148	Physical

2TS-A5	0.530	140	148	Physical
2TS-A5	0.527	156	148	Physical
2TS-A5	0.513	146	148	Physical

2LS intersection data

ID	fLU	Avg_Invol	MBAdn
2LS-2	0.882	132	Exist
2LS-2	0.765	105	Exist
2LS-2	0.862	102	Exist
2LS-2	0.919	119	Exist
2LS-2	0.854	129	Exist
2LS-2	0.761	121	Exist
2LS-2	0.804	161	Exist
2LS-2	0.848	102	Exist
2LS-2	0.811	145	Exist
2LS-2	0.660	68	Exist
2LS-2	0.797	140	Exist
2LS-2	0.917	135	Exist
2LS-4	0.794	55	Exist
2LS-4	0.800	33	Exist
2LS-4	0.938	31	Exist
2LS-4	0.708	34	Exist
2LS-4	0.647	45	Exist
2LS-4	0.763	59	Exist
2LS-4	0.824	50	Exist
2LS-4	0.848	78	Exist
2LS-4	0.725	58	Exist
2LS-4	0.742	99	Exist
2LS-4	0.732	79	Exist
2LS-4	0.773	86	Exist
2LS-7	0.717	90	Non_exst
2LS-7	0.719	174	Non_exst
2LS-7	0.795	116	Non_exst
2LS-7	0.783	69	Non_exst
2LS-7	0.719	86	Non_exst
2LS-7	0.597	79	Non_exst
2LS-7	0.774	153	Non_exst
2LS-7	0.719	87	Non_exst
2LS-7	0.708	95	Non_exst
2LS-7	0.606	75	Non_exst
2LS-7	0.900	76	Non_exst
2LS-7	0.763	71	Non_exst
2LS-8	0.667	24	Non_exst
2LS-8	0.583	28	Non_exst
2LS-8	0.647	42	Non_exst
2LS-8	0.569	77	Non_exst
2LS-8	0.659	62	Non_exst
2LS-8	0.679	71	Non_exst
2LS-8	0.732	76	Non_exst
2LS-8	0.648	65	Non_exst
2LS-8	0.731	71	Non_exst
2LS-8	0.643	41	Non_exst
2LS-8	0.658	50	Non_exst
2LS-8	0.625	39	Non_exst
2LS-A1	0.708	71	Non_exst
2LS-A1	0.706	51	Non_exst
2LS-A1	0.595	50	Non_exst
2LS-A1	0.650	54	Non_exst
2LS-A1	0.563	47	Non_exst

2LS-A1	0.841	96	Non_exst
2LS-A1	0.650	153	Non_exst
2LS-A1	0.676	92	Non_exst
2LS-A1	0.591	53	Non_exst
2LS-A1	0.630	60	Non_exst
2LS-A1	0.583	43	Non_exst
2LS-A1	0.619	48	Non_exst
2LS-O1	0.567	66	Non_exst
2LS-O1	0.636	29	Non_exst
2LS-O1	0.587	54	Non_exst
2LS-O1	0.786	22	Non_exst
2LS-O1	0.605	50	Non_exst
2LS-O1	0.692	36	Non_exst
2LS-O1	0.540	56	Non_exst
2LS-O1	0.500	18	Non_exst
2LS-O1	0.833	21	Non_exst
2LS-O1	0.625	22	Non_exst
2LS-O1	0.750	6	Non_exst
2LS-O1	0.643	20	Non_exst
2LS-O2	0.714	20	Non_exst
2LS-O2	0.778	28	Non_exst
2LS-O2	0.563	15	Non_exst
2LS-O2	0.909	41	Non_exst
2LS-O2	0.625	20	Non_exst
2LS-O2	0.571	20	Non_exst
2LS-O2	0.722	29	Non_exst
2LS-O2	0.955	43	Non_exst
2LS-O2	0.944	34	Non_exst
2LS-O2	0.786	22	Non_exst
2LS-O2	0.625	31	Non_exst
2LS-O2	0.929	26	Non_exst

