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Crashes in the Vicinity of Major Crossroads

Charles G. Allen
Brigham Young University - Provo

CRASHES IN THE VICINITY OF

MAJOR CROSSROADS

by

Charles G. Allen

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of a thesis submitted by

Charles G. Allen

This thesis has been read by each member of the following graduate committee and by majority vote has been found to be satisfactory.

Date	Grant G. Schultz, Chair
Date	Mitsuru Saito
Date	W. Spencer Guthrie

BRIGHAM YOUNG UNIVERSITY

As chair of the candidate's graduate committee, I have read the thesis of Charles G. Allen in its final form and have found that (1) its format, citations, and bibliographical style are consistent and acceptable and fulfill university and department style requirements; (2) its illustrative materials including figures, tables, and charts are in place; and (3) the final manuscript is satisfactory to the graduate committee and is ready for submission to the university library.

Date

Grant G. Schultz
Chair, Graduate Committee

Accepted for the Department

E. James Nelson
Graduate Coordinator

Accepted for the College

Alan R. Parkinson
Dean, Ira A. Fulton College of Engineering and Technology

ABSTRACT

CRASHES IN THE VICINITY OF

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Charles G. Allen

Department of Civil and Environmental Engineering

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Major crossroads are designed to facilitate the conflicting movements of numerous vehicles in a manner that is both safe and efficient. Accesses located within the functional areas of major crossroads add complication to intersection activity due to additional conflicts arising from ingressing and egressing movements at the accesses. In this research, the impact of accesses on crashes within major crossroad functional areas was analyzed. Specifically, the effects of access spacing within functional areas and access setback from intersections were addressed.

In order to conduct the analysis, the functional areas of 159 signalized major-arterial crossroads across the state of Utah were examined. A database was built containing the frequency, type, and severity of functional area crashes as well as the intersection and roadway characteristics within the functional area. Statistical analyses were conducted to determine the influence of accesses in intersection functional areas on functional area crashes.

The statistical analyses show that the existence of accesses within the functional areas was correlated with increased crashes and crash severity costs. In

particular, an increase in commercial access density was associated with increases in crash totals, crash rates, and rear-end crashes in intersection functional areas. The analyses also showed that study site intersections meeting Utah Department of Transportation (UDOT) corner clearance standards exhibited fewer right-angle crashes and lower crash severity costs. Finally, intersections that prohibited all unsignalized access had lower crash totals, crash rates, right-angle crash totals, and rear-end crash totals than intersections that allowed some unsignalized access.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1 Introduction

Traffic safety and operations are important factors in transportation systems. Signalized intersections represent transportation features of particular concern as they are designed to facilitate the conflicting movements of numerous vehicles in a manner that is both safe and efficient. Accesses in the vicinity of major crossroads provide further complication to intersection vehicular activity due to added conflicts arising from ingressing and egressing movements at the accesses. The purpose of this research is to analyze the impact on safety of access location and spacing on major-arterial crossroads.

1.1 Background

The spacing and location of accesses in the vicinity of major-arterial crossroads is subject to varied regulation among jurisdictions. The American Association of State Highway and Transportation Officials (AASHTO) *A Policy on Geometric Design of Highways and Streets* (AASHTO Green Book) recommends, "Ideally, driveways should not be located within the functional area of an intersection or in the influence area of an adjacent driveway" (AASHTO 2004, p. 729). The functional area extends both upstream and downstream of the intersection and should include any auxiliary lanes. The general definition of the intersection functional area includes: 1) the perception-reaction distance, 2) the braking distance, and 3) the queue storage necessary for an approaching vehicle to safely stop at the intersection (AASHTO 2004). The functional area of an intersection is critical in providing for safety and efficiency at signalized and unsignalized intersections.

The AASHTO Green Book also states, "Driveway terminals are, in effect, low-volume intersections; thus their design and location merit special consideration" (AASHTO 2004, p. 348). The regulation of access location at intersections and along arterial corridors is part of the overall principle of "access management." Access management is defined as "the systematic control of the location, spacing, design, and operation of driveways, median openings, interchanges, and street connections to a roadway" (TRB 2003, p. 3).

The implementation of access management principles and techniques has continued to be placed at the forefront of importance for the Utah Department of Transportation (UDOT). UDOT has established state highway access management guidelines as part of the *Accommodation of Utilities and the Control and Protection of State Highway Rights of Way* (UDOT 2006a). The Administrative Rule (R930-6), established as part of this document, aims to provide guidance to UDOT personnel in maintaining and preserving both existing and future capacity on the state roadway network. The Administrative Rule also provides guidance for design, operations, and project management to better implement access management techniques in both existing and future projects.

Past research has examined the effect of crossroads in the vicinity of interchanges and the impact these crossroads have on capacity and safety (Butorac and Wen 2004). Additional research has been conducted on the safety relationship between accesses, roadway corridors, and access management techniques in the state of Utah (Schultz and Braley 2007; Schultz and Lewis 2006). Previous research, however, has not explored the safety impact of accesses in the vicinity of major-arterial crossroads. The relationships between accesses, conflict points, and intersection need to be explored in order to develop guidelines for intersection setback at major-arterial crossroads.

1.2 Problem Statement

The purpose of this research is to analyze the impact on safety of access location and spacing on major-arterial crossroads. Specifically, the effects of access spacing within functional areas and access setback from intersections are addressed. The analyses involved examination of the functional areas of 159 major-arterial intersections across the state of Utah. A database was built containing the frequency, type, and severity of functional area crashes as well as the intersection and roadway characteristics within the functional area. One source of information was safety data collected using a unique yet proven tool available through UDOT. This tool is a geographic information system (GIS) enabled web-delivered data almanac. The databases within this data mining tool include crash data, bridge data, pavement condition data, and average annual daily traffic (AADT). Once the data were compiled, statistical analyses were then conducted to determine what role accesses in intersection functional areas have in describing functional area crashes.

The results of the project provide direction and guidance to UDOT on the design of driveway locations in the vicinity of major-arterial crossroads. In addition, UDOT benefits through a better understanding of the safety effects of access design and placement that can be passed on from the Planning Division through Traffic and Safety, Design, and Permitting.

1.3 Report Organization

This thesis is divided into a series of chapters and appendices. Chapter 2 presents a literature review of access management techniques at signalized intersections as well as a discussion of the calculation of intersection functional areas. Chapter 3 documents the data collection process for the 159 intersections within the study and the formation of the study database. The statistical analysis procedures are discussed in Chapter 4. Finally, Chapter 5 contains the results and conclusions of the report. Following Chapter 5 is a listing of references and a series of appendices which present the raw data collected during the study.

2 Literature Review

A literature review was conducted to examine methods of study and research findings regarding the impact of access location at major intersections. The effect of access management techniques on intersection safety and performance is discussed in Section 2.1. Section 2.2 then details how intersection access management is implemented in the state of Utah. Next, an examination of intersection functional areas and how they are calculated is evaluated in Section 2.3. Section 2.4 outlines crash analysis methods at intersections and describes the UDOT online crash almanac. Finally, Section 2.5 provides a summary of the literature review.

2.1 Access Management Techniques at Intersections

Effective access management is essential to preserving safety within intersections and the surrounding area (Antonucci et al. 2004). Access management is defined as "the systematic control of the location, spacing, design, and operation of driveways, median openings, interchanges, and street connections to a roadway" (TRB 2003, p. 3). Of the eight access management techniques identified in *National Cooperative Highway Research Program Report 420* (Gluck et al. 1999), seven apply directly to intersections. The following sections discuss what impacts the access management techniques of traffic signal spacing, unsignalized access spacing, corner clearance, medians, left-turn lanes, U-turns as an alternative to direct left-turns, and access separation at interchanges have on intersection safety.

2.1.1 *Traffic Signal Spacing*

The spacing of traffic signals has an impact on roadway safety and operations. Closely spaced signals can lead to increased vehicle delay, longer intersection queues, inefficient timing schemes, and an increase in the number of crashes (Gluck et al. 1999). Schultz and Braley (2007) found that shorter signal spacing may also lead to more severe crashes along arterial corridors.

2.1.2 *Unsignalized Access Spacing*

Unsignalized access spacing is the distance along a roadway between two adjacent accesses. The location and spacing of accesses has an important bearing on intersection performance. AASHTO specifies that driveways essentially operate as intersections and should be treated as such. AASHTO further states that crashes occurring at driveways are disproportionately higher than at other intersections; thus special design considerations are warranted for driveways (AASHTO 2004). Gluck et al. (1999) state that, at some locations, as many as one-half of the intersection crashes are driveway-related.

Although limited research is available in the literature regarding access spacing at intersections, several sources (Gluck et al. 1999; Schultz and Braley 2007; Stover and Koepke 2002; TRB 2003) detail the effect of access spacing on roadways in general, which may be applied to the roadways in the vicinity of intersections. Further discussion of the location of an access relative to an intersection is discussed in Section 2.1.3.

Unsignalized access spacing is related to roadway functionality. Multiple driveways along a roadway increase property accessibility but lower overall system mobility. Conversely, a facility with few accesses features high mobility and low accessibility. This relationship is characterized in Figure 2.1 (Stover and Koepke 2002). The number and location of accesses on a roadway should conform to the roadway's intended purpose and its functional classification (TRB 2003).

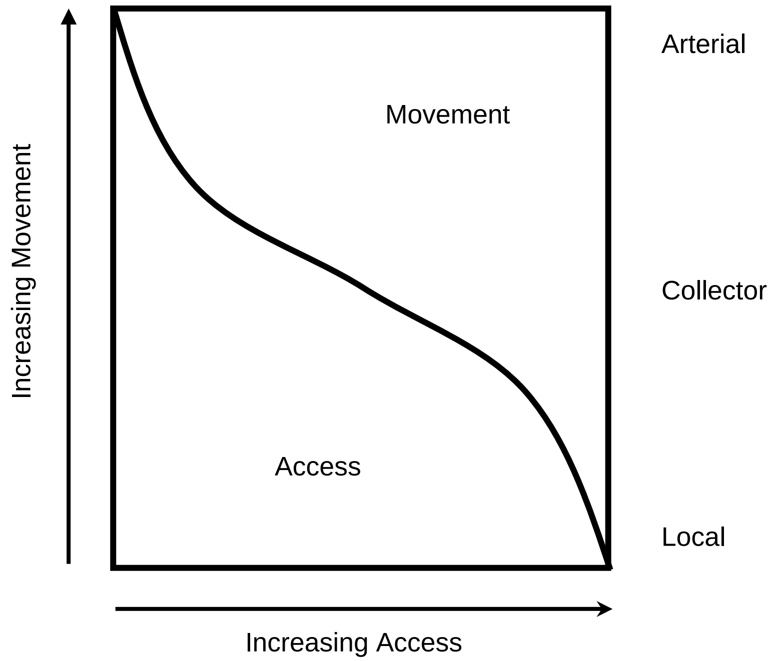

Figure 2.1 Access and movement (adapted from Stover and Koepke 2002).

Unsignalized access spacing also influences roadway safety. Increased access spacing provides greater separation between conflict points and simplifies turning maneuvers. This, in turn, generally leads to fewer crashes and lower vehicle delay. From a review of corridor access studies, Gluck et al. (1999) found that increasing access density from 10 to 20 accesses per mile increased the crash rate by about 30 percent. An additional increase to 40 accesses per mile increased crash rates by about 60 percent. In addition to higher crash rates, Schultz and Braley (2007) also determined that higher access density is correlated with increased crash severity.

The following sections discuss how unsignalized access design and land use influence the magnitude of the impact of an access on the adjacent roadway.

2.1.2.1 Access Design

Researchers report that access throat width and curb design has a relatively small effect on a vehicle's turning speed. Although vehicles may ingress and egress from an access at different speeds depending on the access configuration, the differences are not significant especially when comparing the relative speed between a turning vehicle and a trailing vehicle (Stover and Koepke 2002). The Institute of Transportation Engineers (ITE) *Traffic Engineering Handbook* states that the turning speed of an ingressing vehicle averages 6 to 13 miles per hour (mph). However, the turning vehicle's forward speed relative to the trailing through traffic is much less, averaging only about 1.5 to 2.5 mph. Thus, the speed differential between the vehicle turning into a driveway and any trailing vehicle is essentially the speed of the trailing vehicle (Koepke 1999).

Crash rates have been found to increase exponentially when speed differentials exceed 10 mph (Stover and Koepke 2002). Koepke (1999) reports that a right-turning vehicle begins to have a 10 mph speed differential at least 250 feet upstream of the driveway. Thus, driveway maneuvers can affect traffic conditions at great distances upstream leading to a presumed underreporting of driveway-related crashes.

2.1.2.2 Access Land Use

Schultz and Braley (2007) found that adjacent land use influences a driveway's impact on roadway safety. Their study evaluated the relationship between access density and crash severity. Areas exhibiting primarily commercial land use showed a much stronger positive relationship than residential sectors. Also, areas of commercial land use tended to experience higher crash rates. Because of their lower volumes, closely spaced residential driveways do not have as great an impact as closely spaced commercial driveways. Box (1998) determined that the number of driveways on a road segment is a less descriptive safety factor than the number of commercial units serviced by the driveways.

2.1.3 Corner Clearance

Corner clearance is defined as the distance between an intersection and the nearest driveway (Gluck et al. 1999). The *Access Management Manual* states that the upstream functional distance of an intersection should constitute the minimum corner clearance (TRB 2003). In addition, AASHTO asserts that, "Ideally, driveways should not be located within the functional area of an intersection or in the influence area of the adjacent driveway" (AASHTO 2004, p. 729). Intersection functional areas are discussed in detail in Section 2.3.

In a series of case studies, Gluck et al. (1999) found that corner clearance definitions and specifications vary from jurisdiction to jurisdiction. Some agencies defined corner clearance as the distance between the near edge of the intersection and the near edge of the driveway, while others measured from intersection centerline to driveway centerline. In the same series of studies, minimum corner clearances were reported to range from 16 feet to 300 feet with the majority being between 100 feet and 200 feet.

Inadequate corner clearance leads to numerous intersection operational and safety concerns, which are discussed in the following sections.

2.1.3.1 Effect on Operations

Inadequate corner clearance has been reported to reduce intersection capacities and saturation flow rates (Cheng-Tin and Long 1997). McCoy and Heinmann (1990) found that these saturation flow rate reductions increase with shorter corner clearances and higher driveway volumes. Also, a vehicle entering the traffic stream from a driveway can increase saturation headways by 1.0 to 1.9 seconds. Cheng-Tin and Long (1997) developed a minimum corner clearance equation calibrated to preserve saturation flow rates and keep accesses out of the intersection functional area.

2.1.3.2 Effect on Safety

Safety concerns that arise from inadequate corner clearances have been reported to include blocked driveway ingress and egress, conflicting and misinterpreted turning movements, inadequate weaving distances, and driveway queue spillover into the intersection (TRB 2003). Gluck et al. (1999) concluded that driveway obstruction is the most pervasive problem resulting from poor corner clearance and that intersections featuring multiple inadequate corner clearances are more likely to have higher crash rates. Antonucci et al. (2004) specified that access location should be governed by the probability of a queue from the intersection blocking the driveway. In a conflict point analysis, Cheng-Tin and Long (1997) found that when driveways are located too close to an intersection eight additional vehicular conflicts are introduced to the traffic system. The Transportation Research Board's *Access Management Manual* (TRB 2003) specifies that driveways should not be located within acceleration or deceleration lanes at intersections so as to reduce weaving conflicts.

2.1.4 Medians

Medians have an impact on roadway and intersection safety as they can restrict turning movements, remove turning vehicles from through traffic, and reduce head-on vehicle conflicts. Typical roadway median treatments include a two-way left-turn lane (TWLTL) and non-traversable or raised median. Researchers have found that roadways with a TWLTL exhibit fewer crashes than roadways with no median and that a raised median lowers the crash rate of a roadway even further. However, medians should provide appropriate left-turn and U-turn opportunities so as to not concentrate movements at traffic signals (Gluck et al. 1999). In a study of the effects of medians along corridors, Schultz and Lewis (2006) found that the presence of raised medians tended to shift crashes from mid-block locations to intersections while reducing overall crash severity.

2.1.5 Left-Turn Lanes

Left-turn lanes benefit intersections by removing turning vehicles from the through lanes and improving turning vehicle sight distance. As such, left-turn lanes have been found to reduce right-angle and rear-end crashes at signalized intersections (Gluck et al. 1999). Adequate storage is essential for left-turn lanes so as to not block through traffic. The design of turn-lane storage length is discussed in more detail in Section 2.3.4.

2.1.6 U-Turns as an Alternative to Direct Left Turns

Left-turns that are prohibited by a raised median may be accommodated by U-turns. Prohibited left-turns and their subsequent U-turns can affect intersections in two ways. First, left-turns restricted by a median may be converted into a U-turn at the nearest signalized intersection. The increased turning volumes at the intersection may lead to increased queue storage demand, longer left-turn phasing, and overall lower signal capacity. Second, U-turns that are located upstream and downstream from signalized intersections can either reduce left-turn demand at intersections or provide the opportunity for the intersections to prohibit left-turns completely. This may result in simplified signal phasing and increased intersection capacity (Gluck et al. 1999).

2.1.7 Access Separation at Interchanges

Major crossroads influence the operation and safety of freeway interchanges and their intersecting arterials. Gluck et al. (1999) found that when the crossroads of an arterial are located too close to the interchange, numerous traffic problems arise. Some of the problems include more frequent congestion, inadequate distance for weaving maneuvers, increased crashes, and more complex traffic signal timing. Sufficient access and crossroad spacing from interchange ramps is necessary for proper roadway functionality.

Spacing requirements for accesses and traffic signals adjacent to interchanges vary among jurisdictions. Butorac and Wen (2004) found that the prevailing spacing guidelines in use were derived from the AASHTO publication *A Policy on Design Standards – Interstate Systems* (AASHTO 1991), which specifies 100 foot spacing between interchange ramp and first access in urban areas and 300 foot spacing in rural areas. Jurisdictions that develop their own spacing requirements may base their standard on a number of criteria, including surrounding land use, roadway classification, interchange form, public or private access ownership, type of access, roadway cross section, speed, volume, signal cycle length, and economic impact.

In a survey of several state and provincial DOTs, Gluck et al. (1999) reported that existing access spacing standards ranged from 300 feet to 1,000 feet for rural areas and 100 feet to 700 feet for urban areas. The study also categorized ramp-arterial connections into two types: signalized connections and free-flow connections. Spacing from signalized connections should be governed by typical access and signal spacing criteria. However, spacing from free-flow connections should be determined in consideration of the distance needed to exit the ramp and safely maneuver into position to make a left-turn at the nearest downstream signal. This includes the distance required to merge with arterial traffic and weave into the left-turn bay, as well as the expected queue storage. Depending on the type of ramp connection, number of arterial lanes, left-turn volumes, and signal cycle length, appropriate signal spacing ranges are between 900 feet and 2,100 feet.

In a study of Virginia interchanges, Rakha et al. (2008) found that as the distance between the interchange off ramp and the nearest access increased, the road segment crash rate decreased. Crash rates were reduced by 50 percent when the access spacing increased from the AASHTO 300 foot minimum to 600 feet. In general, Rakha et al. found the AASHTO minimum spacing guidelines to be inadequate.

2.2 Intersection Access Management in Utah

Intersection access management techniques implemented in the state of Utah are set forth in Administrative Rule R930-6 (UDOT 2006a). Within the document, nine access management categories are identified, and signal, street, and access spacing requirements are presented for each category.

Signal spacing, street spacing, and access spacing standards are subject to separate measurement techniques. First, signal spacing is identified as the distance from "the centerline of the existing or future signalized intersection cross street to the centerline of the next existing or future signalized intersection cross street" (UDOT 2006a, p. 83). Second, street spacing is measured as "the distance from leaving point of tangent to receiving point of tangent" (UDOT 2006a, p. 84). Finally, access spacing is defined as "the distance from the inside point of curvature of the radius of an intersection or driveway to the inside point of curvature of the next intersection or driveway radius" (UDOT 2006a, p. 84). Table 2.1 summarizes Utah's spacing standards. As can be seen from the table, street and access spacing requirements range between 150 feet and 1,000 feet while signal spacing requirements range between 1,320 feet and 5,280 feet. Interchange and crossroad spacing requirements range between 500 feet and 1,320 feet (UDOT 2006a).

Table 2.1 UDOT Access Management Standards (adapted from UDOT 2006a)

Category		Description	Minimum Spacing (feet)			Minimum Interchange to Crossroad Spacing (feet)		
			Signal	Street	Access	"A"[1]	"B"[2]	"C"[3]
1	I	Freeway/Interstate	Interstate/Freeway Standards Apply					
2	S-R	System Priority Rural	5,280	1,000	1,000	1,320	1,320	1,320
3	S-U	System Priority Urban	2,640	Not Applicable		1,320	1,320	1,320
4	R-R	Regional Rural	2,640	660	500	660	1,320	500
5	R-PU	Regional Priority Urban	2,640	660	350	660	1,320	500
6	R-U	Regional Urban	2,640	350	200	500	1,320	500
7	C-R	Community Rural	1,320	300	150	Not applicable		
8	C-U	Community Urban	1,320	300	150			
9	O	Other	1,320	300	150			

1. Standard "A" refers to the distance from the interchange off-ramp gore area (point of widening) to the first right-in/out driveway intersection.
2. Standard "B" refers to the distance from the interchange off-ramp gore area (point of widening) to the first major intersection.
3. Standard "C" refers to the distance from the last right in/out driveway intersection to the interchange on-ramp gore area (point of widening).

2.3 Functional Areas of Intersections

Intersections are defined by both their physical area and their functional area. The functional area is larger than the physical area and encompasses any auxiliary lanes (AASHTO 2004). AASHTO states, "Ideally, driveways should not be located within the functional area of an intersection or in the influence area of an adjacent driveway"

(AASHTO 2004, p. 729). The *Access Management Manual* (TRB 2003) lists the preservation of the functional area of intersections and interchanges as one of its 10 access management principles. Figures 2.2 and 2.3 show a sample intersection physical area and functional area, respectively.

Since accesses should not be located within the intersection functional area, it becomes important to determine the functional area's actual linear extents. Functional areas can be divided into upstream and downstream components. Logically, the upstream functional distance has greater extents than the downstream functional distance (TRB 2003). Consequently, accesses on the approach side of an intersection leg require longer corner clearances than departure side accesses. Separate methodologies have been developed for the identification of upstream and downstream functional distances.

The literature suggests that the prevailing methodology for determining the upstream functional distance is to calculate the distance from the intersection required for an approaching driver to stop before reaching the rear of the intersection queue. AASHTO and the *Access Management Manual* divide the upstream functional distance into three components: 1) perception-reaction distance, 2) braking distance, and 3) queue storage (AASHTO 2004; TRB 2003). For intersections with auxiliary lanes, both Stover and Koepke (2002) and Koepke (1999) expand the above proposal into four components: 1) perception-reaction distance, 2) partial braking while moving laterally into the turn lane distance, 3) full braking distance, and 4) queue storage. This definition divides the braking distance into two components because, as drivers move into a turn lane, they decelerate more quickly than after having cleared the traffic stream. Figure 2.4 shows the upstream functional distance according to the four-step method.

The downstream functional distance calculation is a more straightforward procedure. Most sources suggest the downstream functional distance is governed by stopping sight distance. Drivers should have enough distance to avoid an access conflict upon exiting the intersection physical area (TRB 2003).

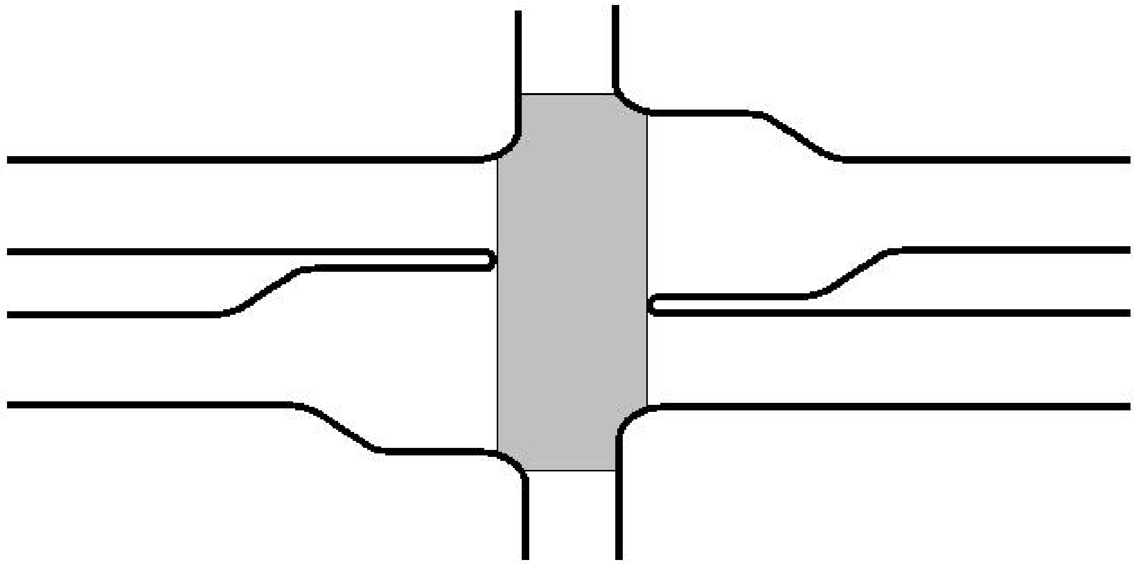

Figure 2.2 Intersection physical area (adapted from Stover and Koepke 2002).

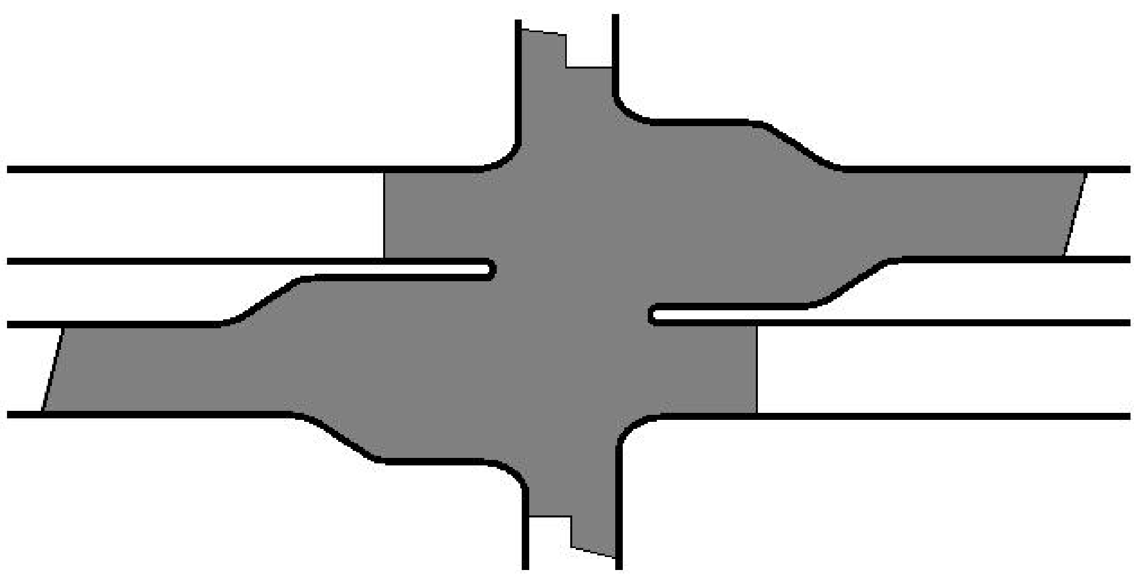

Figure 2.3 Intersection functional area (adapted from Stover and Koepke 2002).

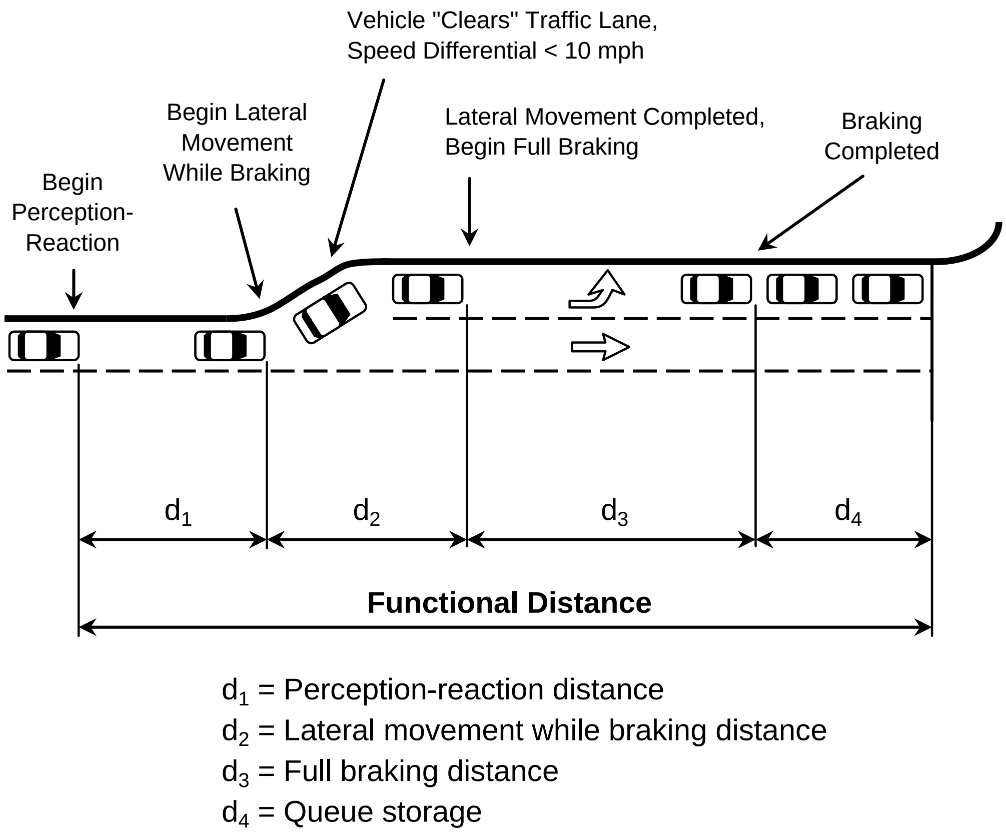

Figure 2.4 Upstream functional distance (adapted from TRB 2003).

Several sources calculate example upstream functional distances according to various roadway speeds. Each source determines the functional distance from the four-component methodology but uses slightly different driver behavioral characteristics and automobile performance parameters as inputs. Consequently, each source yields slightly different results. Table 2.2 compares sample upstream functional distances from various sources. As several different assumptions were evident from the literature regarding upstream functional distance calculation inputs, driver and vehicle parameters for the four functional distance components are discussed in detail in the following sections to help identify parameters for use in the research.

Table 2.2 Comparison of Upstream Functional Distance Calculations

Speed (mph)	Upstream Functional Distance Excluding Storage (feet)[1]		
	Source		
	Traffic Engineering Handbook[2]	Transportation and Land Development[3]	Access Management Manual[4]
30	215	250	270
35	270	320	--
40	335	395	450
45	405	475	--
50	485	570	610

1. Rounded to 5 feet; assumes vehicle has cleared through lane after moving laterally 9 feet while maintaining a speed differential of less than 10 mph.
2. Koepke 1999; "Limiting Conditions;" 1.0 second perception-reaction time; 4.5 ft/s^2 deceleration rate while moving laterally, 9.0 ft/s^2 full deceleration rate.
3. Stover and Koepke 2002; "Desirable Conditions;" 2.0 second perception-reaction time; 5.8 ft/s^2 deceleration rate while moving laterally, 6.7 ft/s^2 full deceleration rate.
4. TRB 2003; "Suburban Conditions;" 2.5 second perception-reaction time; 5.8 ft/s^2 deceleration rate while moving laterally, 6.5 ft/s^2 full deceleration rate.

2.3.1 Perception-Reaction Distance

The perception-reaction distance is a function of vehicle speed and the time required for a driver to recognize and respond to a stimulus. The equation for perception-reaction distance is shown in Equation 2.1 (Stover and Koepke 2002):

$$d_1 = 1.47vt \tag{2.1}$$

where: d_1 = perception-reaction distance (feet),

v = vehicle speed (mph), and

t = driver perception-reaction time (seconds).

Vehicle speeds are generally derived from the posted speed limit, the roadway design speed, or vehicle operating speeds as obtained from a local speed study. Driver perception-reaction times, however, are subject to much more variability within the literature.

The *Manual of Uniform Traffic Control Devices* (MUTCD) defines the time to perceive and react to a roadway sign as the sum of the perception, identification, emotion, and volition (PIEV) time components (FHWA 2003). General warning signs elicit PIEV times of several seconds, while complex warning signs may require 6 or more seconds of PIEV time (FHWA 2003). Perception-reaction times for drivers vary greatly according to age, decision complexity, and expectedness of event. Drivers require more time to react to unexpected events than expected events (AASHTO 2004). In a braking reaction times study, Johansson and Rumar (1971) found that unexpected braking reaction times averaged 35 percent longer than expected braking reaction times. Stover (1993) states that, for drivers familiar with the roadway, reaction time is essentially equivalent to the time required to prepare for braking.

Several researchers have attempted to determine suitable reaction times that encompass most driver performance levels. AASHTO states that, for stopping sight distance equations, a 2.5 second reaction time accounts for the abilities of most drivers (AASHTO 2004). The *Access Management Manual* (TRB 2003) identifies 1.5 second braking reaction times for urban and suburban areas and 2.5 seconds for rural areas. Stover and Koepke (2002) suggest that perception-reaction times may be 1.0 second or less for familiar drivers and more than 2.0 seconds for unfamiliar drivers. Setti et al. (2007) found that an 85th percentile perception-reaction time of 1.0 seconds was appropriate for traffic signal design and consistent with field research. Dewar (1999) calculated sample functional distances using a 2.0 second perception-reaction time for desirable conditions and a 1.0 second perception-reaction time for limiting conditions. Dewar further states, "…a perception-reaction time of 1.5 seconds is sufficient time for most drivers to respond, given a clear stimulus and a fairly straightforward situation" (Dewar 1999, p. 13). Chang et al. (1985) measured perception-reaction times as the period between the onset of a yellow interval at a traffic signal and the illumination of brake lights. The median and 85th percentile perception-reaction times were 1.1 seconds and 1.9 seconds, respectively. However, the researchers conjectured that vehicles at slower approach speeds and located further from the intersection are more likely to delay their braking decision. After accounting for such behavior, the median perception-reaction time was adjusted to 0.9 seconds.

Depending on the nature of vehicle maneuver, the roadway conditions, and assumptions about drivers, suggested perception-reaction times range from 1.0 second to 2.5 seconds. The 2.5 second perception-reaction time is the accepted value for roadway design because it encompasses such a large variety of conditions and circumstances. However, studies concentrated on urban traffic signal behavior recommend perception-reaction times between 1.0 second and 1.5 seconds.

2.3.2 Lateral-Movement-While-Braking Distance

Deceleration while moving into an auxiliary lane is a more complex maneuver than deceleration along a straight path. To accomplish this maneuver, drivers must navigate laterally along a turn-lane taper, evaluate the deceleration rate needed to stop at the end of the turn-lane queue, and be mindful of encroachment upon any trailing through vehicles. Most studies calculate this distance based on the assumption that drivers seek to clear the through lane before a 10 mph speed differential with through traffic is achieved. The equation for lateral movement and braking distance on level terrain is shown in Equation 2.2 (Hibbeler 2001; Stover and Koepke 2002):

$$d_2 = \frac{(1.47v_2)^2 - (1.47(v_2 - 10))^2}{2a_2} \tag{2.2}$$

where: d_2 = lateral movement while braking distance (feet),

v_2 = vehicle speed (mph), and

a_2 = vehicle deceleration rate while moving laterally (feet per second squared).

As with perception-reaction distance, vehicle speed for the lateral-movement-while-braking distance calculation is obtained from the posted speed limit, the roadway design speed, or vehicle operating speeds. The vehicle deceleration rate while moving laterally, however, is a more complex parameter.

Researchers suggest varying deceleration rates for the lateral-movement-while-braking-distance component of a functional area. Stover and Koepke (2002) calculated the maximum deceleration rate that will not produce a 10 mph speed differential before the vehicle has cleared the through lane to be 5.8 feet per second squared (ft/s^2). The calculation assumes lateral clearance is achieved after 9.0 feet of lateral movement with lateral velocities between 3.0 feet per second (ft/s) and 4.0 ft/s. In an earlier publication, Stover (1993) specified lateral movement at 4.0 ft/s and calculated the maximum forward deceleration to be 7.0 ft/s^2. Pline (1996) suggested a lateral speed of 4.0 ft/s while navigating a turn-pocket taper is appropriate for driver comfort. Schurr et al. (2003) used a 4.0 ft/s lateral velocity to calculate a maximum deceleration rate of 3.7 ft/s^2 in order to avoid the 10 mph speed differential. Koepke (1999) used a 3.5 ft/s^2 deceleration rate for desirable conditions and a 4.5 ft/s^2 deceleration rate for limiting conditions in a functional area calculation. In summary, no common consensus on the deceleration rate for a lateral movement maneuver is offered within the literature, but suggestions range between 3.5 ft/s^2 and 7.0 ft/s^2.

2.3.3 Full Braking Distance

Deceleration after lateral transition into the turn lane is a more straightforward calculation than braking with lateral movement. The equation for full braking distance on level terrain is shown in Equation 2.3 (Stover and Koepke 2002):

$$d_3 = \frac{(1.47v_3)^2}{2a_3} \qquad (2.3)$$

where: d_3 = full braking distance (feet),

v_3 = vehicle speed after lateral movement and deceleration (mph), and

a_3 = full vehicle deceleration rate (ft/s^2).

Vehicle speeds for the full braking distance equation are commonly assumed to be 10 mph less than the speed value chosen for the perception-distance equation and the lateral-movement-while-braking distance equation. However, as with the lateral-movement-while-braking distance equation, no prevalent deceleration rate is offered for functional distance calculations.

Full braking deceleration rates have been studied more extensively than deceleration rates with lateral movement. AASHTO (2004) recommends a deceleration rate of 11.2 ft/s^2 for stopping sight distance calculations. Glauz and Harwood (1999) define 10.0 ft/s^2 as a comfortable deceleration rate for vehicle passengers and 16.1 ft/s^2 as a maximum passenger car deceleration rate that occurs during wheel-locked skid conditions. For determining functional distances, Koepke (1999) utilizes full braking deceleration rates of 6.0 ft/s^2 for desirable conditions and 9.0 ft/s^2 for limiting conditions. Chang et al. (1985) suggests a deceleration rate of 10.5 ft/s^2 for level and upgrade conditions and 10.0 ft/s^2 for downgrade conditions. Wortman et al. (1985) observed the behavior of first to stop vehicles for traffic signals at the onset of the yellow interval. The resulting deceleration rates ranged from 8.3 ft/s^2 to 13.2 ft/s^2 with an average of rate 11.6 ft/s^2. Gates et al. (2007) identified 50th and 85th percentile deceleration rates of 9.9 ft/s^2 and 12.9 ft/s^2, respectively, for vehicles stopping for a traffic signal.

In summary, suggested full braking deceleration rates range between 6.0 ft/s^2 and 16.1 ft/s^2. The AASHTO recommendation of 11.2 ft/s^2 is the accepted value for roadway design, and studies focused on reactions to red lights recommend similar values.

2.3.4 Queue Storage

Auxiliary lane queue storage should be sufficient to accommodate the number of vehicles likely to accrue in a given study interval. At signalized intersections, several factors influence the auxiliary-lane queue storage length, including signal cycle length, signal phase plan, turning volumes, opposing through volumes, and vehicle type distribution. At a minimum, the storage length should be sufficient to accommodate two passenger cars (AASHTO 2004). At an average vehicle storage distance of 25 feet, this translates to a 50-foot minimum storage length. Because storage requirements can vary considerably between right-turn lanes and left-turn lanes at the same intersection, the longer of the two should be used for the upstream functional area determination (TRB 2003).

Many methods have been developed to estimate appropriate turn-bay storage length. Qi et al. (2007) categorized left-turn bay length determination methods into three groups: 1) rule-of-thumb methods, 2) analysis-based methods, and 3) simulation-based methods. Stover and Koepke (2002) also identify a category for the Leisch nomograph method. The following sections discuss the storage length determination methods mentioned above.

2.3.4.1 Rule-of-Thumb Methods

One commonly used rule of thumb is to estimate the critical turn-bay queue length by doubling the average queue length. AASHTO (2004) states that a turn-bay length one-half to two times the average demand will be sufficient to accommodate occasional surges. In a study of protected left-turns at signalized intersections, Gattis (2000) found that the "double-the-average" rule often over-predicted the necessary queue length. However, when three-quarters of the queues carried over to the subsequent cycle, the double-the-average rule was more accurate.

A second rule of thumb is to estimate the appropriate left-turn storage by multiplying 1.0 foot by the number of left-turning vehicles per hour for cycles of 60 seconds or less. For cycles between 60 seconds and 120 seconds, the number of left-turning vehicles per hour is multiplied by 2.0 feet (Stover and Koepke 2002).