2LR intersection data

ID	f_{LU}	Avg_Invol	Short	Taper	drp-type
2LR-1	0.903	377	685	527	Right drop
2LR-1	0.927	421	685	527	Right drop
2LR-1	0.860	376	685	527	Right drop
2LR-1	0.803	360	685	527	Right drop
2LR-1	0.852	303	685	527	Right drop
2LR-1	0.871	245	685	527	Right drop
2LR-1	0.877	251	685	527	Right drop
2LR-1	0.787	258	685	527	Right drop
2LR-1	0.800	225	685	527	Right drop
2LR-1	0.815	236	685	527	Right drop
2LR-1	0.818	252	685	527	Right drop
2LR-1	0.787	191	685	527	Right drop
2LR-2	0.696	188	646	470	Right drop
2LR-2	0.727	191	646	470	Right drop
2LR-2	0.822	250	646	470	Right drop
2LR-2	0.781	228	646	470	Right drop
2LR-2	0.889	288	646	470	Right drop
2LR-2	0.810	256	646	470	Right drop
2LR-2	0.688	190	646	470	Right drop
2LR-2	0.670	134	646	470	Right drop
2LR-2	0.611	111	646	470	Right drop
2LR-2	0.707	129	646	470	Right drop
2LR-2	0.667	113	646	470	Right drop
2LR-2	0.700	58	646	470	Right drop
2LR-4	0.778	351	944	286	Right drop
2LR-4	0.857	379	944	286	Right drop
2LR-4	0.765	335	944	286	Right drop
2LR-4	0.808	306	944	286	Right drop
2LR-4	0.841	274	944	286	Right drop
2LR-4	0.722	245	944	286	Right drop
2LR-4	0.743	205	944	286	Right drop
2LR-4	0.688	207	944	286	Right drop
2LR-8	0.743	290	548	500	Right drop
2LR-8	0.756	260	548	500	Right drop
2LR-8	0.835	354	548	500	Right drop
2LR-8	0.792	342	548	500	Right drop
2LR-8	0.842	394	548	500	Right drop
2LR-8	0.771	364	548	500	Right drop
2LR-8	0.843	424	548	500	Right drop
2LR-8	0.797	322	548	500	Right drop
2LR-8	0.829	348	548	500	Right drop
2LR-8	0.810	345	548	500	Right drop
2LR-8	0.750	265	548	500	Right drop
2LR-9	0.645	76	790	260	Right drop
2LR-9	0.641	99	790	260	Right drop
2LR-9	0.689	123	790	260	Right drop
2LR-9	0.744	115	790	260	Right drop
2LR-9	0.688	87	790	260	Right drop
2LR-9	0.700	83	790	260	Right drop
2LR-9	0.667	95	790	260	Right drop
2LR-9	0.618	93	790	260	Right drop
2LR-9	0.776	89	790	260	Right drop
2LR-9	0.628	105	790	260	Right drop

2LR-9	0.563	75	790	260	Right drop
2LR-13	1.000	184	735	361	Left drop
2LR-13	0.989	173	735	361	Left drop
2LR-13	0.906	173	735	361	Left drop
2LR-13	0.953	197	735	361	Left drop
2LR-13	0.963	149	735	361	Left drop
2LR-13	0.768	257	735	361	Left drop
2LR-13	0.813	222	735	361	Left drop
2LR-13	0.815	199	735	361	Left drop
2LR-13	0.891	165	735	361	Left drop
2LR-13	0.895	128	735	361	Left drop
2LR-A6	0.583	69	1574	774	Right drop
2LR-A6	0.719	46	1574	774	Right drop
2LR-A6	0.692	77	1574	774	Right drop
2LR-A6	0.607	32	1574	774	Right drop
2LR-A6	0.656	42	1574	774	Right drop
2LR-A6	0.600	40	1574	774	Right drop
2LR-A6	0.786	44	1574	774	Right drop
2LR-A6	0.618	40	1574	774	Right drop
2LR-A6	0.667	60	1574	774	Right drop
2LR-A6	0.786	47	1574	774	Right drop
2LR-A6	0.556	57	1574	774	Right drop
2LR-A6	0.717	49	1574	774	Right drop