2.3.4.2 Analysis-Based Methods

Most analysis-based methods operate on the principle of successfully storing 95 percent of all queue demands. Often, a Poisson distribution is utilized to identify the 95th percentile queue from the average queue demand. Gattis (2000) compared distribution methods and found that a binomial distribution performed just as well, or better, than Poisson's distribution. Kikuchi et al. (1993) developed a methodology based on the probability of one of two failure scenarios occurring: 1) overflow of the left-turn lane or 2) blockage of the turn-lane entrance by queued through vehicles. Appropriate turn-bay lengths are calculated for each failure condition, and the longer of the two is recommended for use. Qi et al. (2007) developed an analysis model that considers the left-turning vehicle arrival rate and the number of queued vehicles that carry over from the previous cycle. The model produced results similar to observed queue lengths.

Left-turn storage may also be calculated from the formula shown in Equation 2.4 (Stover and Koepke 2002):

$$L_{LT} = \frac{V}{N} k_{LT} s \tag{2.4}$$

where: L_{LT} = left-turn storage length (feet),

V = left-turn volume (vehicles per hour),

N = cycles per hour,

k_{LT} = a constant, usually assumed to be 2.0, and

s = average storage length per vehicle (feet).

Equation 2.5 (Stover and Koepke 2002) is a similar method for right-turn lanes:

$$L_{RT} = \frac{V}{N} \frac{R}{C} k_{RT} s \tag{2.5}$$

where: L_{RT} = right-turn storage length (feet),

R = length of red phase (seconds),

C = cycle length (seconds), and

k_{RT} = random arrival factor, k = 1.5 when right-turn on red is permitted, k = 2.0 when right-turn on red is not permitted.

In cases where dual turn lanes are implemented, the equation output lengths may be divided by 1.8.

2.3.4.3 Simulation-Based Methods

Simulation-based methods utilize statistical and traffic simulation programs to identify critical queue lengths. The programs can consider various types of conditions, such as left-turn protection, signal phasing, and turning volumes. However, the simulations require extensive calibration, and outputs are only applicable to sites similar to the sites used to generate the input conditions (Qi et al. 2007).

2.3.4.4 Liesch Nomograph Method

Leisch developed a nomograph that determines turn-bay length as a function of turning volume, cycle length, and percent heavy vehicles. The nomograph outputs both the 95th percentile and 85th percentile queue length to be used as the desirable and minimum turn-bay design lengths, respectively (Stover and Koepke 2002).

2.3.4.5 Comparison of Methods

Table 2.3 compares recommended left-turn storage lengths according to a rule-of-thumb method, Equation 2.4, and the Liesch nomograph (Stover and Koepke 2002). As can be seen from the table, each method produces similar results.

Table 2.3 Comparison of Left-Turn-Lane Storage Methods

Recommended Left-Turn Storage Length (feet)[1]		
Method		
Rule of Thumb[2]	Equation[3]	Liesch Nomograph[3]
200	175	175

1. Based on 200 left-turning vehicles per hour, cycle length of 60 seconds, 5 percent heavy vehicles, and average vehicle storage length of 25 feet; rounded up to the nearest 25 feet.
2. Stover and Koepke 2002
3. Stover and Koepke 2002; desirable conditions

2.4 Crash Analysis at Intersections

Intersections are a significant source of operational and safety concerns within transportation systems. A nationwide study of intersection crashes found that over 20 percent of all roadway fatalities between 1997 and 2004 occurred at intersections. Eighty-three percent of these fatalities occurred within the limits of the intersection, while 17 percent occurred in the intersection-related area, which includes the intersection approaches and exits. In Utah, for the same time period, 6 percent of all fatalities occurred at signalized intersections, while 5 percent occurred at stop-controlled intersections (Subramanian and Lombardo 2007).

Crash data may be analyzed through a variety of methods. The following sections discuss how crash data are evaluated through crash rates, crash severity, and the UDOT Data Almanac.

2.4.1 Crash Rates

Crash rates provide a way to compare crash totals from site to site while accounting for roadway parameters such as traffic volumes or length of road. The standard intersection crash rate equation measures crashes per year per million entering vehicles (MEV) and is shown in Equation 2.6 (Fricker and Whitford 2004):

$$R_{INT} = \frac{C_{INT}}{365 \times V_E} 1{,}000{,}000 \tag{2.6}$$

where: R_{INT} = intersection crash rate (crashes per year per MEV),

C_{INT} = intersection crashes per year, and

V_E = total entering volumes of all intersection legs (vehicles per day).

Another commonly used equation measures crashes along a road segment in crashes per year per million vehicle miles (MVM), as outlined in Equation 2.7 (Fricker and Whitford 2004):

$$R_{SEG} = \frac{C_{SEG}}{365 \times V_R \times L} 1{,}000{,}000 \tag{2.7}$$

where: R_{SEG} = road segment crash rate (crashes per year per MVM),

C_{SEG} = road segment crashes per year,

V_R = two-way roadway volume (vehicles per day), and

L = length of road segment (miles).

2.4.2 Crash Severity

Crash severity represents the most serious injury of all injuries sustained in a crash event. The National Safety Council (NSC) (2007) classifies crash severity into five categories: fatal, incapacitating injury, non-incapacitating evident injury, possible injury, or non-injury (NSC 2007). One way to evaluate crash severity at a location is to total the costs associated with the severity level of each crash. In an implementation of the Federal Highway Administration's (FHWA) Highway Safety Improvement Program (FHWA 2005), UDOT identified crash costs for each severity level in the state of Utah (UDOT 2006b). Crash costs are updated on a yearly basis, and 2008 costs were obtained from UDOT personnel (Michael Kaczorowski, personal communication, June 26, 2008). Table 2.4 summarizes the UDOT 2008 crash costs according to severity.

Table 2.4 UDOT Crash Costs 2008

NSC Crash Category	UDOT Cost
Non-injury	\$ 4,400
Possible injury	\$ 42,000
Non-incapacitating evident injury	\$ 80,000
Incapacitating injury	\$ 785,000
Fatal	\$ 785,000

2.4.3 UDOT Data Almanac

The UDOT Data Almanac maintains a database of crash types and locations along Utah state routes and local federal-aid routes. The purpose of the database is to provide rapid access and analysis of state transportation information. In addition to crash data, the database also includes pavement condition information, AADT, and bridge data (Anderson et al. 2006).

The database is a GIS based web program that allows the user to construct custom queries to obtain specific crash information. Data are presented in tabular formats that can be copied into spreadsheets for reference or further analysis. The database is designed to facilitate data analysis in six ways (Anderson et al. 2006):

1. Custom tables and reports are created with only selected parameters, excluding unneeded data. This simplifies the analysis by focusing on what is important to each individual user.
2. Placing the data on a "smart map" allows the decision-maker to visually identify hot spots or deficient areas. The analysis can be further refined by extracting selected information from the map as needed.
3. Simple statistical processes can be applied to the data by location using "Fixed Segment," "Floating Segment," or "Cluster" analysis.
4. Providing information from multiple databases in one web site allows users to conduct "loose" integration of the data. Information extracted through a series of queries from different data sources can be saved into a single spreadsheet for analysis.
5. Decision-makers will have more time to analyze the data since less time is required to gather and compile the information. This will enhance the identification of problem areas, program delivery, and improved designs.
6. The system is designed to quickly download data for performance measurement. The effectiveness of improvements can be monitored over time in an efficient manner.

2.5 Summary of Literature Review

In this chapter, a review of study methods and research regarding the impact of access location at major intersections was presented. The effect of access management techniques at intersections, in general, and the intersection access management standards for the state of Utah were examined. A discussion regarding intersection functional areas was given. Finally, intersection crash analysis methods were outlined, and the UDOT Data Almanac was documented. The next chapter documents the data collected in preparation for an analysis of intersection access location and safety.

3 Data Collection

In order to evaluate the impact on safety of accesses within intersection functional areas, intersection and crash information was gathered from 159 intersections across the state of Utah. This chapter discusses the purposes and methodology for obtaining the data. Section 3.1 documents the site selection process for the 144 analysis intersections and 15 reference intersections. The intersection data and crash data obtained from each site are presented in Sections 3.2 and 3.3, respectively. Finally, Section 3.4 provides a summary of the data collection process.

3.1 Site Selection

The first step in the site selection process was to identify the study corridors from which the analysis intersections would be chosen. Corridors were identified in cooperation with UDOT personnel, who provided a sample of corridors with a broad range of traffic volumes. The selected roadways included both principal and minor arterials from all areas of Utah and from each of the four UDOT regional jurisdictions.

The second site selection step was to evaluate the signalized intersections along each corridor and identify the locations suitable for analysis. The goal of the intersection selection was to create a consistent and reliable dataset by filtering out the intersections with abnormal characteristics or unusual designs. Consistent design among intersections helped to simplify the statistical comparisons and reduce the potential for influential outliers in the data. Intersections were deemed suitable for analysis if they satisfied the following criteria:

- Intersection is signal controlled
- Intersection must have four, at-grade, two-way street approach legs

- Approach legs must not be private access connections unless the private access serves a commercial development and features striping or median separation between ingress and egress lanes
- Corridor is classified as UDOT Access Category 4 through Category 9 (UDOT 2006a)
- Surrounding roadway did not undergo construction during the analysis period

In total, 144 intersections from 20 roadway corridors were chosen for analysis. Figure 3.1 shows the distribution of study intersections by UDOT regional jurisdictions. Forty-one of the intersections are located in UDOT Region 1 within Box Elder, Cache, Davis, and Weber counties. Forty-six intersections are located in UDOT Region 2 and within either Tooele County or Salt Lake County. Thirty-seven intersections are located in UDOT Region 3 within Uintah, Utah, and Wasatch counties. Finally, 20 intersections are in UDOT Region 4 within Iron County and Washington County. A full listing of study site route numbers and street names can be found in Appendix A. A listing of study site UDOT regional jurisdictions can be found in Appendix B.

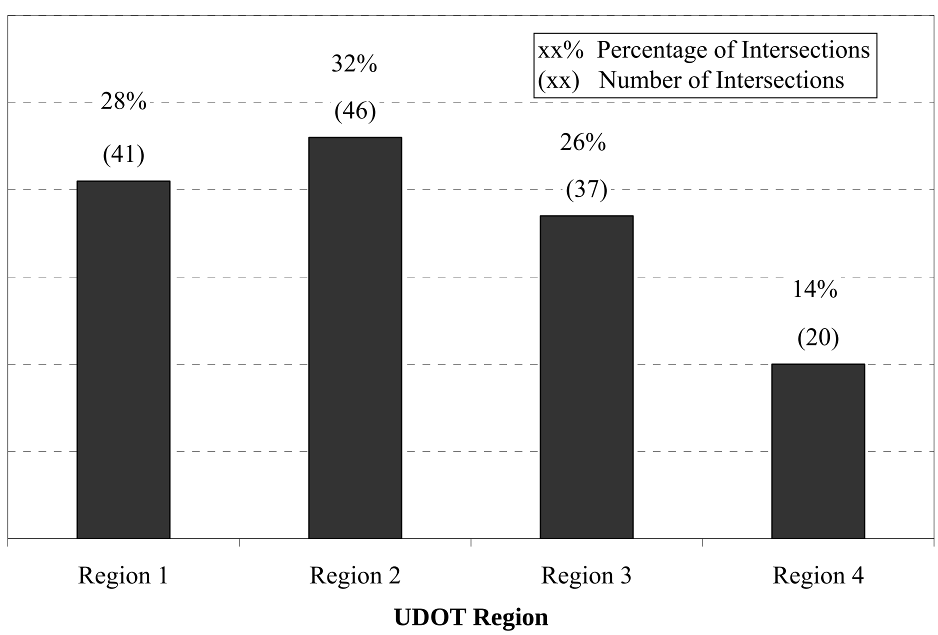

Figure 3.1 Distribution of study intersections among UDOT regions.

In addition to the study intersections, two reference corridors, Bangerter Highway and Van Winkle Expressway, were selected for analysis to provide a comparison with sites that are not permitted to have any unsignalized access. Both corridors are designated as "System Priority Urban" according to the UDOT Access Classification system discussed in Section 2.2. Reference intersections were selected from the reference corridors using the same screening criteria as the study intersections with the exception of the access classification requirement. After the intersection selection process was conducted, 15 sites were available for the reference analysis. All reference intersections are located in Salt Lake County within UDOT Region 2.

The analysis period was selected as the most recent three years of available crash data. In general, 2003 to 2005 was the analysis period for most study intersections. Corridors that did not have 2005 crash data available were evaluated from 2002 to 2004. Appendix B lists each study site analysis period.

3.2 Intersection Data

Study intersections were thoroughly evaluated in order to gather a large set of potential explanatory variables. Because crash data patterns may be influenced by a number of parameters, obtaining as much information as possible was important to account for all potentially influential intersection factors. This section discusses the purpose and methodology utilized to acquire intersection location, attribute, geometry, functional area, and access data.

3.2.1 Location

Within the UDOT Data Almanac, intersections and crashes are assigned a highway milepost location at a precision of hundredths of a mile. In order to compare crash locations with intersection features, the intersection milepost locations were obtained from the crash database utilizing the "Points of Interest" search tool (Anderson et al. 2006). At sites where the minor street was also a state route or a local federal-aid route, the intersection location according to the minor street milepost system was also obtained. Milepost locations were then cross-checked with those available in the UDOT publications *Traffic on Utah Highways 2004* (UDOT 2004), *Traffic on Utah Highways 2005* (UDOT 2005a), and UDOT Highway Reference Information (UDOT 2008a). Further comparisons were made to crash cluster locations, which are discussed in detail in Section 3.3.1.

3.2.2 Attributes

Several types of attribute data were acquired from each study site, including access classification, functional classification, AADT, left-turn protection, speed limit, proximity to freeway interchange, and median type. This section discusses the process and reasoning for obtaining each attribute type.

3.2.2.1 Access Classification

Utah state highway access management standards are assigned to roadways according to an access management classification system, as summarized in Table 2.1 in Section 2.2. Among other specifications, these requirements set forth the signal spacing, access spacing, and corner clearance standards for each corridor. Intersection access classifications were obtained in order to provide comparisons as to whether adherence to the access management standards influences intersection crash rates.

The *State Highway Access Category Inventory* (UDOT 2006c) lists access classification break points for each state highway according to milepost. The previously acquired intersection milepost locations were used to determine the access category for each intersection. Figure 3.2 shows the distribution of study sites by access category. As illustrated in the figure, the majority of study sites were either the Regional Priority Urban or the Regional Urban classification. Only 17 percent of the data fell within the remaining two classifications. A full listing of study site access classifications can be found in Appendix A.

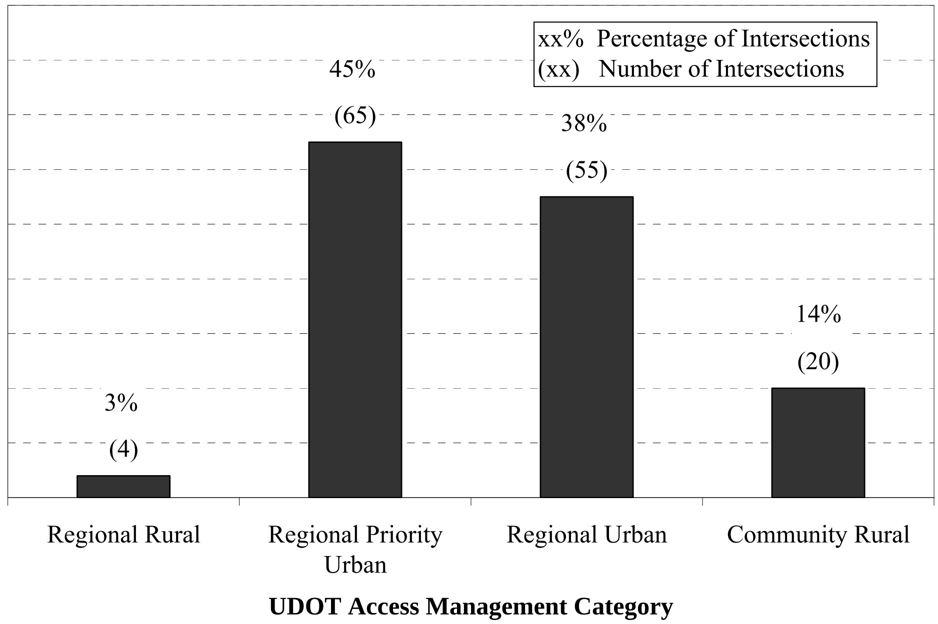

Figure 3.2 Distribution of study intersections by access management category.

3.2.2.2 Functional Classification

As previously discussed in Section 2.1.2, roadway functional classification is directly related to accessibility. Roads designed for heavier throughput generally have fewer accesses. Conversely, roads intended to provide accessibility are allowed additional accesses at the expense of vehicle throughput and progression.

The functional classification for each intersection was determined from online functional classification maps available from UDOT (UDOT 2005b). Figure 3.3 shows the distribution of study intersections by functional class. As can be seen from Figure 3.3, 83 percent of intersections are principal arterial corridors, while 17 percent are classified as minor arterial corridors. A full listing of study site functional classifications can be found in Appendix A.

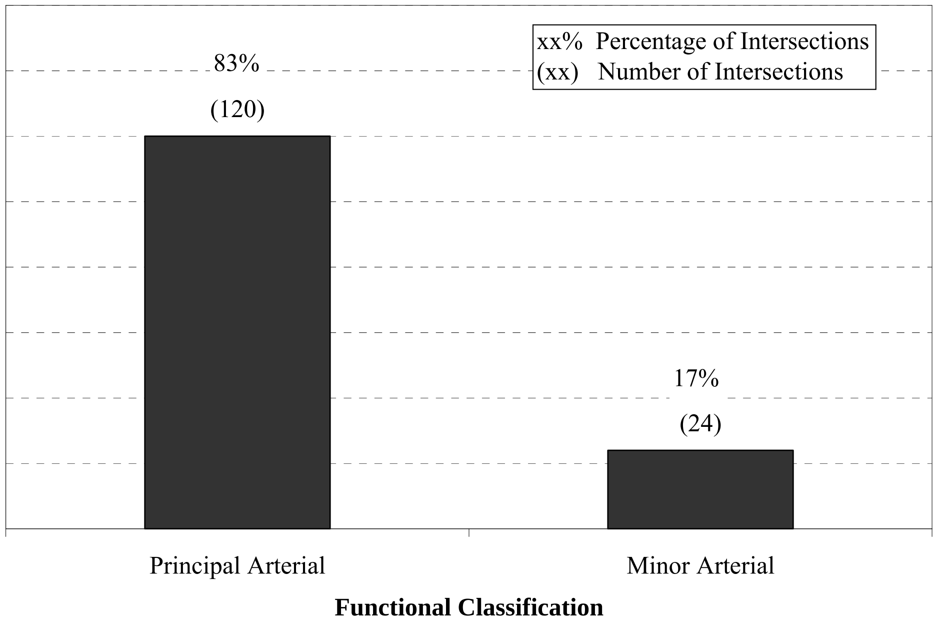

Figure 3.3 Distribution of study intersections by functional classification.

3.2.2.3 AADT

Traffic volumes provide a way to normalize crash reporting across locations. Sites with higher volumes are generally expected to experience more crashes due to the increased potential for traffic conflicts. Additionally, roadway volume is an input for the intersection crash rate equation and the road segment crash rate equation discussed in Section 2.4.1.

UDOT documents three-year traffic volumes for road segments along all state highways and local federal-aid roads in its annual publication, *Traffic on Utah Highways* (UDOT 2008b). Since all study intersections occurred along state highway corridors, AADT volumes were available for all study site major-street approach legs. Minor-street volumes, however, were obtainable only for sites in which the minor street was another state highway or local federal-aid route.

AADT volumes were acquired for each study year and computed into three-year averages. In most cases, an intersection listed separate AADT volumes for corresponding approaches. Both approach volumes were then averaged together to obtain the roadway average entering volume. A full listing of average intersection AADT volumes for each study site can be found in Appendix C.

Figure 3.4 shows the distribution of major-street entering volumes. The majority of study sites exhibited AADT volumes between 20,000 vehicles per day (vpd) and 40,000 vpd. Fifteen percent of study sites featured AADT volumes greater or equal to than 40,000 vpd, while only 11 percent had AADT volumes less than 10,000 vpd.

When available, minor-street average approach volumes were also obtained. The distribution of minor-street approach volumes is shown in Figure 3.5. As can be seen from Figure 3.5, minor streets exhibited much lower volumes than major streets. Seventy percent of study sites with minor-street data available featured AADT volumes less than 20,000 vpd.

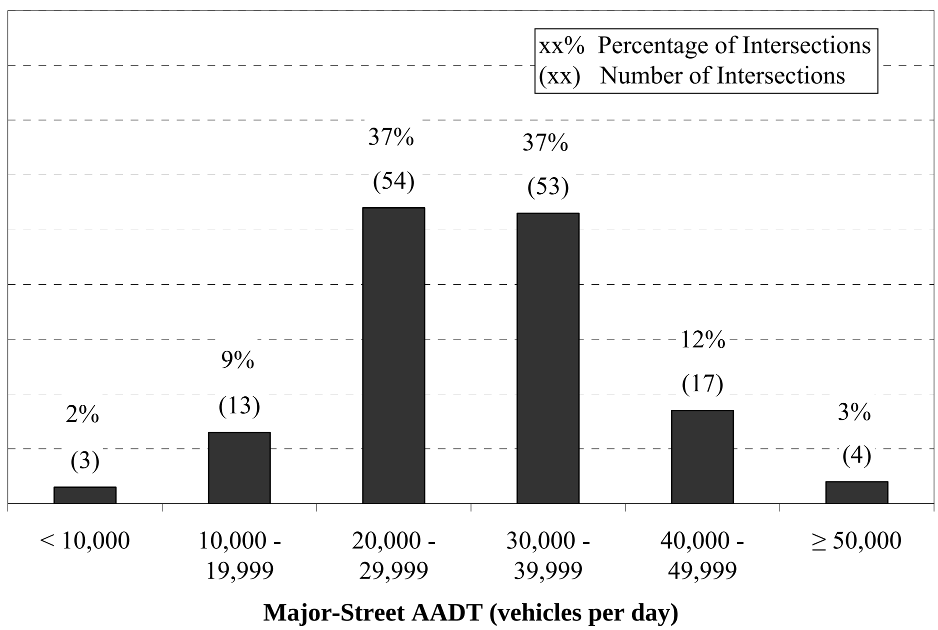

Figure 3.4 Distribution of study intersections by major-street AADT.

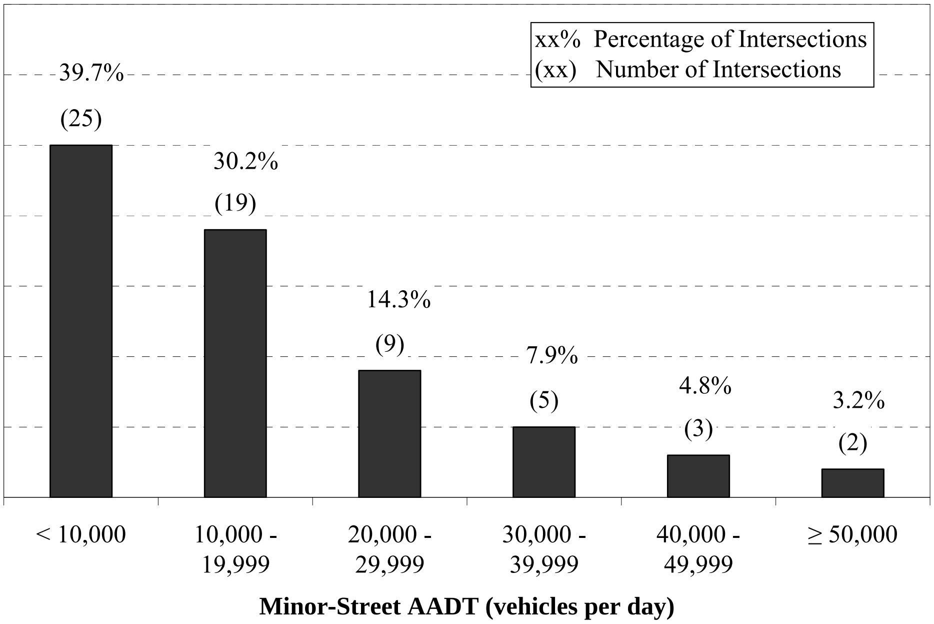

Figure 3.5 Distribution of intersections by minor-street AADT (when available).

3.2.2.4 Left-Turn Protection

Left turns at intersections are a complicated maneuver as drivers must judge opposing vehicle speeds, select an appropriate gap, and be mindful of the traffic signal phasing. Increasing left-turn volumes lead to increased traffic conflicts, and thus increased crash potential. Since obtaining turning movement counts for each of the 159 study and reference intersections was not feasible, the presence and type of left-turn phasing for both corridor and crossroad approaches was evaluated. UDOT implements left-turn signal phasing when left-turn volumes and/or left-turn crashes reach critical thresholds (UDOT 2006d). For example, protected-permitted phasing may be implemented when one of the following four warrants is satisfied:

- Warrant I: Left-turn volume exceeds 100 vehicles per hour and the left-turn demand to capacity ratio to exceeds 0.90 for one hour of the day
- Warrant II: Left-turn, three-year crash average exceeds 0.80 crashes per million vehicles

- Warrant III: Both Warrant I and Warrant II are met at 80 percent of their threshold levels
- Warrant IV: Left-turn volumes frequently exceed storage and disrupt through traffic

Because left-turn protection is implemented only when one of the four warrants is satisfied, left-turn protection can be an indicator of left-turning volumes, left-turn crash rates, and storage capacity.

Information about the presence and type of left-turn phasing was acquired through examination of intersection lane configurations and traffic signal heads. Aerial imagery from Google Earth (Google 2008a), Yahoo Maps (Yahoo 2008), and Windows Live Maps (Microsoft 2008) and street imagery from Google Maps (Google 2008b) and UDOT Roadview Explorer (UDOT 2008c) were used to evaluate left-turn phasing. Left-turn protection was recorded as one of three types: 1) permitted, 2) protected-permitted and, 3) protected. Figure 3.6 shows the distribution of left-turn protection on major-street approaches.

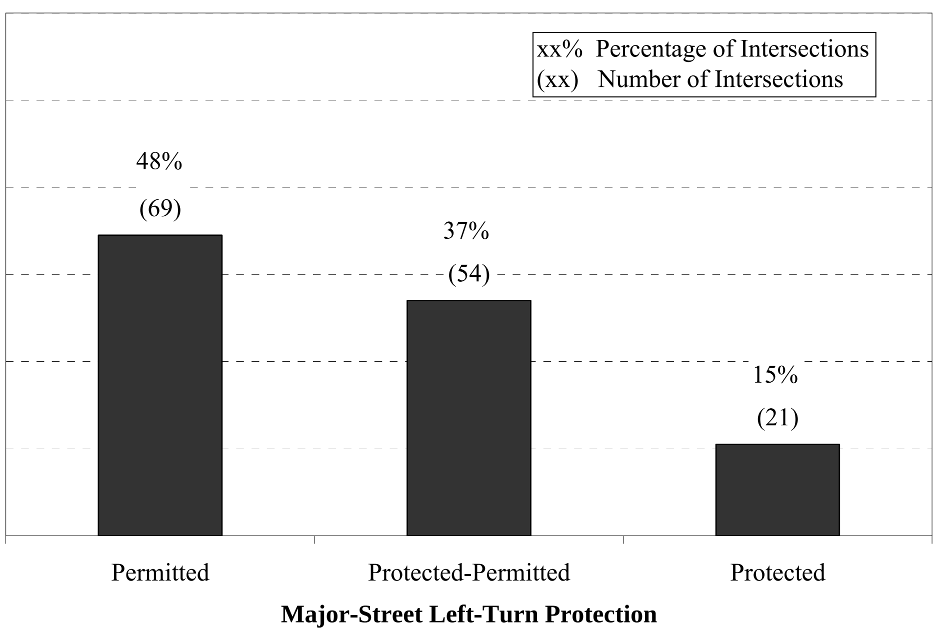

Figure 3.6 Distribution of study intersections by major-street left-turn protection.

Figure 3.7 shows the distribution of left-turn protection on minor-street approaches.

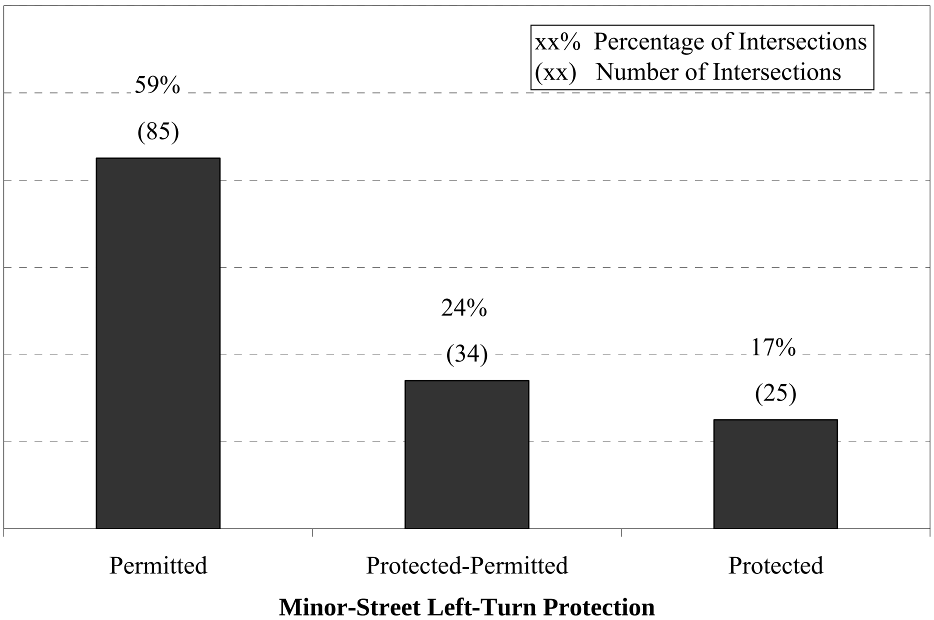

Figure 3.7 Distribution of study intersections by minor-street left-turn protection.

As can be seen from Figure 3.6 and Figure 3.7, intersection distribution decreases with the degree of left-turn protection for both major and minor streets. However, the proportion of minor streets with permitted left-turn phasing was much greater than for major streets. A full listing of study site left-turn protection phasing can be found in Appendix B.

3.2.2.5 Speed Limit

Posted speed limits provide an estimation of typical roadway operating speeds. Because faster moving vehicles require longer braking distance, speed is likely to have an impact on crash rates. Intersection posted speed limits were obtained for each major-street approach by examining street imagery from Google Maps (Google 2008b) and UDOT Roadview Explorer (UDOT 2008c). In almost every case, the posted speed limit on each approach was identical. At the one site with differing approach speed limits, the larger of the two was used for analysis. Figure 3.8 shows the speed limit distribution among intersections. As shown in Figure 3.8, most intersections featured speed limits between 35 and 45 mph. The 50 mph speed limit category was the smallest category with only two total intersections. A full listing of study site speed limits can be found in Appendix B.

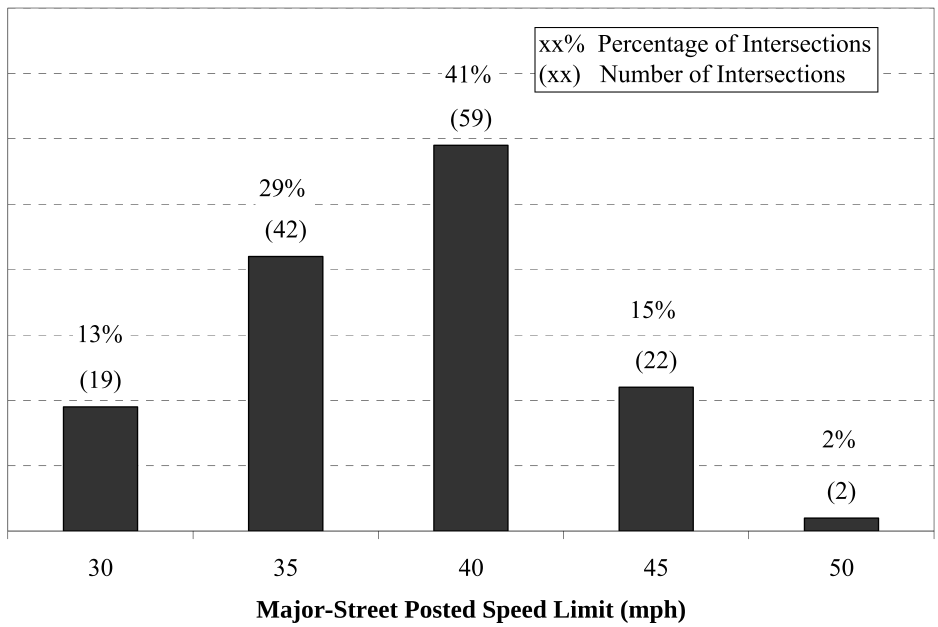

Figure 3.8 Distribution of study intersections by major-street posted speed limit.

3.2.2.6 Proximity to Freeway Interchange

In Utah, state highway intersections that are adjacent to freeways are subject to different access management standards (UDOT 2006a). Generally, spacing between freeway exit/entrance ramp termini and accesses should be longer because of the merging and weaving maneuvers associated with navigating into the appropriate turn lanes. UDOT access management standards require that major intersections be located at least one-quarter mile from freeway ramp termini (UDOT 2006a). Therefore, study intersections that were the first traffic signal within one-quarter mile of a freeway exit/entrance ramp were flagged as being adjacent to a freeway for the analysis. Figure 3.9 shows the distribution of study sites adjacent to freeways. As can be seen from the graph, only seven study intersections, or 5 percent of all intersections, were classified as adjacent to a freeway. A full listing of study site proximities to freeway interchanges can be found in Appendix C.

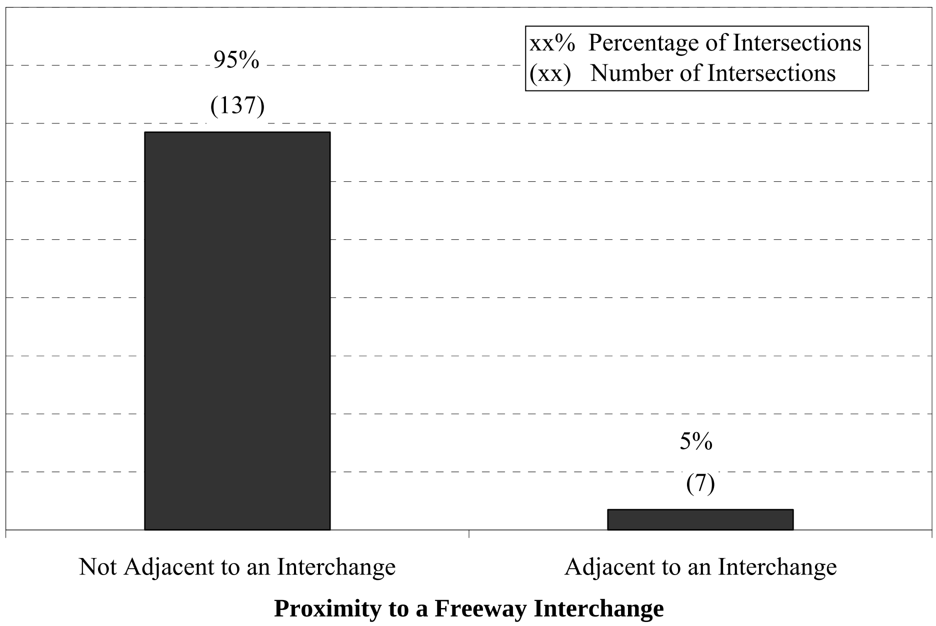

Figure 3.9 Distribution of study intersections by proximity to a freeway interchange.

3.2.2.7 Median Type

Medians have generally been shown to have an impact on roadway safety. Gluck et al. (1999) found that roadways with a TWLTL are characterized by fewer crashes than roadways with no median and that a raised median lowers the crash rate of a roadway even further. Using aerial imagery from Google Earth (Google 2008a), Yahoo Maps (Yahoo 2008), and Windows Live Maps (Microsoft 2008) and street imagery from Google Maps (Google 2008b) and UDOT Roadview Explorer (UDOT 2008c), the median type on each study site approach was categorized. The three median categories were 1) raised median, 2) TWLTL, or 3) no median. In some instances, intersection approaches contained a raised median for only the portion of the roadway closest to the intersection. For such cases, the approaches were assigned to the raised median category. Figure 3.10 shows the distribution of median type by intersection major-street approach. The majority of approaches had a TWLTL, while most of the remaining approaches contained a raised median. Only three approaches, or 1 percent, had no median. A full listing of study site median types can be found in Appendix B.

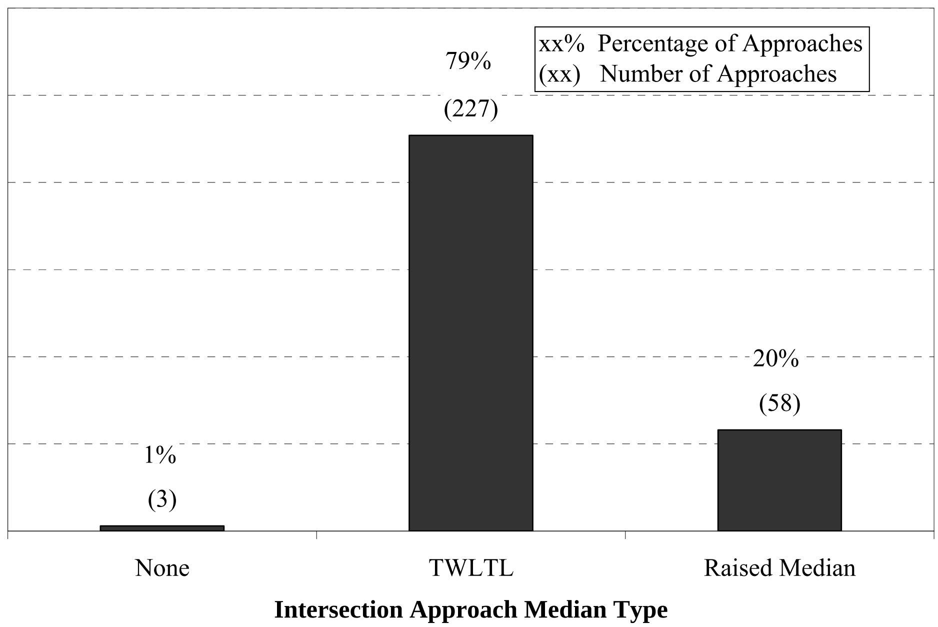

Figure 3.10 Distribution of major-street approaches by median type.

3.2.3 Geometry

Two types of geometric measurements were obtained from each study site, including lane configuration and upstream corner clearance. This section discusses the process and reasoning for obtaining both types.

3.2.3.1 Lane Configuration

Roadway lane configuration data was acquired for two reasons. First, more roadway lanes lead to additional traffic conflicts and greater distances for left-turning vehicles to traverse, increasing exposure time to oncoming traffic. Second, as previously mentioned in Section 3.2.2.3, AADT volumes were not always available for intersection minor streets. Thus, roadway lane configuration was used as an indicator for minor-street traffic volumes. Using aerial imagery from Google Earth (Google 2008a), Yahoo Maps (Yahoo 2008), and Windows Live Maps (Microsoft 2008) and street imagery from Google Maps (Google 2008b) and UDOT Roadview Explorer (UDOT 2008c), the number of through lanes for both major-street approaches and minor-street approaches was gathered. In cases where opposing approaches featured unequal numbers of through lanes, the average number of through lanes rounded down to the nearest whole number was utilized. Shared lanes, such as a combined through and right-turn lane, were counted as a through lane. Figure 3.11 shows the distribution of approach through lanes on major streets.

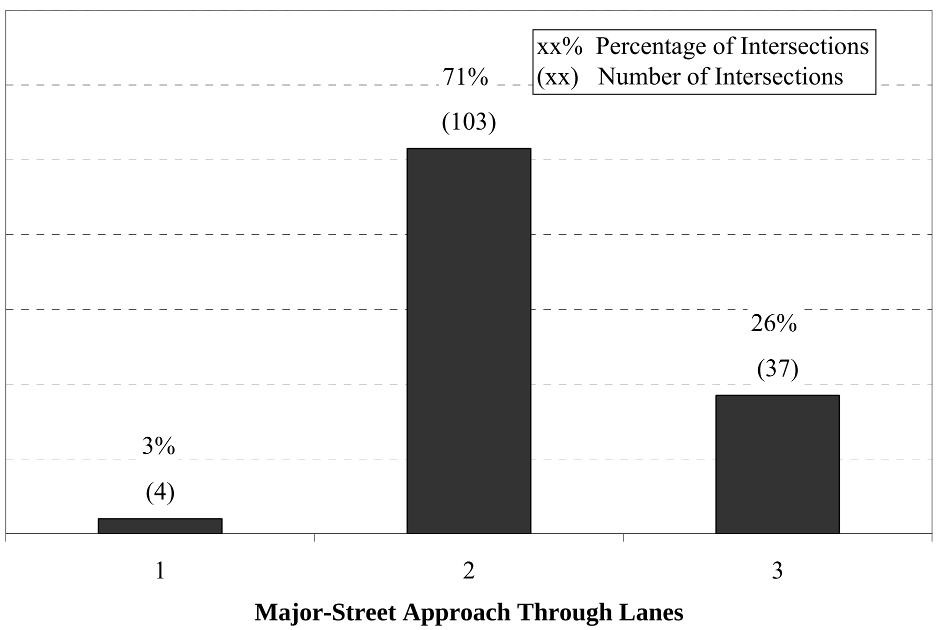

Figure 3.11 Distribution of study intersections by major-street through lanes.