3TE intersection data

ID	fLU	Avg_Invol	Short	MBAup
3TE-5	0.867	449	1529	Non_exst
3TE-5	0.916	381	1529	Non_exst
3TE-5	0.875	446	1529	Non_exst
3TE-5	0.901	375	1529	Non_exst
3TE-5	0.853	446	1529	Non_exst
3TE-5	0.827	484	1529	Non_exst
3TE-5	0.901	533	1529	Non_exst
3TE-5	0.839	429	1529	Non_exst
3TE-5	0.822	376	1529	Non_exst
3TE-5	0.910	369	1529	Non_exst
3TE-5	0.761	339	1529	Non_exst
3TE-5	0.859	324	1529	Non_exst
3TE-8	0.765	415	593	Exist
3TE-8	0.785	463	593	Exist
3TE-8	0.794	440	593	Exist
3TE-8	0.752	461	593	Exist
3TE-8	0.789	429	593	Exist
3TE-8	0.725	391	593	Exist
3TE-8	0.756	389	593	Exist
3TE-8	0.747	322	593	Exist
3TE-8	0.711	314	593	Exist
3TE-8	0.725	285	593	Exist
3TE-8	0.714	296	593	Exist
3TE-A3	0.765	862	1178	Non_exst
3TE-A3	0.797	758	1178	Non_exst
3TE-A3	0.790	813	1178	Non_exst
3TE-A3	0.766	877	1178	Non_exst
3TE-A3	0.823	1028	1178	Non_exst
3TE-A3	0.755	856	1178	Non_exst
3TE-A3	0.764	810	1178	Non_exst
3TE-A3	0.797	819	1178	Non_exst
3TE-A3	0.752	632	1178	Non_exst
3TE-A3	0.773	610	1178	Non_exst
3TE-A3	0.753	549	1178	Non_exst
3TE-A4	0.654	193	120	Exist
3TE-A4	0.615	210	120	Exist
3TE-A4	0.610	233	120	Exist
3TE-A4	0.600	248	120	Exist
3TE-A4	0.609	200	120	Exist
3TE-A4	0.685	226	120	Exist
3TE-A4	0.529	245	120	Exist
3TE-A4	0.632	264	120	Exist
3TE-A4	0.636	286	120	Exist
3TE-A4	0.597	287	120	Exist
3TE-A4	0.612	288	120	Exist

3TS intersection data

ID	f_{LU}	HVpct	Rtvol	MBAdn
3TS-1	0.810	2.21	87.3	Exist
3TS-1	0.781	0.81	74.9	Exist
3TS-1	0.833	1.48	87.9	Exist
3TS-1	0.764	0.26	112.1	Exist
3TS-1	0.815	1.39	103.9	Exist
3TS-1	0.748	0.66	95.1	Exist
3TS-1	0.738	0.38	96.8	Exist
3TS-1	0.808	1.06	136.2	Exist
3TS-1	0.804	0.58	120.0	Exist
3TS-1	0.789	0.26	60.9	Exist
3TS-1	0.786	0.58	44.4	Exist
3TS-1	0.796	0.28	45.7	Exist
3TS-4	0.741	2.58	454.8	Non_exst
3TS-4	0.783	1.82	546.0	Non_exst
3TS-4	0.898	2.11	576.2	Non_exst
3TS-4	0.960	2.28	768.0	Non_exst
3TS-4	0.875	0.86	708.0	Non_exst
3TS-4	0.807	1.12	620.0	Non_exst
3TS-4	0.760	1.62	553.5	Non_exst
3TS-4	0.845	1.04	588.0	Non_exst
3TS-4	0.730	2.42	408.7	Non_exst
3TS-4	0.846	1.79	470.2	Non_exst
3TS-4	0.843	0.49	354.8	Non_exst
3TS-5	0.697	2.42	136.1	Non_exst
3TS-5	0.696	1.15	157.2	Non_exst
3TS-5	0.712	0.84	150.0	Non_exst
3TS-5	0.664	1.01	143.4	Non_exst
3TS-5	0.736	1.05	166.9	Non_exst
3TS-5	0.760	2.41	180.0	Non_exst
3TS-5	0.752	4.19	121.6	Non_exst
3TS-5	0.780	3.82	149.8	Non_exst
3TS-5	0.725	3.21	126.8	Non_exst
3TS-5	0.775	4.15	118.6	Non_exst
3TS-5	0.738	1.48	114.5	Non_exst
3TS-5	0.789	2.22	116.5	Non_exst
3TS-11	0.756	4.79	16.6	Non_exst
3TS-11	0.858	5.18	11.9	Non_exst
3TS-11	0.904	4.61	19.8	Non_exst
3TS-11	0.931	4.62	15.8	Non_exst
3TS-11	0.932	3.75	15.8	Non_exst
3TS-11	0.859	4.19	18.5	Non_exst
3TS-11	0.962	5.82	12.1	Non_exst
3TS-11	0.902	5.53	7.9	Non_exst
3TS-11	0.778	3.92	19.3	Non_exst
3TS-11	0.818	5.56	26.5	Non_exst
3TS-11	0.762	3.51	12.6	Non_exst
3TS-11	0.752	3.88	11.4	Non_exst
3TS-A1	0.663	1.62	40.3	Exist
3TS-A1	0.740	0.99	131.7	Exist
3TS-A1	0.791	1.35	263.3	Exist
3TS-A1	0.828	0.44	452.6	Exist
3TS-A1	0.859	2.31	407.8	Exist
3TS-A1	0.837	0.69	352.8	Exist