Figure 3.12 shows the distribution of the approach through lanes on minor streets. A comparison of Figures 3.11 and 3.12 shows that minor streets have, on average, less through lanes than major streets have. A combined 97 percent of major streets have, on average, two or more through lanes, while 71 percent of minor streets have, on average, less than two through lanes. A full listing of major and minor-street lane classifications is contained in Appendix C.

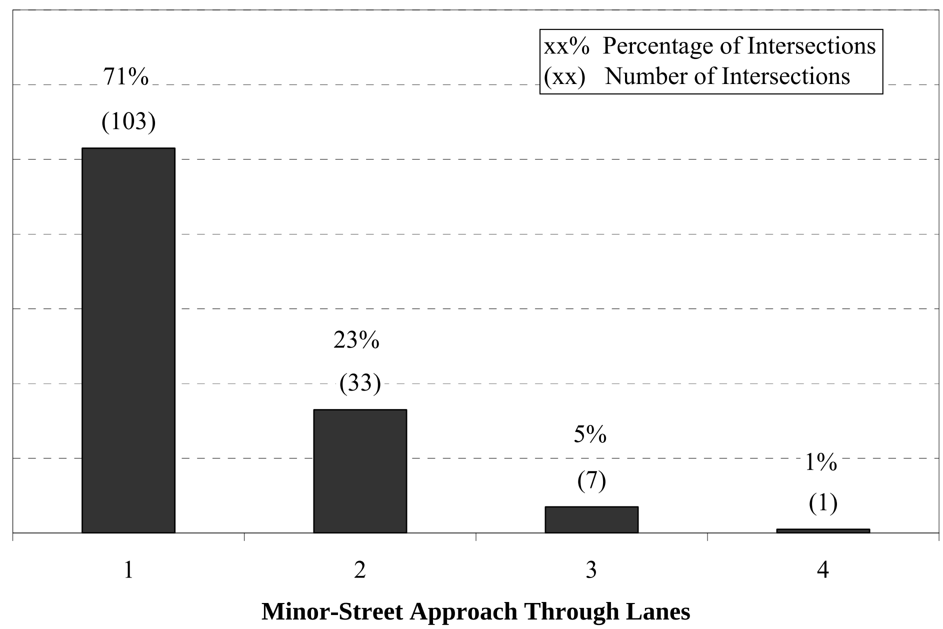

Figure 3.12 Distribution of study intersections by minor-street through lanes.

3.2.3.2 Upstream Corner Clearance

Upstream corner clearance is an important measurement because inadequate corner clearances can lead to blocked accesses and increased weaving conflicts (TRB 2003). According to UDOT standards, upstream corner clearance from a traffic signal to a driveway is measured between inside points of curvature (UDOT 2006a). The upstream-approach corner clearance was measured for each major-street approach using aerial imagery and measurement tools from Google Earth (Google 2008a). Figure 3.13 shows the distribution of major-street upstream approach corner clearances. As can be seen from Figure 3.13, the majority of approaches featured corner clearances less than 200 feet. However, a significant number of approaches had relatively long corner clearances as evidenced by the 16 percent of approaches within the 500 feet or greater distribution category. A full listing of study site corner clearances can be found in Appendix D.

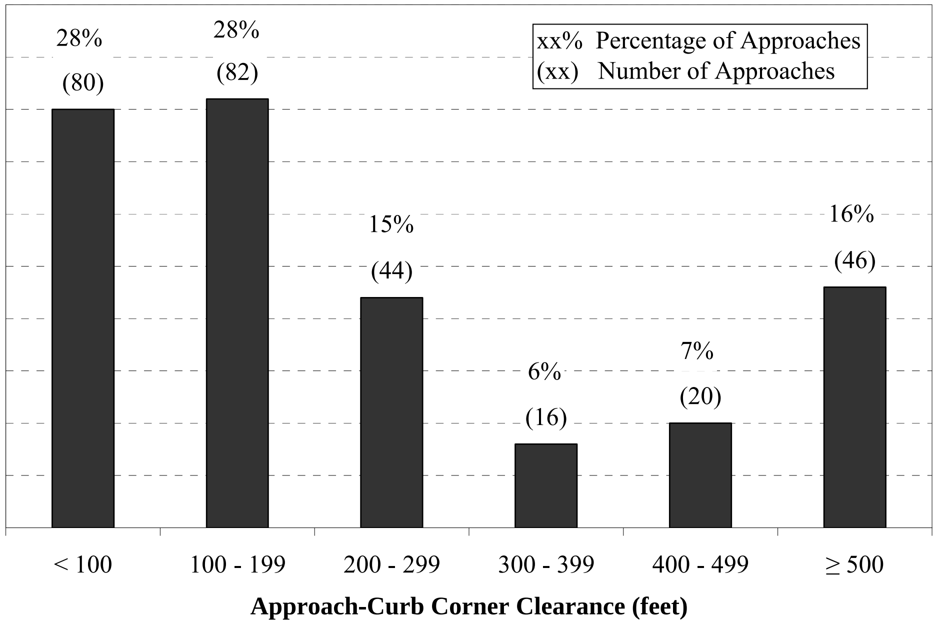

Figure 3.13 Distribution of major-street approaches by approach corner clearance.

3.2.4 Functional Area

Quantitatively comparing functional area access densities and crash rates necessitated the development of a methodology to identify the functional area of each intersection. As discussed in Section 2.3, no standard procedure for determining the functional area of an intersection could be found in the literature; however two major guidelines were reported. First, AASHTO (2004) states that the functional area should encompass any auxiliary lanes. Second, both Stover and Koepke (2002) and Koepke (1999) present a methodology to calculate the functional distance of an intersection approach by summing: 1) perception-reaction distance, 2) partial-braking-while-moving laterally-into-the-turn-lane distance, 3) full braking distance, and 4) queue storage. Figure 2.4 in Section 2.3 diagrams the above functional distance methodology and is repeated as Figure 3.14 for reference. Once both approach functional distances are computed, they can be combined with the intersection physical area to obtain the overall functional area of the intersection.

Because the literature presented no clear consensus regarding functional area assessment, a technique that satisfies both guidelines was developed. First, the functional distance of an approach was calculated according to Stover and Koepke's (2002) and Koepke's (1999) methodology. Second, this distance was compared to the length of the longest auxiliary turn lane (including taper), and the greater of the two was accepted as the approach functional distance. Third, the process was repeated for the opposite approach, and both functional distances were added to the intersection's physical area to obtain the complete intersection functional area.

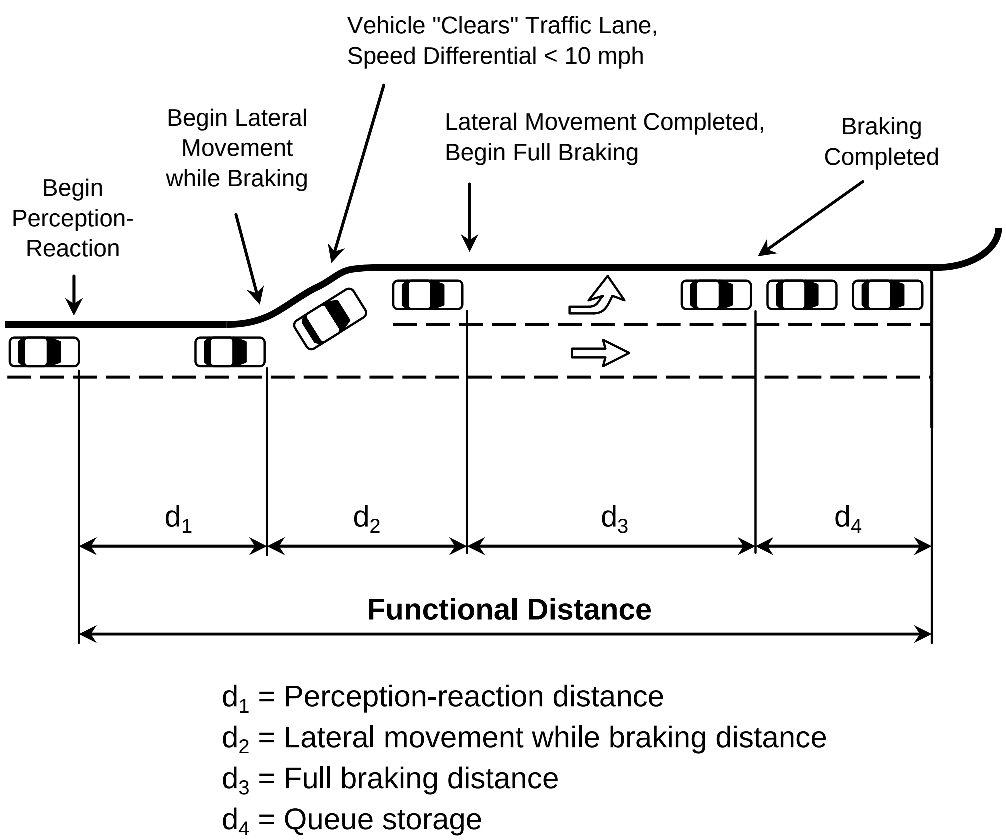

Figure 3.14 Upstream functional distance (adapted from TRB 2003).

To effectively calculate the functional distance according to the methodology mentioned above, a set of driver behavior characteristics must be assumed. As indicated in Section 2.3, various driver reaction times, vehicle deceleration parameters (deceleration while moving laterally and full braking deceleration), and queuing analysis methods are available in the literature. The following sections document the parameters and methods chosen to calculate study site functional distances.

3.2.4.1 Perception-Reaction Distance

As previously mentioned in Section 2.3.1, perception-reaction distance is a function of vehicle speed and perception-reaction time. Equation 2.1 in Section 2.3.1 shows the equation for the perception-reaction distance and is repeated as Equation 3.1 for reference. Since vehicle speeds vary, the posted speed limit was used to approximate average vehicle speed. Within the literature, sample functional distance calculations utilize perception-reaction times ranging from 1.0 second to 2.5 seconds.

$$d_1 = 1.47vt \tag{3.1}$$

where: d_1 = perception-reaction distance (feet),

v = vehicle speed (mph), and

t = driver perception-reaction time (seconds).

AASHTO states that a 2.5-second brake reaction time represents the capabilities of most drivers in a variety of driving situations (2004). Several factors, however, suggested using a short perception-reaction time for the purposes of this study. First, a turning maneuver is an expected event and most drivers make the decision to turn well in advance of the intersection. Thus, because the perception component of the procedure is negligible, the perception-reaction time is essentially the time required to transition the foot from the gas pedal to the brake pedal (Stover 1993). Second, every study site is located in an urban area, and perception-reaction times have been found to be shorter in urban conditions (TRB 2003). Finally, at major urban intersections, the majority of drivers are familiar with the layout, which correlates with shorter perception-reaction times (Stover and Koepke 2002). Because of the anticipatory nature of the maneuver, the urban study site conditions, and the familiarity of drivers, a perception-reaction time of 1.0 second was selected for the functional distance calculation.

3.2.4.2 Lateral-Movement-While-Braking Distance

As shown in Equation 2.2 in Section 2.3.2, the lateral-movement-while-braking distance is a function of vehicle speed and the forward deceleration rate implemented as a vehicle moves laterally into the turn lane. Equation 2.2 is repeated as Equation 3.2 for reference. The literature suggests that this deceleration rate is applied until the driver clears the through lane and before a 10 mph speed differential is achieved (Stover and Koepke 2002). Again, the roadway speed limit was used to estimate average vehicle speed. Forward deceleration was assumed to be 5.8 ft/s^2, as recommended by Stover and Koepke (2002). At this rate, a vehicle maintaining a lateral speed between 3 ft/s and 4 ft/s can clear the through lane before a 10 mph speed differential is achieved (Stover and Koepke 2002). The 5.8 ft/s^2 deceleration rate is also within the bounds of the other deceleration rates reported in the literature.

$$d_2 = \frac{(1.47v_2)^2 - (1.47(v_2 - 10))^2}{2a_2} \qquad (3.2)$$

where: d_2 = lateral movement while braking distance (feet),

v_2 = vehicle speed (mph), and

a_2 = vehicle deceleration rate while moving laterally (ft/s^2).

3.2.4.3 Full Braking Distance

As shown in Equation 2.3 in Section 2.3.3, the full braking distance is a function of vehicle speed and full deceleration rate. Equation 2.3 is repeated as Equation 3.3 for reference. Full deceleration occurs after a vehicle has cleared the through lane and reduced its speed by no more than 10 mph. Thus, average vehicle speed at onset of full deceleration was assumed to be 10 mph below the posted speed limit.

$$d_3 = \frac{(1.47v_3)^2}{2a_3} \tag{3.3}$$

where: d_3 = full braking distance (feet),

v_3 = vehicle speed after lateral movement and deceleration (mph), and

a_3 = full vehicle deceleration rate (ft/s^2).

Full deceleration is greater than deceleration while moving laterally. Most sample functional distances in the literature are derived using full deceleration rates up to 9.0 ft/s^2 (Stover and Koepke 2002; Koepke 1999). Studies that examine braking in detail suggest acceptable deceleration rates of 11.2 ft/s^2 (AASHTO 2004), 10.0 ft/s^2 (Koepke 1999; Chang et al. 1985), 11.6 ft/s^2 (Wortman et al. 1985), and 9.9 ft/s^2 (Gates et al. 2007). Thus, a full deceleration rate of 10.0 ft/s^2 was determined to be a suitable value for the full-braking-distance calculation.

3.2.4.4 Queue Storage

As reviewed in Section 2.3.4, numerous methods are available for determining appropriate turn-bay storage length. However, each of these techniques requires microscopic traffic parameters such as turn volumes, cycle lengths, and phase plans as inputs. Because obtaining this level of detail for each of the 159 study and reference intersections would be infeasible, an alternate method of estimating typical queue storage was developed.

The alternate queue storage estimation method was based on turn-bay design principles. One typical design guideline is that storage lanes should accommodate queue demands 95 percent of the time (AASHTO 2004). Gattis (2000) identified that the 95 percent probability queue demand can be roughly approximated by doubling the average number of queued vehicles. Additionally, AASHTO states, "The storage length is a function of the probability of occurrence of events that should usually be based on one and one-half to two times the average number of vehicles that would store per cycle…"

(AASHTO 2004, p. 715). Thus, turn bays designed according to these guidelines feature storage lengths about twice as long as the average queues.

Although turn-bay design may vary from location to location, the assumption was made that most turn bays are designed to accommodate 95 percent of the queues. With this assumption, the average queue length can be estimated by multiplying the existing storage length by some reduction factor. A reduction factor consistent with the double the average approximation would equal 0.5. Equation 3.4 shows the average queue length and storage length relationship:

$$L_Q = L_S \alpha \tag{3.4}$$

where: L_Q = average turn-bay queue length (feet),

L_S = existing turn-bay storage length (feet), and

α = reduction factor (assume 0.5).

To calibrate the reduction factor to the study data and account for regional variation in turn-bay design, a sensitivity analysis was conducted at 11 study site turn bays. Queuing patterns at each turn bay were observed during off-peak hours, and the maximum number of accumulated vehicles per cycle was recorded for 10 cycles. Assuming a vehicle storage length of 25 feet per vehicle, the number of queued vehicles was converted into a queue length. This average maximum queue length was then compared to the turn-bay storage length, defined as the distance between the stop line and the beginning of the turn-bay striping. The turn-bay striping distance was obtained from Google Earth aerial imagery and measurement tools (Google 2008a). Next, the ratio of average maximum queue length to storage length was calculated for each turn bay. Table 3.1 shows the results of the sensitivity analysis. As can be seen from the table for the 11 sensitivity analysis intersections, the storage length reduction factor averages approximately 0.5, as assumed. The 0.5 reduction factor from the sensitivity analysis was applied to each study intersection to obtain average queue lengths. In cases where intersections featured a right- and left-turn bay, the longer of the two was used for the calculation.

Table 3.1 Reduction Factor Sensitivity Analysis

Major Street	Minor Street	Approach Direction	Turn Bay	Average Maximum Queue Length (feet)	Storage Length (feet)	Ratio
University Avenue	1200 South	SB	Left	55	100	0.55
University Avenue	East Bay Blvd	SB	Left	93	200	0.46
University Parkway	State Street	EB	Left	163	350	0.46
University Parkway	State Street	WB	Left	100	220	0.45
University Parkway	Main Street	WB	Left	78	240	0.32
State Street	4500 South	SB	Left	138	580	0.24
State Street	5900 South	NB	Left	70	250	0.28
State Street	6100 South	SB	Left	103	410	0.25
Redwood Road	4100 South	SB	Left	208	260	0.80
Redwood Road	4200 South	SB	Left	48	100	0.48
Redwood Road	4700 South	NB	Left	178	200	0.89
Average						**0.47**

3.2.4.5 Compilation of Functional Area

To compile an intersection functional area, both upstream functional distances were added to the intersection physical area, which was defined as the distance between the stop lines. Figure 3.15 shows the components of an intersection functional area.

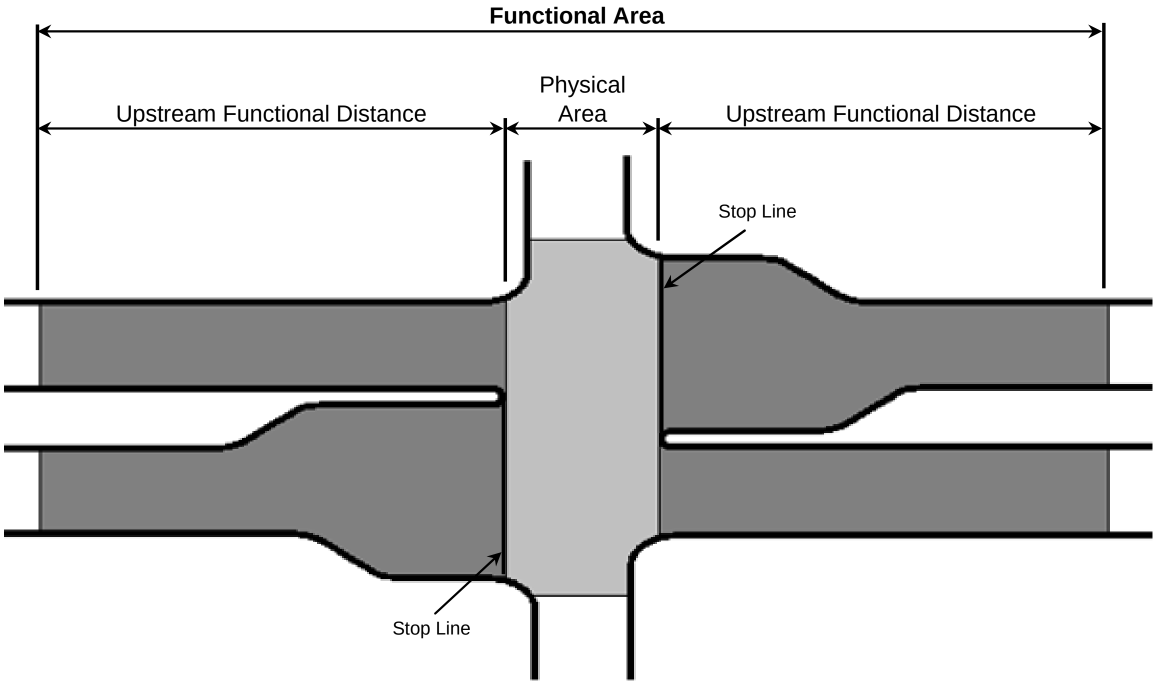

Figure 3.15 Components of an intersection functional area.

As can be seen from Figure 3.15, downstream functional distances were not defined in this analysis. Because data in the UDOT Data Almanac do not provide sufficient detail to determine whether a crash occurred while vehicles were approaching or departing an intersection, delineating between upstream and downstream functional distances was not appropriate. In other words, the downstream functional distance was assumed to be the same length as the upstream functional distance on the same leg of the intersection. Thus, crashes and roadway features within the upstream functional distance were included in the intersection functional area regardless of the vehicles' direction of travel or which side of the street the roadway features were located.

Figure 3.16 shows the distribution of study site functional area lengths. As can be seen from Figure 3.16, the majority of functional areas ranged between 600 feet and 1,000 feet. A full listing of study site functional area lengths can be found in Appendix E.

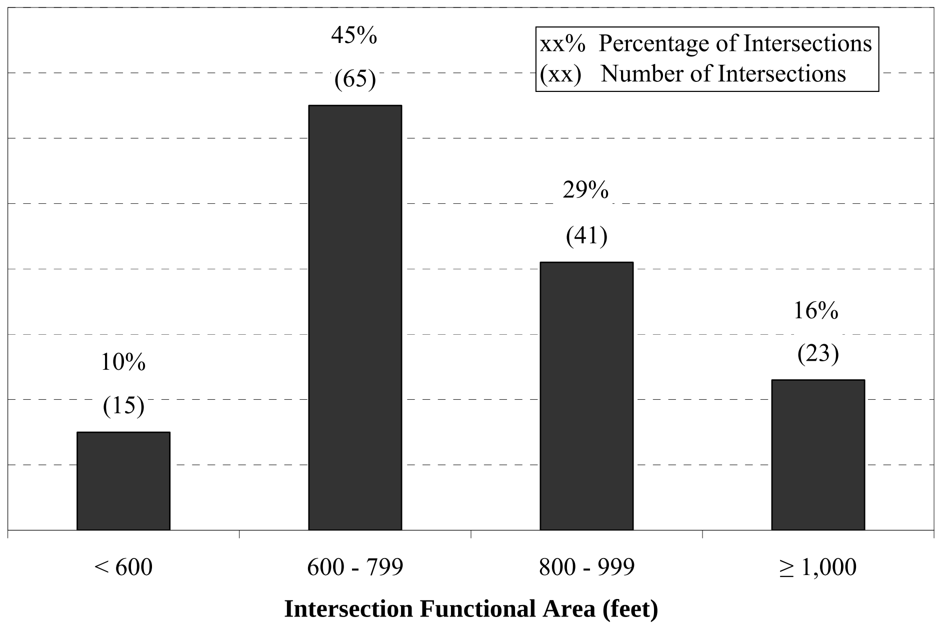

Figure 3.16 Distribution of study intersections by length of functional area.

3.2.4.6 Functional Area Overlap

Closely spaced intersections have the potential to share crashes within overlapping functional areas. These intersections may exhibit unnaturally high crash rates because crashes occurring in one intersection's functional area may actually be attributed to activity from the adjacent intersection. In order to account for this possibility, functional areas at closely spaced signals were measured, and study sites were flagged if the functional areas overlapped. Figure 3.17 shows the distribution of study sites with overlapping functional areas. Nearly one-fourth, or 22 percent, of study site functional areas overlapped with the functional area of an adjacent signal. A full listing of study site functional area overlaps can be found in Appendix E.

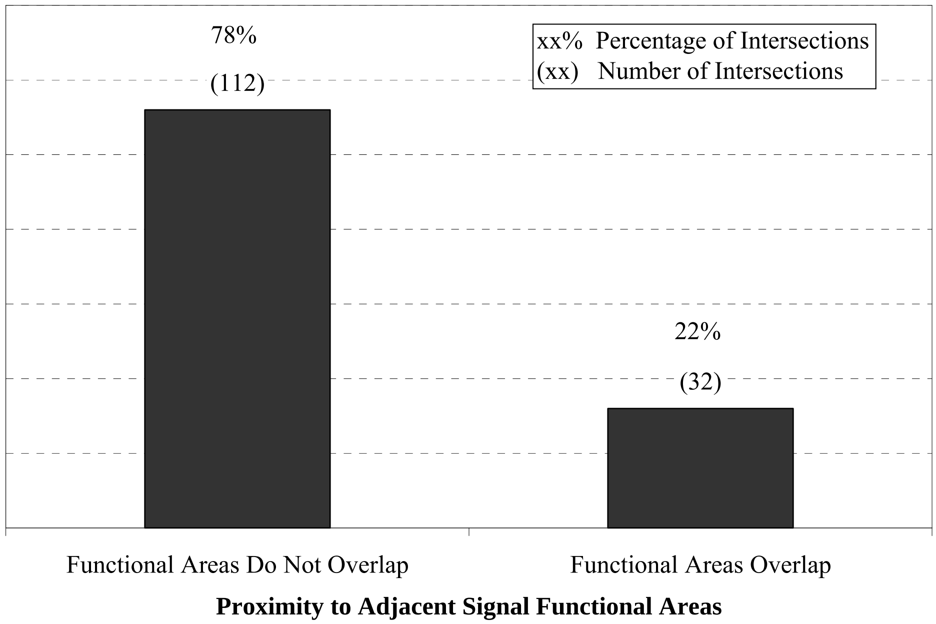

Figure 3.17 Distribution of study intersections by functional area overlap.

3.2.5 Accesses

After the functional area of each study site was defined, the quantity and attributes of accesses within the functional area were determined. The type of access data gathered includes total accesses, total conflict points, access density, conflict point density, and land use. This section discusses the methods and purposes for gathering the access data. A full listing of access and conflict point totals and densities according to land use is contained in Appendix F.

3.2.5.1 Total Accesses

The distance of an access from the intersection stop line was used to determine whether the access was within the intersection functional area. The distance was measured from the intersection stop line to the centerline of each upstream access utilizing aerial imagery and measurement tools from Google Earth (Google 2008a). Accesses that were directly aligned with one another on opposite sides of the street were counted as one access. Figure 3.18 shows the distribution of accesses within each study intersection functional area. The majority of intersection functional areas contained between one and 15 accesses. Only 7 percent of intersection functional areas contained no accesses, and only 2 percent, or three intersections, had 15 or more accesses.

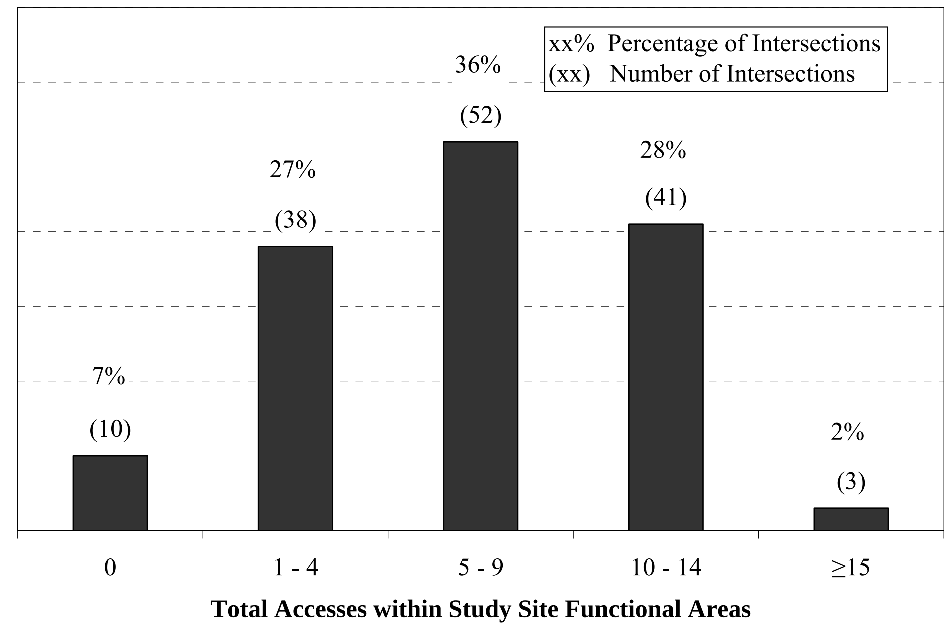

Figure 3.18 Distribution of study intersections by total accesses.

3.2.5.2 Total Conflict Points

The number of conflict points associated with each access is an indication of the potential for interference with the through-traffic stream at the access. Each access was evaluated to determine how many conflict points it creates with the adjacent roadway.

The number of conflict points generated by an access is a function of the roadway median type, number of lanes, access type, and the presence of an access on the opposing side of the street. First, median type influences conflict points by prohibiting left-turns. An access along a roadway with a raised median produces only two conflict points, as illustrated in Figure 3.19. Second, additional through lanes increase conflict points on undivided roads, as illustrated in Figure 3.20 and Figure 3.21. The same effect is evident when an access is located within an auxiliary lane, as illustrated in Figure 3.22. Third, an access can be designed to limit certain movements, reducing conflict points, as illustrated in Figure 3.23, an exit-only access. Finally, the presence of an access on the opposite side of the street adds conflict points because turning movements for the two accesses cross paths; however, some simplifications may be made depending on the property type. For example, most opposing accesses serve small, unrelated developments. The likelihood that a patron accessing one site will also need to access the site directly across the roadway is rare. Therefore, for this analysis, unless the opposing accesses were an intersecting crossroad, or both served large developments, no cross-street trips were assumed to be made. Figures 3.24 and 3.25 show conflict point diagrams for opposing accesses with cross-street trips and without cross-street trips, respectively.

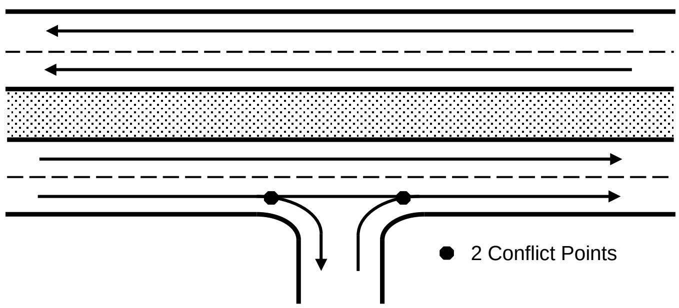

Figure 3.19 Conflict points for an access on roadway with raised median.

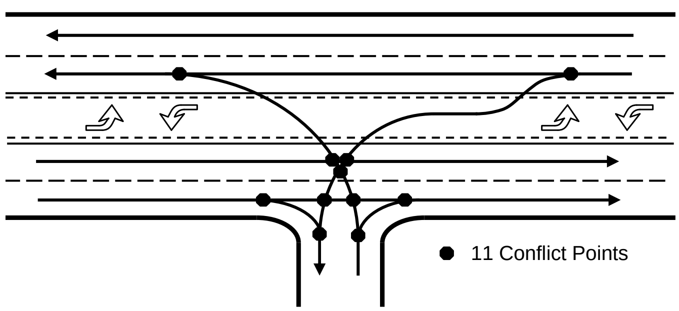

Figure 3.20 Conflict points for an access on a four-lane roadway with a TWLTL.

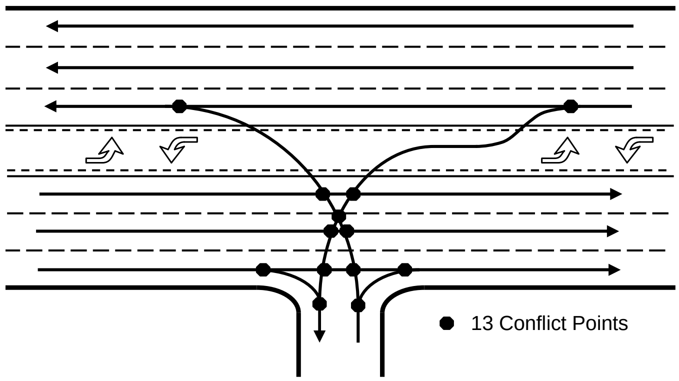

Figure 3.21 Conflict points for an access on a six-lane roadway with a TWLTL.

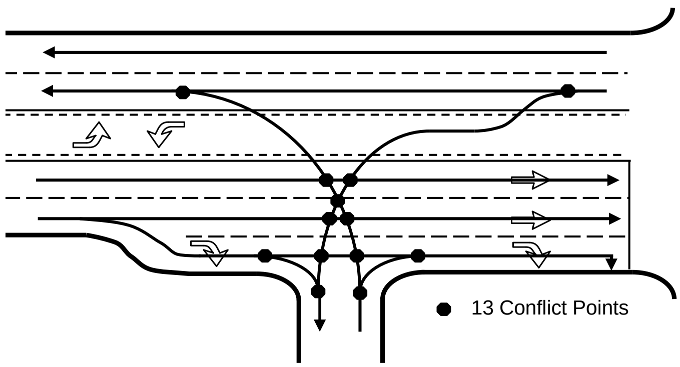

Figure 3.22 Conflict points for access within right-turn lane on a four-lane roadway.

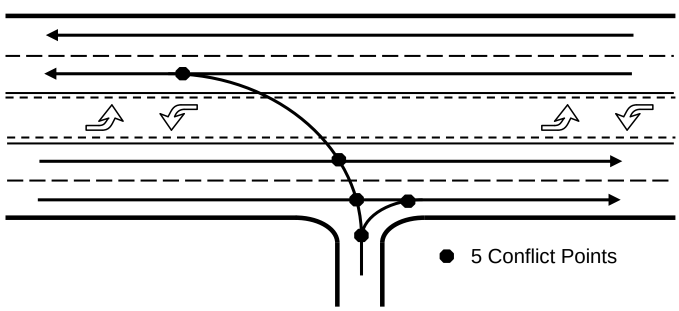

Figure 3.23 Conflict points for an exit-only access.

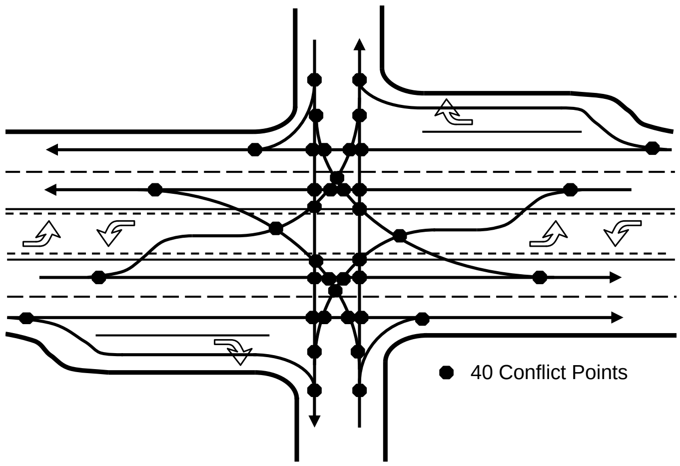

Figure 3.24 Conflict points for opposing accesses with cross-street trips.

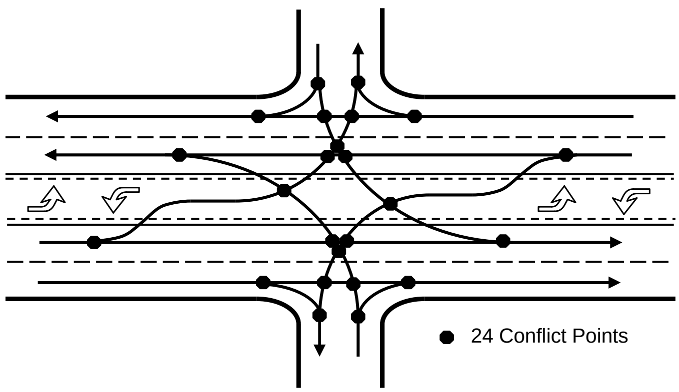

Figure 3.25 Conflict points for opposing accesses without cross-street trips.

Aerial imagery from Google Earth (Google 2008a), Yahoo Maps (Yahoo 2008), and Windows Live Maps (Microsoft 2008) and street imagery from Google Maps (Google 2008b) and UDOT Roadview Explorer (UDOT 2008c) were used to determine access and roadway configuration and tabulate access conflict points. Figure 3.26 shows the distribution of conflict points by study site intersection. Most study functional areas contained between 1 and 150 conflict points. Seven percent, or 10 intersection functional areas, featured no conflict points.

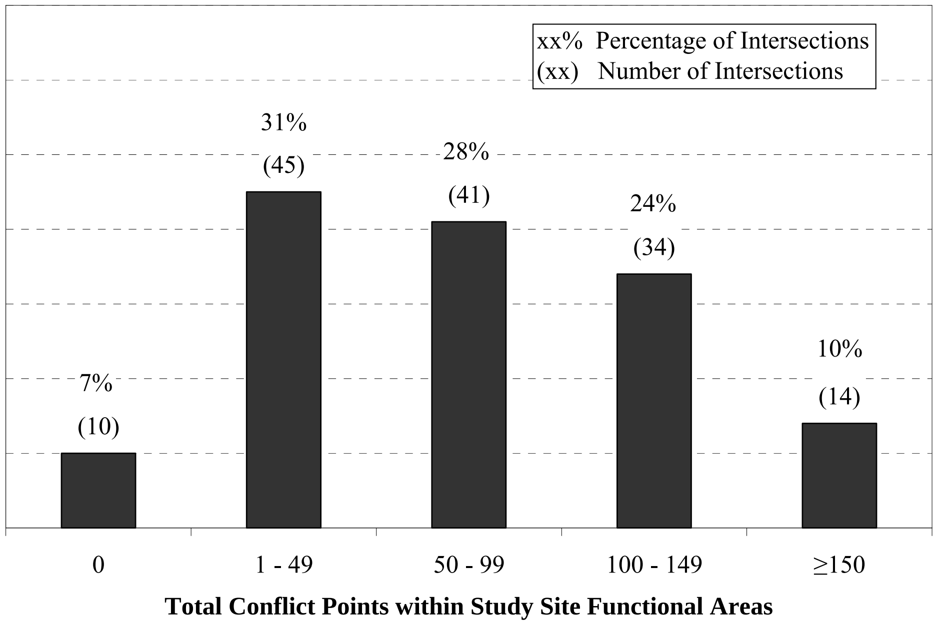

Figure 3.26 Distribution of study intersections by total conflict points.

3.2.5.3 Access Density

Because functional areas vary in size, calculating access density was important in order to normalize the number of accesses exhibited by each intersection and provide a better methodology for site to site comparison. In addition, access density has been found to be correlated with increased crash rates and crash severity (Gluck et al. 1999; Schultz and Braley 2007).

Based on the number of accesses within the functional area of each intersection, the functional area access density was calculated. Equation 3.5 shows the formula for access density. Because intersection physical areas cannot contain accesses, access density was calculated according to the combined lengths of the approach functional distances. Figure 3.27 shows the distribution of access densities by intersection. As illustrated in Figure 3.27, access density distribution was fairly uniform among study sites.

$$D_{Access} = \frac{A}{L_{FA}} 1{,}000 \tag{3.5}$$

where: D_{Access} = functional area access density (accesses per 1,000 feet),

A = accesses in the intersection functional area, and

L_{FA} = total length of intersection functional distances (feet).

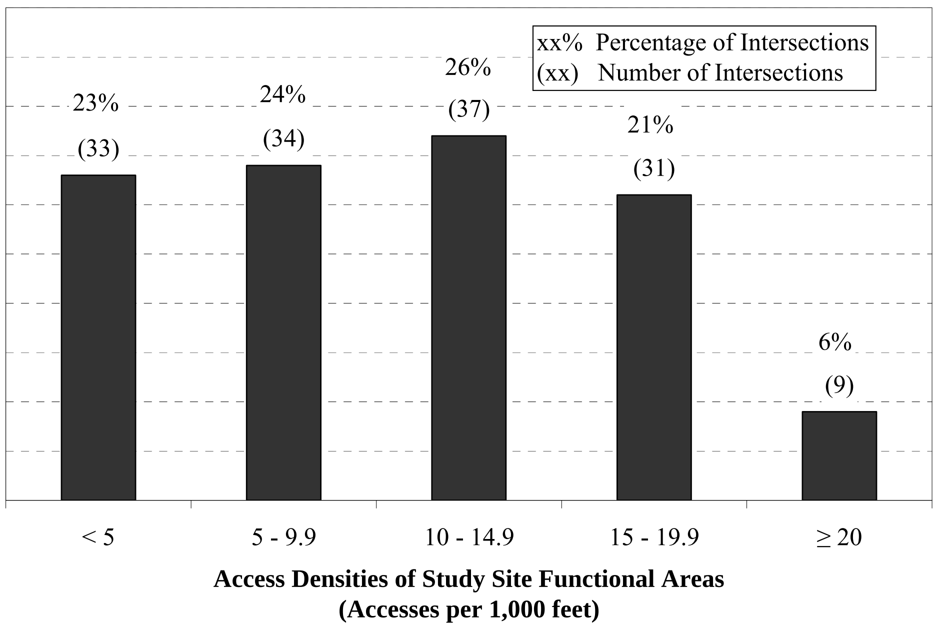

Figure 3.27 Distribution of study intersections by access density.

3.2.5.4 Conflict Point Density

As with access density, conflict point density was calculated for each intersection functional area in order to facilitate site-to-site comparisons. Again, because intersection physical areas cannot contain accesses, conflict point density was calculated according to the lengths of the functional distances. Equation 3.6 shows the conflict density formula, and Figure 3.28 shows the distribution of conflict densities by intersection. Conflict point density was more heavily distributed on the low side as compared to access density.

Thirty-one percent of intersection functional areas had a conflict point density of less than 50 conflict points per 1,000 feet.

$$D_{CP} = \frac{CP}{L_{FA}} 1{,}000 \tag{3.6}$$

where: D_{CP} = functional area conflict point density (conflict points per 1,000 feet), and

CP = conflict points in the intersection functional area.