3TS-A1	0.819	1.03	343.5	Exist
3TS-A1	0.746	0.53	426.5	Exist
3TS-A1	0.914	0.68	193.7	Exist
3TS-A1	0.883	1.51	159.7	Exist
3TS-O1	0.744	4.40	14.7	Non_exst
3TS-O1	0.686	2.52	0.0	Non_exst
3TS-O1	0.681	2.04	4.0	Non_exst
3TS-O1	0.735	2.04	4.0	Non_exst
3TS-O1	0.767	2.37	12.0	Non_exst
3TS-O1	0.776	2.31	20.4	Non_exst
3TS-O1	0.768	4.68	0.0	Non_exst
3TS-O1	0.734	1.95	3.8	Non_exst
3TS-O1	0.713	2.36	11.3	Non_exst

APPENDIX D:

SAS ® output for lane utilization prediction models

2TE

- **Multicollinearity test**

The REG Procedure

Model: MODEL1
Dependent Variable: fLU fLU

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	5	0.88443	0.17689	37.63	<.0001
Error	82	0.38540	0.00470		
Corrected Total	87	1.26983			

Root MSE	0.06856	R-Square	0.6965
Dependent Mean	0.72850	Adj R-Sq	0.6780
Coeff Var	9.41064		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Tolerance	Variance Inflation
Intercept	Intercept	1	0.46693	0.02910	16.04	<.0001	.	0
Avg_lnvol	Avg_lnvol	1	0.00041783	0.00006577	6.35	<.0001	0.83963	1.19100
Short	Short	1	0.00010851	0.00002254	4.81	<.0001	0.69174	1.44563
N_sign	N-sign	1	-0.06579	0.01558	-4.22	<.0001	0.61236	1.63303
N_mark	N-mark	1	0.01795	0.00327	5.49	<.0001	0.75832	1.31870
DdrwyupR	DdrwyupR	1	0.01855	0.00334	5.56	<.0001	0.95302	1.04929

- ANOVA

The GLM Procedure

Dependent Variable: L_fLU log(fLU)

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	5	1.67114192	0.33422838	43.61	<.0001
Error	82	0.62849108	0.00766453		
Corrected Total	87	2.29963299			

R-Square	Coeff Var	Root MSE	L_fLU Mean
0.726699	-26.53061	0.087547	-0.329986

Source	DF	Type I SS	Mean Square	F Value	Pr > F
drp_type	1	0.18105129	0.18105129	23.62	<.0001
MBAdn	1	0.89524275	0.89524275	116.80	<.0001
Avg_lnvol	1	0.24355396	0.24355396	31.78	<.0001
Short	1	0.12322931	0.12322931	16.08	0.0001
N_sign	1	0.22806460	0.22806460	29.76	<.0001

Source	DF	Type III SS	Mean Square	F Value	Pr > F
drp_type	1	0.56332971	0.56332971	73.50	<.0001
MBAdn	1	0.27697770	0.27697770	36.14	<.0001
Avg_lnvol	1	0.41079819	0.41079819	53.60	<.0001
Short	1	0.26253920	0.26253920	34.25	<.0001
N_sign	1	0.22806460	0.22806460	29.76	<.0001

Parameter		Estimate	Standard Error	t Value	Pr > |t|
Intercept		-.5393337219 B	0.02769065	-19.48	<.0001
drp_type	Physical	-.2182460253 B	0.02545702	-8.57	<.0001
drp_type	Usage change	0.0000000000 B	.	.	.
MBAdn	Exist	0.1477394390 B	0.02457631	6.01	<.0001
MBAdn	Non_exst	0.0000000000 B	.	.	.
Avg_lnvol		0.0006272527	0.00008568	7.32	<.0001
Short		0.0001782071	0.00003045	5.85	<.0001
N_sign		-.1046509093	0.01918477	-5.45	<.0001

2TS

- **Multicollinearity test**

The REG Procedure

Model: MODEL1
Dependent Variable: fLU fLU

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	6	1.14537	0.19089	55.10	<.0001
Error	106	0.36722	0.00346		
Corrected Total	112	1.51259			