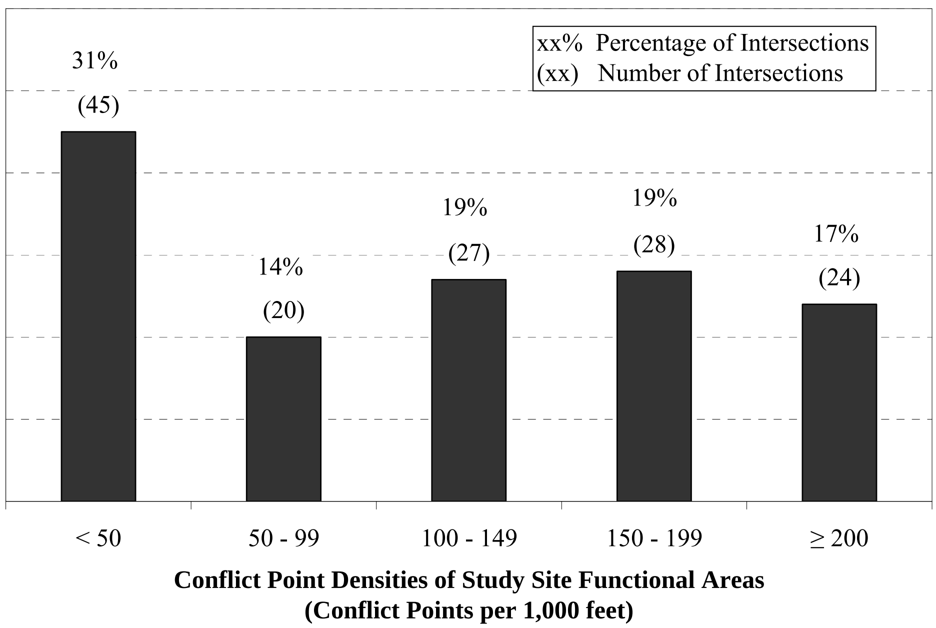

Figure 3.28 Distribution of study intersections by conflict point density.

3.2.5.5 Land Use

The land use type serviced by each access was identified because land use is an indicator of driveway volume. Also, commercial land use accesses have been found to have a stronger relationship with crash rates than residential land use accesses (Schultz and Braley 2007).

Land use was examined with aerial imagery from Google Earth (Google 2008a), Yahoo Maps (Yahoo 2008), and Windows Live Maps (Microsoft 2008) and street imagery from Google Maps (Google 2008b) and UDOT Roadview Explorer (UDOT 2008c). Each access was coded as serving either a residential or a commercial property. The distribution of access land use is shown in Figure 3.29. Eighty-six percent, or 867 of the accesses within study site functional areas, served a commercial land use. The remaining 14 percent, or 140 accesses, served residential properties.

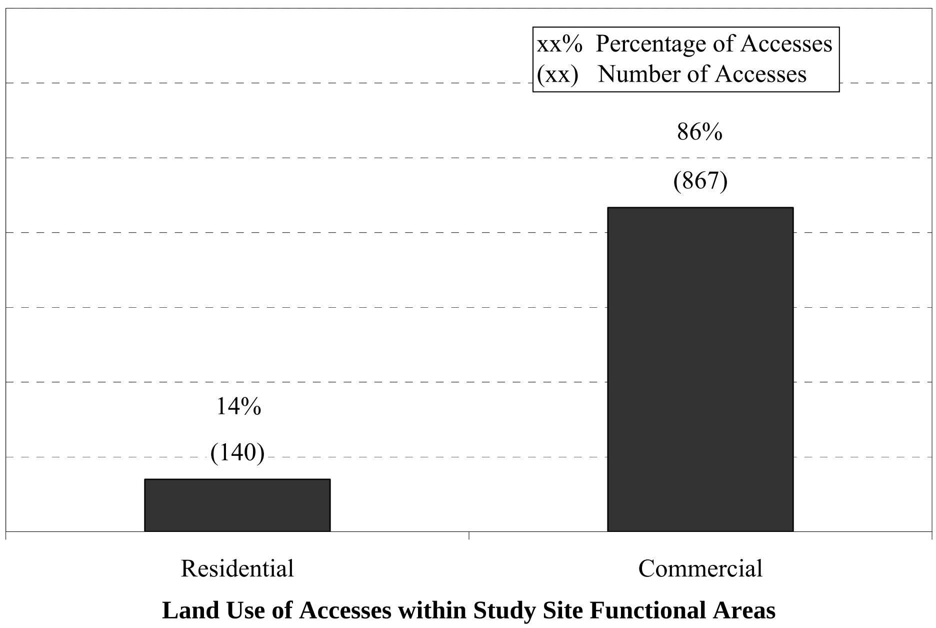

Figure 3.29 Distribution of study intersections by access land use.

3.3 Crash Data

With the functional area of each intersection defined, crash patterns within site functional areas were then evaluated. The following sections discuss purposes and methods used to collect the crash data.

3.3.1 Crash Totals

As previously mentioned in Section 3.2.1, the UDOT Data Almanac assigns crashes and intersections to a highway milepost location at a precision of hundredths of a mile. Intersection functional areas, however, were measured in units of feet. Therefore, converting functional area boundaries measurements into highway milepost locations was necessary in order to identify which roadway crashes resided within the functional area.

The functional area conversion from feet into mileposts was a multi-step process. First, the intersection milepost was assumed to be located at the center of the intersection. Since a distance of 0.01 miles translates to 52.8 feet, upstream and downstream mileposts were positioned every 52.8 feet proceeding away from the center of the intersection. Next, the length that the functional area extends from the center of the intersection into each approach was determined. This was computed by combining the functional distance of the approach with the distance from the intersection center to the stop line of the same approach. The length from the intersection center to the stop line was obtained by dividing the intersection physical area by two. Finally, the length of the functional area from the center of the intersection into the approach was rounded down to the nearest multiple of 52.8 feet. The corresponding roadway milepost was then designated as a functional area boundary milepost. Crashes located up to and including the milepost were considered to be located within the study site functional area. Figure 3.30 illustrates the functional area and roadway milepost relationship.

For some study sites, the intersecting minor street was another state route or local federal-aid route. In such cases, within the UDOT Data Almanac, a portion of the crashes occurring in the intersection physical area are sometimes assigned to the minor street. Using the minor-street milepost location previously obtained, the crashes occurring at the minor-street intersection milepost and at 0.01 mileposts away from either side of the intersection were included in the overall functional area crash total. The crashes at +/- 0.01 mileposts from the minor-street intersection milepost were included because they represent crashes within 52.8 feet of the center of the intersection. Usually, this distance is still within the intersection physical area. Crashes located beyond +/- 0.01 mileposts generally occur outside the bounds of the study area.

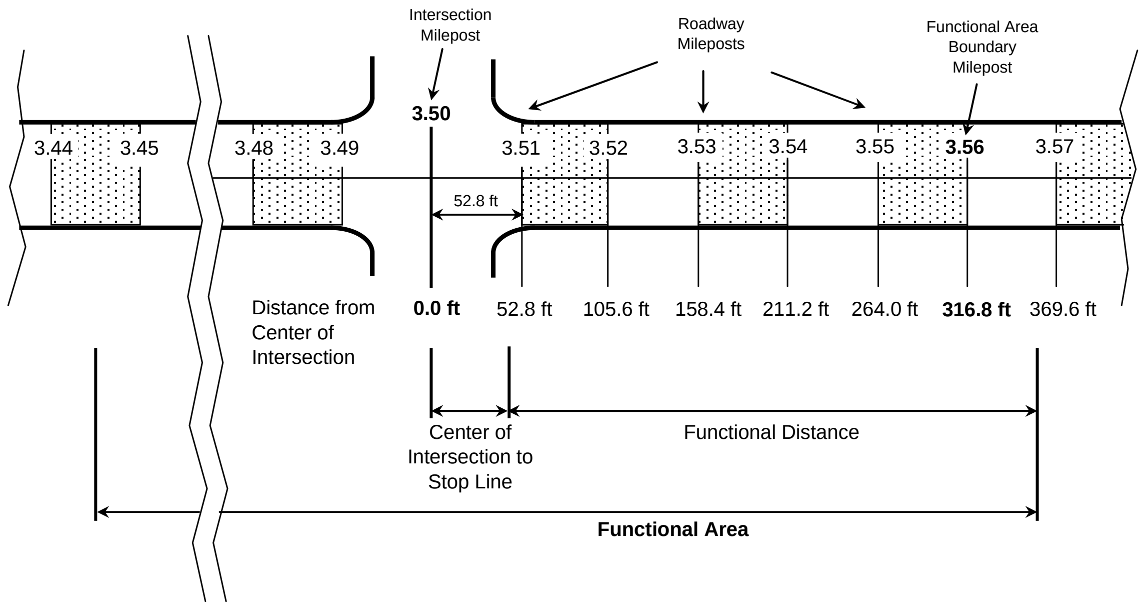

Figure 3.30 Relationship between functional area and roadway milepost system.

As mentioned in Section 3.2.1, intersection milepost locations provided by the Data Almanac "Points of Interest" search tool were cross-checked with crash data clustering. At most study sites, the natural clustering of crash data coincided with the intersection milepost. Occasionally, however, the crash cluster was located at some distance upstream or downstream of the intersection milepost. In these instances, the intersections were examined for anomalies, such as a major access or a sharp bend in the road, which might be causing the majority of crashes to occur away from the intersection area. If no unusual intersection features existed at the crash cluster location, the intersection milepost was manually adjusted to match the crash cluster.

Figure 3.31 shows the distribution of three-year crash totals within the study site functional areas. Most intersections featured between 20 and 60 crashes for the three-year study period. However, the distribution indicates that several sites had relatively high crash totals, as 15 percent of study sites had greater than or equal to 100 crashes. A full listing of study site crash totals can be found in Appendix G.

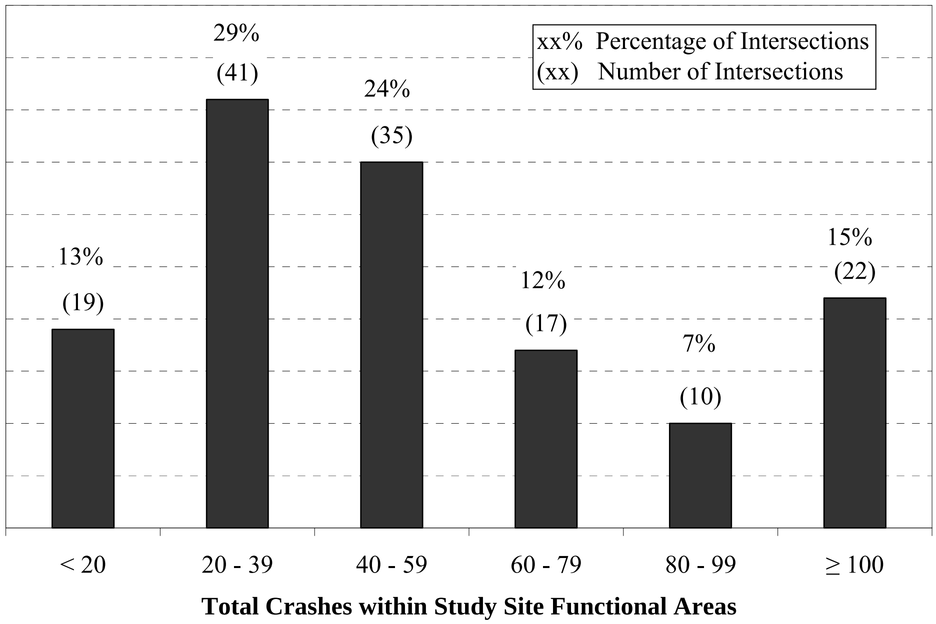

Figure 3.31 Distribution of study intersections by functional area crashes.

3.3.2 Crash Rates

Crash rates provide a way to compare crash frequencies while accounting for the influential roadway conditions that may vary from site to site, such as traffic volumes or roadway length. As presented in Section 2.4.1, intersection crash rates are computed in units of crashes per MEV. The number of entering vehicles is comprised of the average entering volumes from both major-street and minor-street approaches. However, because minor-street approach volumes were not consistently available, the intersection crash rate was calculated with major-street approach volumes only. The effect of minor-street traffic were accounted for with other variables, as discussed in Section 4.2.1.1. Equation 3.7 shows the adjusted intersection crash rate equation used for analysis:

$$R_{MAJ} = \frac{C_{INT}}{365 \times V_{MAJ}} 1{,}000{,}000 \qquad (3.7)$$

where: R_{MAJ} = adjusted intersection crash rate (crashes per year per million major-street entering vehicles [MMSEV]),

C_{INT} = intersection crashes per year, and

V_{MAJ} = entering volumes of major-street legs (vehicles per day).

Figure 3.32 shows the distribution of major-street intersection crash rates among study intersections. Compared to the crash total distribution, the crash rate distribution is weighted more heavily on the low end. However, like the crash total distribution, the crash rate distribution features a significant number of study sites with relatively high crash rates. Fourteen percent of intersections had a crash rate greater than or equal to 3.0 crashes per MMSEV. A full listing of study site crash rates can be found in Appendix H.

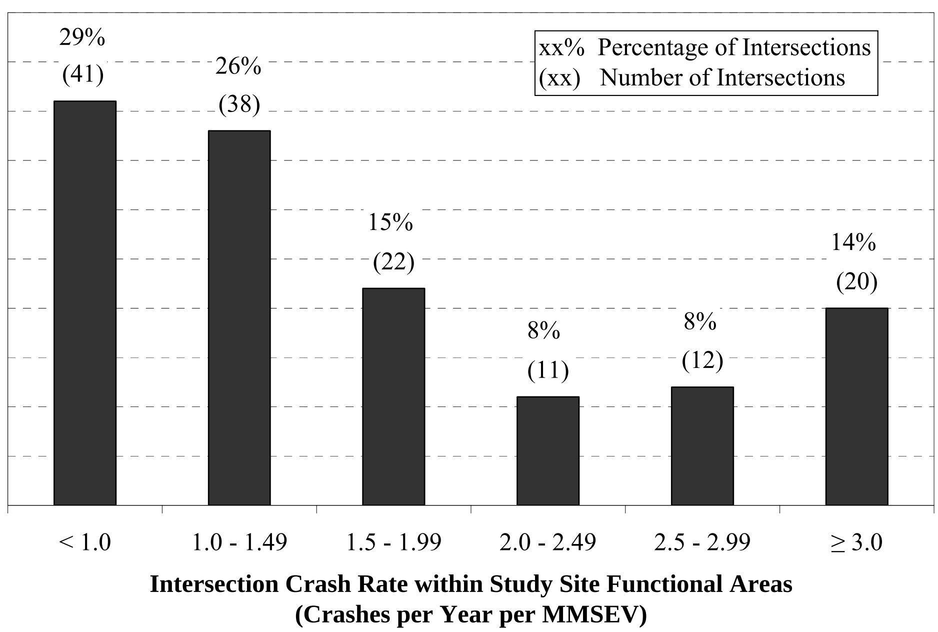

Figure 3.32 Distribution of study intersections by adjusted intersection crash rate.

3.3.3 *Crash Severity*

Crash totals and crash rates provide information concerning the frequency of crashes at a site, but they do not account for the degree of personal and property damage incurred. Crash severity scores are used to evaluate how severely victims are harmed by crashes. Roadway conditions such as speed, access density, land use, and the presence of raised medians can influence the average crash severity along a roadway (Schultz and Braley 2007; Schultz and Lewis 2006).

Crash severities for each study site were evaluated according to the UDOT-identified crash costs discussed in Section 2.4.2. Severity levels were obtained for each crash by utilizing the "Advanced Search" option within the Data Almanac to create a custom search filter. The Data Almanac categorizes crash severities into five classifications that correspond with the NSC crash severity categories (Anderson et al. 2006; NSC 2007). Table 3.2 shows the Data Almanac and NSC severity categories along with the associated UDOT crash costs (UDOT 2006b).

Table 3.2 Crash Severity Descriptions and Costs

NSC Classification[1]	Data Almanac Equivalent[2]	UDOT Cost[3]
Non-injury	No Injury	$ 4,400
Possible injury	Possible injury	$ 42,000
Non-incapacitating evident injury	Bruises and Abrasions	$ 80,000
Incapacitating injury	Broken Bones or Bleeding Wounds	$ 785,000
Fatal	Fatal	$ 785,000

1. NSC 2007
2. Anderson et al. 2004
3. UDOT 2006b

Figure 3.33 shows the distribution of crashes among the five severity levels. As can be seen from Figure 3.33, the percentage of crashes decreases as severity level increases. The “no injury category” contained twice as many crashes as the next largest category. On the opposite end of the severity scale, only 12 total crashes, which rounds down to 0.1 percent, included a fatality.

As discussed in Section 2.4.2, UDOT has developed damage costs based on the crash severity level. A severity score was calculated for each study intersection by totaling the associated severity damage costs for every crash occurring in the functional area and dividing by the number of study years. Figure 3.34 shows the distribution of severity scores among study intersections. The distribution shows a decreasing percentage of intersections as total severity score increases. However, as with the crash total distribution and the crash rate distribution, several intersections feature relatively high severity scores. Specifically, 8 percent of intersections had a severity score greater than or equal to $2,500,000. Appendix G contains a full listing of study site crashes according to severity as well as a full listing of study site total severity costs.

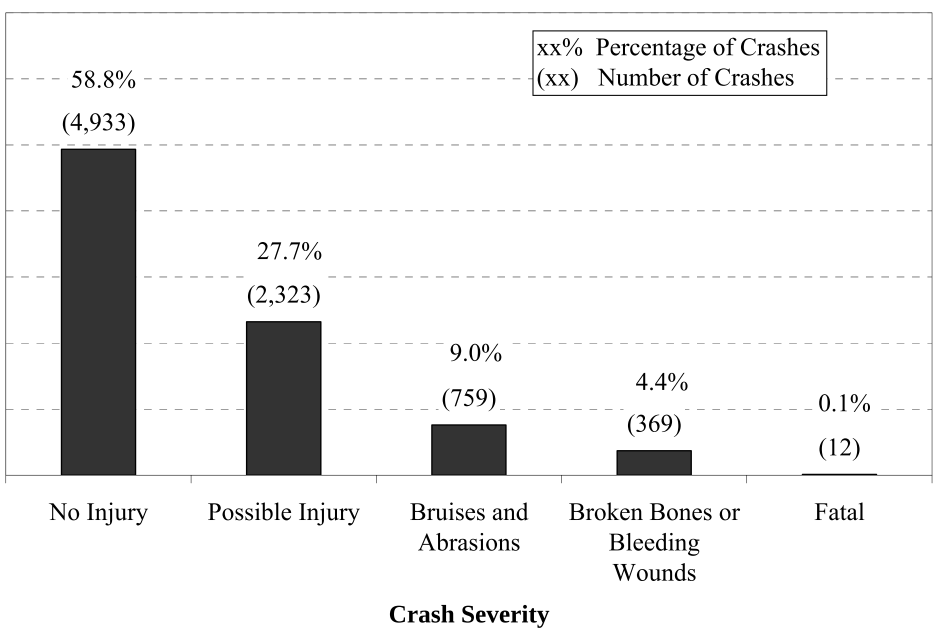

Figure 3.33 Distribution of crashes by severity.

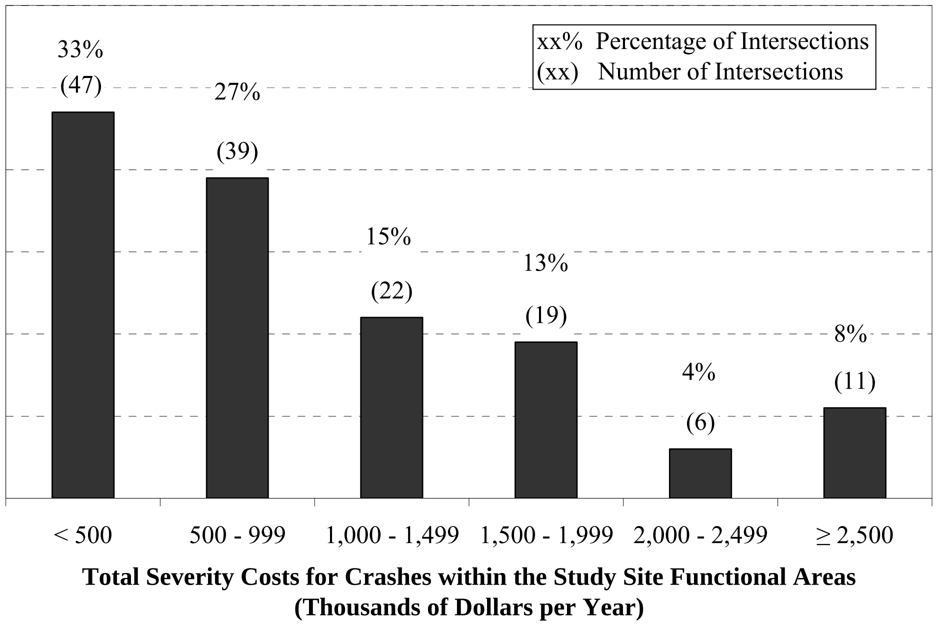

Figure 3.34 Distribution of study intersections total severity costs.