Root MSE	0.05886	R-Square	0.7572
Dependent Mean	0.69698	Adj R-Sq	0.7435
Coeff Var	8.44485		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Tolerance	Variance Inflation
Intercept	Intercept	1	0.59618	0.02329	25.60	<.0001	.	0
Avg_lnvol	Avg_lnvol	1	0.00006570	0.00006682	0.98	0.3277	0.37680	2.65393
Short	Short	1	0.00019812	0.00001172	16.90	<.0001	0.76372	1.30937
RTpct	RTpct	1	0.03765	0.04087	0.92	0.3590	0.80212	1.24670
N_sign	N-sign	1	-0.10985	0.01846	-5.95	<.0001	0.38515	2.59640
N_mark	N-mark	1	0.01643	0.00338	4.86	<.0001	0.72213	1.38479
DdrwydnR	DdrwydnR	1	0.00106	0.00046768	2.27	0.0250	0.58384	1.71279

- ANOVA

The GLM Procedure

Dependent Variable: fLU fLU

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	3	1.13490585	0.37830195	109.18	<.0001
Error	109	0.37768521	0.00346500		
Corrected Total	112	1.51259106			

R-Square	Coeff Var	Root MSE	fLU Mean
0.750306	8.445607	0.058864	0.696981

Source	DF	Type I SS	Mean Square	F Value	Pr > F
drp_type	1	0.41480554	0.41480554	119.71	<.0001
Short	1	0.69937240	0.69937240	201.84	<.0001
Avg_lnvol	1	0.02072791	0.02072791	5.98	0.0161

Source	DF	Type III SS	Mean Square	F Value	Pr > F
drp_type	1	0.22457710	0.22457710	64.81	<.0001
Short	1	0.59322129	0.59322129	171.20	<.0001
Avg_lnvol	1	0.02072791	0.02072791	5.98	0.0161

Parameter	Estimate	Standard Error	t Value	Pr > |t|
Intercept	0.5882381543 B	0.01409324	41.74	<.0001
drp_type Physical	-.1231181222 B	0.01529294	-8.05	<.0001
Short	0.0001414414	0.00001081	13.08	<.0001
Avg_lnvol	0.0001209948	0.00004947	2.45	0.0161

2LS

- **Multicollinearity test**

The REG Procedure

Model: MODEL1

Dependent Variable: fLU fLU

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	4	0.26714	0.06678	12.69	<.0001
Error	55	0.28956	0.00526		
Corrected Total	59	0.55670			

Root MSE	0.07256	R-Square	0.4799
Dependent Mean	0.72713	Adj R-Sq	0.4420
Coeff Var	9.97882		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Tolerance	Variance Inflation

Intercept	Intercept	1	0.62474	0.05556	11.24	<.0001	.	0
Avg_lnvol	Avg_lnvol	1	0.00087947	0.00031207	2.82	0.0067	0.68006	1.47046
Short	Short	1	0.00014290	0.00003508	4.07	0.0001	0.89536	1.11686
Taper	Taper	1	-0.00016976	0.00007561	-2.25	0.0288	0.69208	1.44491
HVpct	HVpct	1	-0.00228	0.00202	-1.13	0.2642	0.82943	1.20565

- ANOVA

The GLM Procedure

Dependent Variable: fLU fLU

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	2	0.26246736	0.13123368	25.42	<.0001
Error	57	0.29423507	0.00516202		
Corrected Total	59	0.55670244			

R-Square	Coeff Var	Root MSE	fLU Mean
0.471468	9.880950	0.071847	0.727128

Source	DF	Type I SS	Mean Square	F Value	Pr > F
MBAdn	1	0.20633593	0.20633593	39.97	<.0001
Avg_lnvol	1	0.05613144	0.05613144	10.87	0.0017

Source	DF	Type III SS	Mean Square	F Value	Pr > F
MBAdn	1	0.14999796	0.14999796	29.06	<.0001
Avg_lnvol	1	0.05613144	0.05613144	10.87	0.0017

Parameter		Estimate	Standard Error	t Value	Pr > |t|
Intercept		0.6161019242 B	0.02258482	27.28	<.0001
MBAdn	Exist	0.1048934591 B	0.01945878	5.39	<.0001
Avg_lnvol		0.0008636141	0.00026189	3.30	0.0017

2LR

- **Multicollinearity test**

The REG Procedure

Model: MODEL1
Dependent Variable: fLU fLU

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	4	0.22423	0.05606	9.33	<.0001
Error	59	0.35442	0.00601		
Corrected Total	63	0.57865			

Root MSE	0.07751	R-Square	0.3875
Dependent Mean	0.78416	Adj R-Sq	0.3460
Coeff Var	9.88398		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Tolerance	Variance Inflation
Intercept	Intercept	1	0.54381	0.08498	6.40	<.0001	.	0
Avg_lnvol	Avg_lnvol	1	0.00035868	0.00010844	3.31	0.0016	0.80318	1.24506
Short	Short	1	-0.00000165	0.00009389	-0.02	0.9860	0.79275	1.26143
N_sign	N-sign	1	-0.01350	0.03273	-0.41	0.6814	0.57512	1.73877
fstinfo	fstinfo	1	0.00049625	0.00015385	3.23	0.0021	0.61906	1.61535

- ANOVA

The GLM Procedure

Dependent Variable: fLU fLU

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	4	0.39751526	0.09937881	32.37	<.0001
Error	59	0.18113523	0.00307009		
Corrected Total	63	0.57865049			

R-Square	Coeff Var	Root MSE	fLU Mean
0.686970	7.065959	0.055408	0.784159

Source	DF	Type I SS	Mean Square	F Value	Pr > F
drp_type	1	0.15705671	0.15705671	51.16	<.0001
Avg_lnvol	1	0.21333443	0.21333443	69.49	<.0001
Short	1	0.00022203	0.00022203	0.07	0.7889
Taper	1	0.02690208	0.02690208	8.76	0.0044

Source	DF	Type III SS	Mean Square	F Value	Pr > F
drp_type	1	0.24455128	0.24455128	79.66	<.0001
Avg_lnvol	1	0.08628428	0.08628428	28.10	<.0001
Short	1	0.01541499	0.01541499	5.02	0.0288
Taper	1	0.02690208	0.02690208	8.76	0.0044

Parameter	Estimate	Standard Error	t Value	Pr > |t|
Intercept	0.3228481150 B	0.11807488	2.73	0.0082
drp_type Left lane drop	0.1756439378 B	0.01967993	8.93	<.0001
Avg_lnvol	0.0004527185	0.00008540	5.30	<.0001
Short	0.0002366810	0.00010563	2.24	0.0288
Taper	0.0003966131	0.00013398	2.96	0.0044

3TE

- **Multicollinearity test**

The REG Procedure

Model: MODEL1
Dependent Variable: fLU fLU

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	3	0.33468	0.11156	66.81	<.0001
Error	41	0.06846	0.00167		
Corrected Total	44	0.40315			

Root MSE	0.04086	R-Square	0.8302
Dependent Mean	0.75358	Adj R-Sq	0.8178
Coeff Var	5.42250		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Tolerance	Variance Inflation
Intercept	Intercept	1	0.60893	0.01603	38.00	<.0001	.	0
Avg_lnvol	Avg_lnvol	1	-0.00009817	0.00004226	-2.32	0.0252	0.45743	2.18610
Short	Short	1	0.00016431	0.00001323	12.42	<.0001	0.71809	1.39259
N_sign	N-sign	1	0.01694	0.00620	2.73	0.0092	0.58166	1.71920

- ANOVA

The GLM Procedure

Dependent Variable: fLU fLU

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	3	0.35443630	0.11814543	99.45	<.0001
Error	41	0.04870891	0.00118802		
Corrected Total	44	0.40314521			

R-Square	Coeff Var	Root MSE	fLU Mean
0.879178	4.573868	0.034468	0.753579

Source	DF	Type I SS	Mean Square	F Value	Pr > F
Short	1	0.32102775	0.32102775	270.22	<.0001
MBAup	1	0.02973927	0.02973927	25.03	<.0001
Avg_lnvol	1	0.00366928	0.00366928	3.09	0.0863

Source	DF	Type III SS	Mean Square	F Value	Pr > F
Short	1	0.14348284	0.14348284	120.77	<.0001
MBAup	1	0.03224090	0.03224090	27.14	<.0001
Avg_lnvol	1	0.00366928	0.00366928	3.09	0.0863

Parameter		Estimate	Standard Error	t Value	Pr > |t|
Intercept		0.4033380156 B	0.04452887	9.06	<.0001
Short		0.0002813991	0.00002561	10.99	<.0001
MBAup	Exist	0.1621229359 B	0.03112097	5.21	<.0001
Avg_lnvol		0.0000575972	0.00003277	1.76	0.0863

3TS

- **Multicollinearity test**

The REG Procedure

Model: MODEL1
Dependent Variable: fLU fLU

Analysis of Variance

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	6	0.10547	0.01758	7.90	<.0001
Error	47	0.10459	0.00223		
Corrected Total	53	0.21006			

Root MSE	0.04717	R-Square	0.5021
Dependent Mean	0.77900	Adj R-Sq	0.4385
Coeff Var	6.05554		

Parameter Estimates

Variable	Label	DF	Parameter Estimate	Standard Error	t Value	Pr > |t|	Tolerance	Variance Inflation
Intercept	Intercept	1	0.37088	0.11797	3.14	0.0029	.	0
Avg_lnvol	Avg_lnvol	1	-0.00008561	0.00006958	-1.23	0.2247	0.56942	1.75619
Short	Short	1	0.00014506	0.00004683	3.10	0.0033	0.17987	5.55944
Rtvol	Rtvol	1	0.00024667	0.00004991	4.94	<.0001	0.39479	2.53298
HVpct	HVpct	1	0.01007	0.00815	1.24	0.2225	0.51585	1.93855
N_sign	N-sign	1	0.04405	0.01632	2.70	0.0096	0.26546	3.76706
N_mark	N-mark	1	0.04573	0.01102	4.15	0.0001	0.15861	6.30480

- **ANOVA**

The GLM Procedure

Dependent Variable: fLU   fLU

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	3	0.05811335	0.01937112	10.51	<.0001
Error	39	0.07190491	0.00184372		
Corrected Total	42	0.13001826			

R-Square	Coeff Var	Root MSE	fLU Mean
0.446963	5.598510	0.042939	0.766963

Source	DF	Type I SS	Mean Square	F Value	Pr > F
MBAdn	1	0.04283225	0.04283225	23.23	<.0001
Rtvol	1	0.00525402	0.00525402	2.85	0.0994
HVpct	1	0.01002709	0.01002709	5.44	0.0250

Source	DF	Type III SS	Mean Square	F Value	Pr > F
MBAdn	1	0.03567169	0.03567169	19.35	<.0001
Rtvol	1	0.00621573	0.00621573	3.37	0.0740
HVpct	1	0.01002709	0.01002709	5.44	0.0250

Parameter		Estimate	Standard Error	t Value	Pr > |t|
Intercept		0.6823308732 B	0.02153525	31.68	<.0001
MBAdn	Exist	0.0791174889 B	0.01798696	4.40	<.0001
Rtvol		0.0001144581	0.00006234	1.84	0.0740
HVpct		0.0170780480	0.00732315	2.33	0.0250

APPENDIX E:

Collision data for analysis

Collision data

ID No.	ADT (veh/day)		Collisions per 3 yrs		Section length (mi)		Collision rate (collision/mvmt)	
	Upstrm	Dnstrm	Upstrm	Dnstrm	Upstrm	Dnstrm	Upstrm	Dnstrm
2LR-1	10562	12426	4	0	0.09	0.21	9.085	0.000
2LR-11	6909	6909	0	0	0.14	0.13	0.000	0.000
2LR-12	6435	6435	0	4	0.12	0.14	0.000	3.939
2LR-13	3771	4847	0	3	0.08	0.19	0.000	3.006
2LR-14	10816	10816	0	1	0.09	0.14	0.000	0.583
2LR-16	13817	13817	4	10	0.07	0.25	11.537	2.634
2LR-17	15180	15180	1	3	0.15	0.15	0.576	1.241
2LR-2	7224	8499	0	0	0.09	0.19	0.000	0.000
2LR-4	8598	8598	1	2	0.09	0.22	2.407	0.960
2LR-8	6058	8640	2	3	0.17	0.18	2.503	1.757
2LR-9	4104	4320	1	2	0.19	0.18	1.596	2.342
2LR-A2	11143	11143	4	0	0.15	0.14	3.229	0.000
2LR-A3	13029	13029	2	5	0.10	0.45	2.570	0.784
2LR-A5	7883	7883	0	4	0.12	0.15	0.000	3.039
2LR-A6	7000	7000	0	1	0.18	0.42	0.000	0.314
2LS-11	1300	12000	0	1	0.11	0.16	0.000	0.462
2LS-12	1600	7100	0	1	0.13	0.37	0.000	0.344
2LS-13	1200	6600	0	0	0.10	0.15	0.000	0.000
2LS-2	2050	11000	2	3	0.18	0.15	6.918	1.665
2LS-4	2000	13000	5	4	0.14	0.23	24.255	1.246
2LS-7	1617	12000	1	1	0.11	0.09	9.621	0.804
2LS-8	1242	15000	1	9	0.10	0.19	15.596	2.859
2LS-9	1500	15000	0	7	0.05	0.17	0.000	2.446
2LS-A1	1083	22000	0	1	0.11	0.09	0.000	0.484
2LS-A3	1200	7500	0	1	0.05	0.18	0.000	0.691
2LS-A4	2000	20000	3	4	0.11	0.24	24.029	0.755
2TE-11	18000	18000	0	2	0.08	0.22	0.000	0.469
2TE-12	12000	12000	1	3	0.15	0.23	0.773	0.990
2TE-13	31000	31000	1	6	0.08	0.07	1.088	2.674
2TE-14	31000	20000	1	7	0.11	0.32	0.527	0.990
2TE-17	15000	15000	5	13	0.19	0.27	2.184	2.924
2TE-19	16000	16000	1	3	0.07	0.12	2.598	1.454
2TE-24	21000	21000	2	5	0.19	0.24	0.624	0.889
2TE-3	27000	17000	4	11	0.19	0.36	0.971	1.658
2TE-5	18000	18000	0	1	0.04	0.10	0.000	0.516
2TE-A1	17000	17000	1	4	0.19	0.25	0.385	0.857
2TE-A11	5000	5000	0	0	0.19	0.12	0.000	0.000
2TE-A12	16000	16000	1	15	0.19	0.28	0.409	3.056
2TE-A3	17000	17000	0	1	0.19	0.17	0.000	0.312
2TE-A4	31000	31000	6	31	0.16	0.24	1.634	3.729
2TE-A5	22000	19000	8	6	0.19	0.15	2.382	1.962
2TE-A9	19000	12000	2	13	0.13	0.23	1.247	4.346

2TS-13	13000	13000	2	5	0.19	0.23	1.008	1.508
2TS-14	19000	19000	3	18	0.14	0.17	1.532	5.110
2TS-15	6400	6400	9	1	0.19	0.13	9.213	1.103
2TS-19	21000	17000	0	1	0.15	0.14	0.000	0.375
2TS-2	20000	19000	3	6	0.19	0.27	0.983	1.066
2TS-25	17000	12000	0	1	0.19	0.15	0.000	0.523
2TS-27	5000	5000	0	5	0.11	0.05	0.000	19.288
2TS-28	17000	17000	2	3	0.19	0.24	0.771	0.660
2TS-29	16000	14000	0	1	0.19	0.32	0.000	0.206
2TS-3	18000	18000	0	0	0.19	0.19	0.000	0.000
2TS-30	26000	26000	15	5	0.16	0.28	4.994	0.632
2TS-37	7800	7800	1	3	0.19	0.15	0.840	2.375
2TS-39	25000	18000	0	5	0.18	0.23	0.000	1.115
2TS-4	22000	22000	2	9	0.19	0.49	0.596	0.756
2TS-44	10000	10000	1	9	0.19	0.14	0.655	5.953
2TS-45	17000	27000	7	21	0.19	0.23	2.698	3.107
2TS-5	7800	7800	5	2	0.19	0.41	4.200	0.576
2TS-6	17000	17000	6	6	0.19	0.32	2.312	0.999
2TS-7	19000	18000	5	5	0.23	0.22	1.329	1.151
2TS-9	6300	6300	2	0	0.19	0.05	2.080	0.000
2TS-A3	19000	19000	4	0	0.10	0.06	3.831	0.000
2TS-A5	9800	9800	6	1	0.19	0.09	4.095	1.000
2TS-A6	17000	17000	0	3	0.10	0.06	0.000	2.865
3TE-11	34000	34000	7	5	0.24	0.10	1.007	1.323
3TE-8	45000	44000	16	30	0.19	0.15	2.329	4.029
3TE-A2	38000	38000	2	2	0.17	0.14	0.409	0.355
3TE-A3	46000	46000	10	12	0.19	0.30	1.424	0.795
3TE-A4	30000	32000	7	5	0.15	0.11	2.080	1.353
3TE-A5	33000	33000	2	2	0.13	0.08	0.678	0.654
3TS-1	43000	37000	44	18	0.19	0.19	6.704	2.323
3TS-11	38000	46000	1	6	0.27	0.15	0.111	0.817
3TS-2	43000	43000	0	21	0.06	0.14	0.000	3.208
3TS-4	46000	40000	1	39	0.12	0.20	0.274	4.456
3TS-5	17000	17000	14	3	0.12	0.20	10.703	0.820
3TS-A1	34000	27000	3	5	0.13	0.28	0.985	0.596
3TS-A2	30000	30000	0	14	0.19	0.28	0.000	1.524
3TS-A3	43000	42000	6	14	0.19	0.29	0.914	1.062
3TS-A4	42000	43000	6	5	0.19	0.22	0.946	0.491