3.3.4 Crash Type

The UDOT Data Almanac codes crash types as one of 26 different crash descriptions (Anderson et al. 2006). To simplify analysis, the crash descriptions were combined into six categories as identified by Schultz and Braley (2007) and presented in Table 3.3. By utilizing the "Advanced Search" option in the Data Almanac, a custom search filter was created to sort functional area crashes into the above-mentioned categories. Figure 3.35 shows the distribution of all study site crashes by crash type. As can be seen from the graph, right-angle crashes and rear-end crashes comprise the majority of crash types. Of the remaining four crash types, no single category contained more than 6 percent of total crashes. Appendix H contains a full listing of study site crashes according to crash type.

Table 3.3 Crash Type Categories (adapted from Schultz and Braley 2007)

Crash Type	Data Almanac Collision Description
Right Angle	Both vehicles straight, approaching at an angle
	One vehicle straight, one coming from right, turning right
	One vehicle straight, one coming from left, turning left
	One vehicle straight, one coming from right, turning left
	One vehicle straight, one coming from left turning right
	Opposite directions, both vehicles turning left
	Opposite directions, one turning left, one turning right
	Opposite directions, one vehicle straight, one vehicle turning left
	Approaching at an angle, both vehicles turning right
	Approaching at an angle, both vehicles turning left
	Approaching at an angle, one turning left, one turning right
Rear End	Same direction, both vehicles straight, rear end
	Same direction, one vehicle straight, one turning right, rear end
	Same direction, one vehicle straight, one turning left, rear end
Side Swipe	Same direction, both straight, side swipe
Head On	Opposite directions, both vehicles straight, head on
	Opposite directions, both straight, side swipe
Single Vehicle	Single vehicle
Other	Same direction, one vehicle straight, one turning right
	Same direction, one vehicle straight, one turning left
	Same direction, both vehicles turning left
	Same direction, both vehicles turning right
	Same direction, one vehicle turning right, one vehicle turning left
	Backing
	One vehicle straight, one vehicle making U-turn
	One moving, one parked

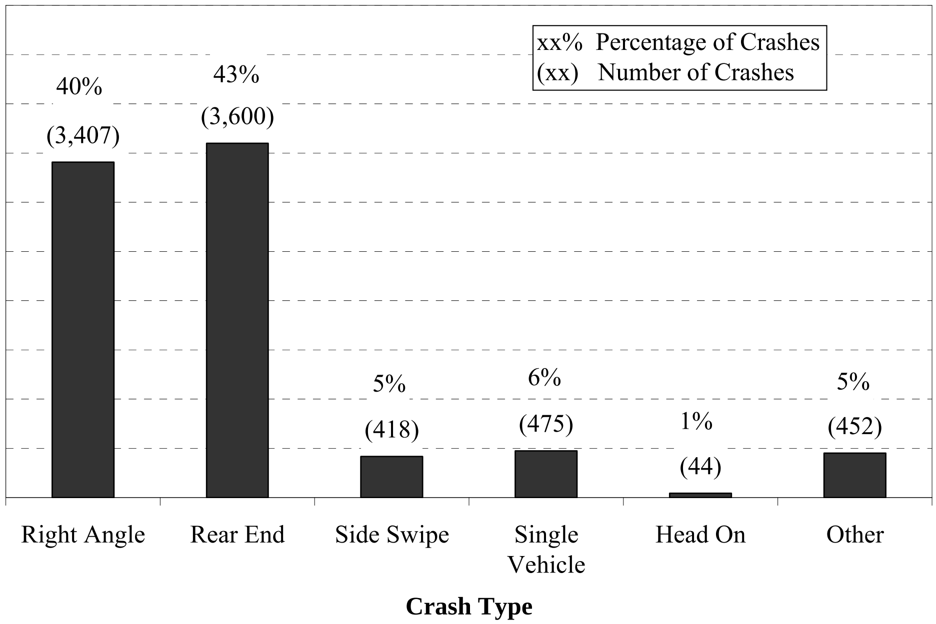

Figure 3.35 Distribution of crashes by crash type.

3.4 Summary of Data Collection

This chapter presented the purposes and methods utilized to gather information about accesses and crashes within intersection functional areas. A set of study sites and reference sites were identified, and both intersection data and crash date were obtained from each site. Chapter 4 presents the statistical analysis utilized to evaluate the relationships between access and crashes within intersection functional areas.

4 Intersection Analysis

In Chapter 3, the data collection process for the 144 study intersections and 15 reference intersections was documented. This chapter describes the statistical analyses conducted on the data to investigate the relationship between accesses and crashes within intersection functional areas. Section 4.1 discusses the statistical approach taken to analyze intersection functional area crash patterns. Section 4.2 documents the steps taken to prepare the data for analysis, including the development of independent and dependent variable sets. Study site analyses of each of the dependent crash variables are described in Sections 4.3 through 4.6, while Section 4.7 discusses the analyses of reference site data. Finally, Section 4.8 summarizes the findings of the intersection analyses.

4.1 Statistical Approach

Intersection crashes can be influenced by a wide variety of factors. As a result, the impact of access location was isolated from all other roadway and intersection characteristics. Stepwise variable selection and multiple linear regression analyses were employed to investigate the relationships between access location and intersection crashes.

In multiple linear regression analysis, a set of independent variables are calibrated to describe the mean of the distribution of a single dependent variable. One output of multiple linear regression analysis is a set of regression coefficients for the independent variables. Regression coefficients describe how independent variables are associated with the mean of the dependent variable in the context of the effects of all the other independent variables. Thus, regression coefficients may be unique from model to model, depending on what other independent variables have been included (Ramsey and

Schafer 2002). Multiple linear regression models may be expressed in a mathematical form as outlined in Equation 4.1. It should be noted that the mathematical regression equations presented within this study are not suitable for crash pattern prediction. Rather, the equations are provided only to serve as a visual comparison between variable regression coefficients.

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots + \beta_n X_n \tag{4.1}$$

where: Y = mean of the distribution of the dependent variable,

β_0 = constant,

β_i = regression coefficient of independent variable,

X_i = independent variable, and

n = number of independent variables.

For complex multiple linear regression, the set of independent variables can be quite large, so a sequential variable selection process, such as stepwise variable selection, may be used to reduce the independent variable set into a smaller group that contains those variables most significantly correlated with the dependent variable of interest. Stepwise variable selection starts out with a constant mean model and alternately adds and removes variables that meet or exceed a pre-determined significance level. The process is repeated until no independent variables can be added or removed from the model (Ramsey and Schafer 2002).

To isolate the impact of access location from other intersection characteristics, stepwise variable selection was conducted in two iterations. The first iteration was performed on only variables deemed to be non-access-related so as to produce a set of independent variables that describe intersection crashes according to non-access-related factors. The second iteration was then conducted on the variables determined to be access-related, while the non-access-related variables selected in the first iteration were manually included in the output variable set. In essence, the first iteration created a model with non-access-related variables, and the second iteration added to the model the access-related variables that provide further descriptive power. A discussion of the classification of access and non-access-related variables is given in Section 4.2.1.7.

By splitting the stepwise selection process into two iterations, the variable selection process functioned as a test for significance on the impact of access-related factors. The null hypothesis of the test was that intersection crashes are only associated with non-access-related factors, and a regression model representing such a relationship was produced from the first stepwise iteration. The alternative hypothesis was that intersection crashes are significantly associated with access-related factors after consideration of the non-access-related factors. Selection of access-related variables in the second stepwise iteration supported the alternative hypothesis. Such variables represented an access-related factor that had a significant role in describing the intersection crash variable even after the non-access-related factors were accounted for.

The final set of variables included both non-access-related and access-related variables that significantly described the intersection crash variable. Using the final independent variable output set, a multiple linear regression model was formed to evaluate the quantitative association these variables have with the intersection crash variable. This process was conducted for each dependent crash variable, as defined in Section 4.2.2.

The statistical software package NCSS (Hintze 2007) was used to perform the stepwise variable selection and the multiple linear regression analyses of the study data. For the stepwise variable selection, the significance level for both the addition and removal of variables was set at a *p*-value of 0.05.

4.2 Data Preparation

To prepare the data for statistical analysis, the intersection and crash data described in Chapter 3 were organized into sets of independent and dependent variables, respectively. The following sections identify the independent and dependent variables used in the statistical analysis and discuss how they were prepared for analysis.

4.2.1 Independent Variables

Independent variables were obtained from the information acquired in the intersection data collection process, as described in Section 3.2. To prepare for statistical analyses, independent variables were created from a three step process. First, the raw intersection data were refined to form independent variables suitable for multiple linear regression analysis. For example, because multiple linear regression can only be conducted on quantitative inputs, qualitative variables were converted into a quantitative form. Also, uneven distribution among some independent variables required reclassification of distribution break points to create more normally distributed data. Finally, variable adjustments were made to account for the effect of incomplete minor-street AADT data, the variability between opposing major-street approaches at the same intersection, and access land use. The second step in the independent variable preparation process was to classify independent variables as either being access-related or non-access-related. The final step was to compute correlation coefficients for all independent variables to reduce the possibility of redundancy in the regression models. The following sections document the variables that required refinement from the raw data, identify the access-related and non-access-related variables, discuss the correlation coefficient analysis, and present a final listing of the independent variables used for analysis.

4.2.1.1 AADT

AADT volumes were obtained for every study site major street. As previously mentioned in Section 3.2.2.3, minor-street AADT volumes were not available for every study site. Therefore, minor-street AADT could not be included as a variable in the statistical analysis. However, because the volumes experienced on the crossroad can have a major bearing on the number of potential intersection crashes, other factors were utilized to account for minor-street volumes. These factors included the number of minor-street through lanes and major-street and minor-street left-turn protection phasing.

The number of minor-street through lanes and the presence and type of left-turn protection phasing were obtained to help account for minor-street AADT volumes. Figure 4.1 shows how minor-street volumes vary by minor-street size. As can be seen from Figure 4.1, more minor-street through lanes are correlated with higher minor-street AADT volumes for the 63 study sites where minor-street AADT volumes were available.

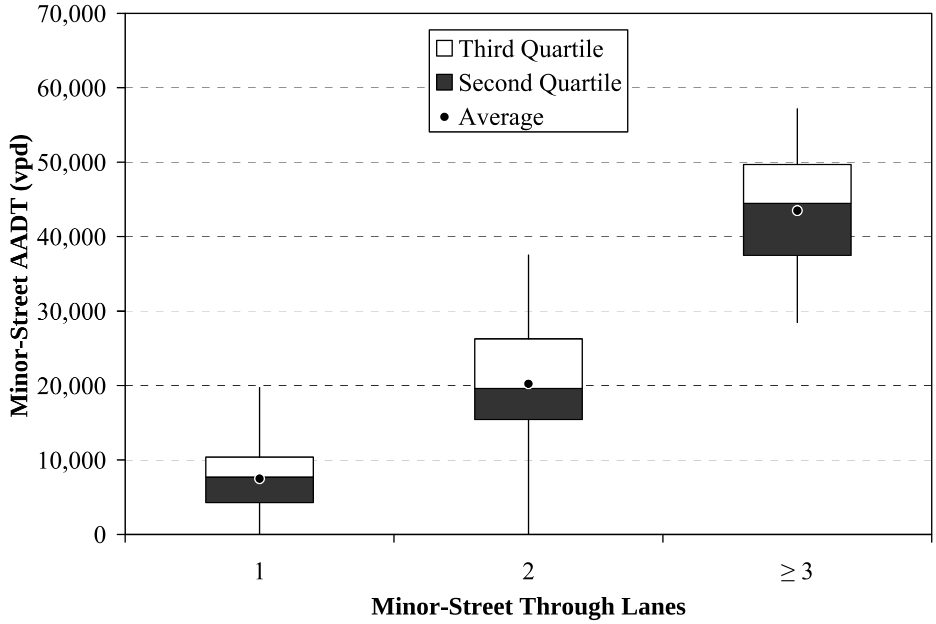

Figure 4.1 Box plot of minor-street AADT and minor-street through lanes.

Figures 4.2 and 4.3 show how minor-street volumes vary by minor-street left-turn phasing and major-street left-turn phasing, respectively. As evident in Figures 4.2 and 4.3, when minor- and major-street left-turn phasing progresses from permitted to protected-permitted to protected phasing, minor-street AADT volumes tend to increase. These relationships were assumed to be typical of all study sites. In summary, the number of minor-street though lanes and left-turn protection variables were used as indicators of minor-street AADT.

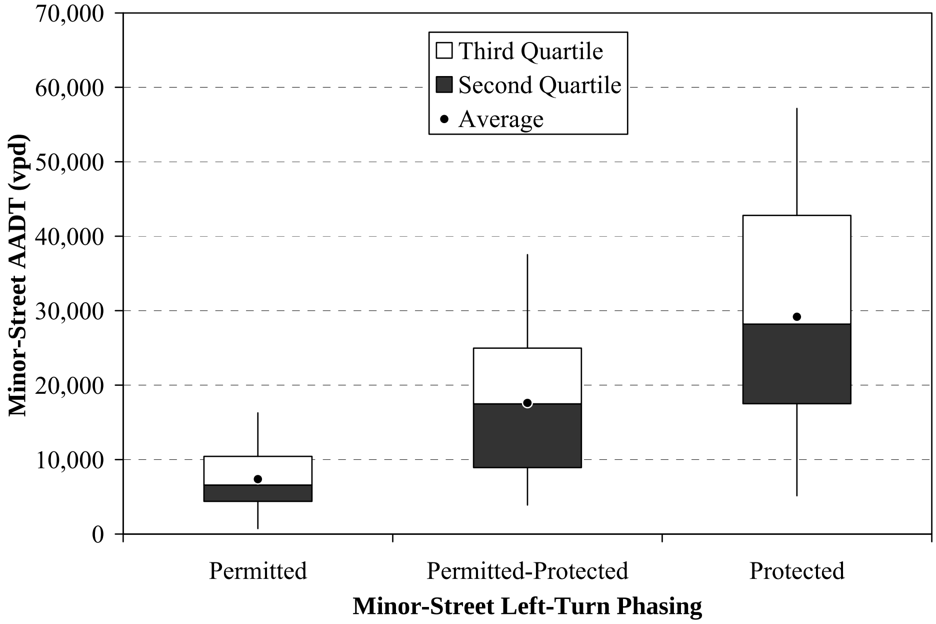

Figure 4.2 Box plot of minor-street AADT and minor-street left-turn phasing.

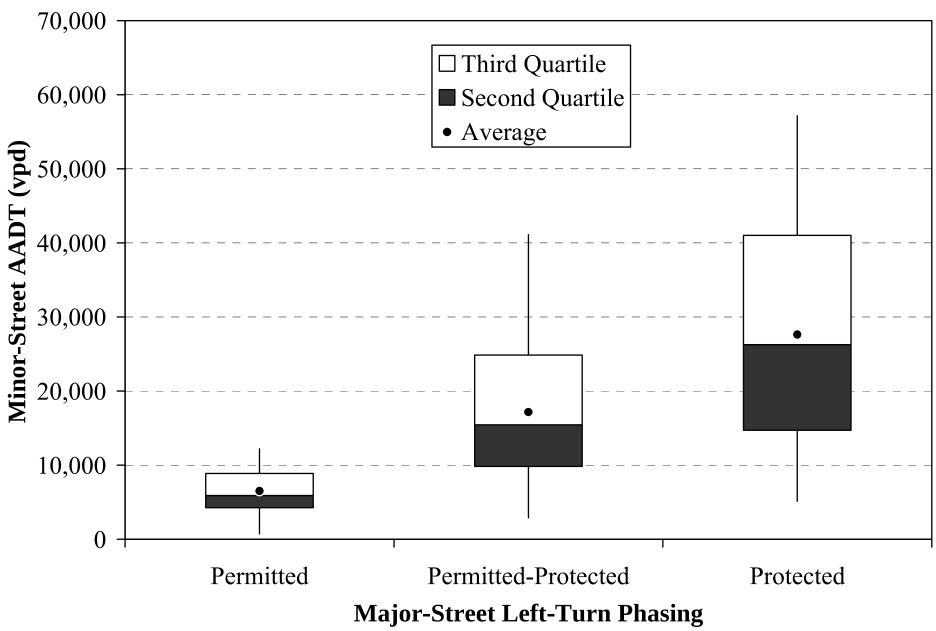

Figure 4.3 Box plot of minor-street AADT and major-street left-turn phasing.

4.2.1.2 Median Type

In order to account for intersections with varying median types, the intersection major-street median data were consolidated into a single median score. The median score takes into account the number of intersection major-street approaches that feature a raised median. For example, an intersection with a raised median on both major-street approaches would receive a score of 2. An intersection with a raised median on one approach and a TWLTL or no median on the other approach receives a score of 1. Finally, an intersection with a TWLTL or no median on both major-street approaches receives a score of 0. Figure 4.4 shows the distribution of study intersections according to median score. As can be seen from Figure 4.4, three-quarters of the study sites had a median score of 0, meaning that no raised medians are present on any of the major-street approaches.

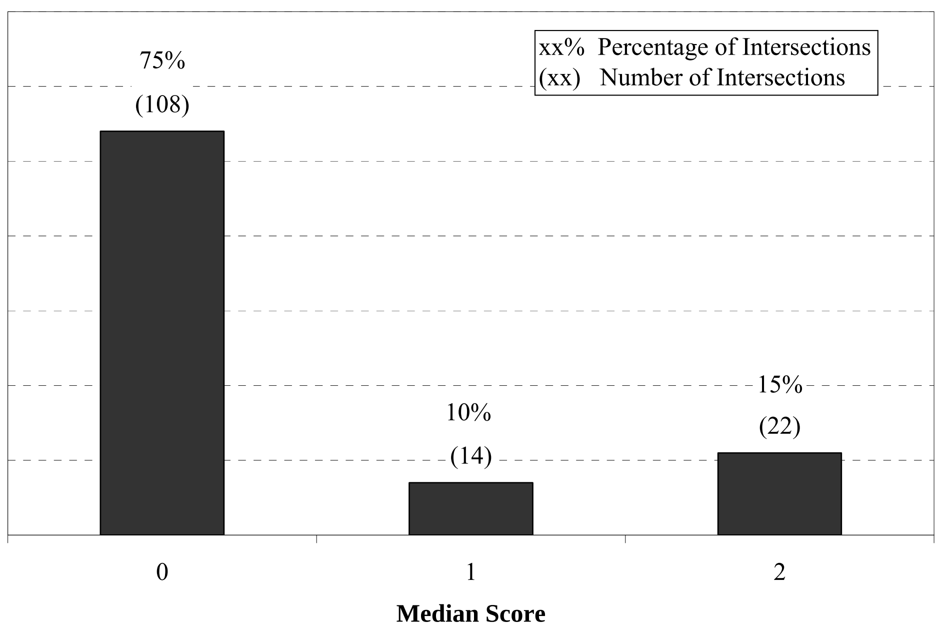

Figure 4.4 Distribution of study intersections by median score.

4.2.1.3 Corner Clearance

Each study intersection contained two major-street corner clearance measurements. As with the median data, the corner clearance information was simplified into a single variable. An average corner clearance measurement was examined but proved to be infeasible because of large outliers in the corner clearance distribution. For example, an intersection with a 50-foot corner clearance on one approach clearly has at least one driveway that may cause severe interference with intersection operations. Supposing the opposite approach had a corner clearance of 1,000 feet, the average corner clearance for the intersection would be computed as 525 feet. If a second intersection were to feature two corner clearances of 525 feet each, this intersection would also produce a 525-foot average corner clearance. Although both intersections produce identical average corner clearances, the effect of the driveways at each intersection are not the same. Because the 1,000-foot corner clearance skews the average corner clearance at the first intersection, the effect of the 50-foot corner clearance is masked.

Since the average corner clearance is not a reliable value, an approach similar to the median type variable was taken in the development of a corner clearance score. To compute the corner clearance score, each major-street approach-side corner clearance was evaluated as to whether it complies with the UDOT corner clearance standards (UDOT 2006a), previously discussed in Section 2.2. Every corner clearance that failed to meet UDOT standards increased the corner clearance score by 1. For example, an intersection with both major-street approach-side corner clearances below the UDOT standards receives a score of 2. An intersection with one corner clearance below UDOT standards and one corner clearance that meets UDOT standards receives a score of 1. Finally, an intersection with corner clearances that both meet UDOT standards receives a score of 0. Figure 4.5 shows the distribution of corner clearance scores. Almost half of all study sites had a corner clearance score of 2, meaning both major-street approach-side corner clearances failed to meet UDOT corner clearance standards. Only 15 percent of study sites met UDOT requirements on both major-street approach-side corner clearances.

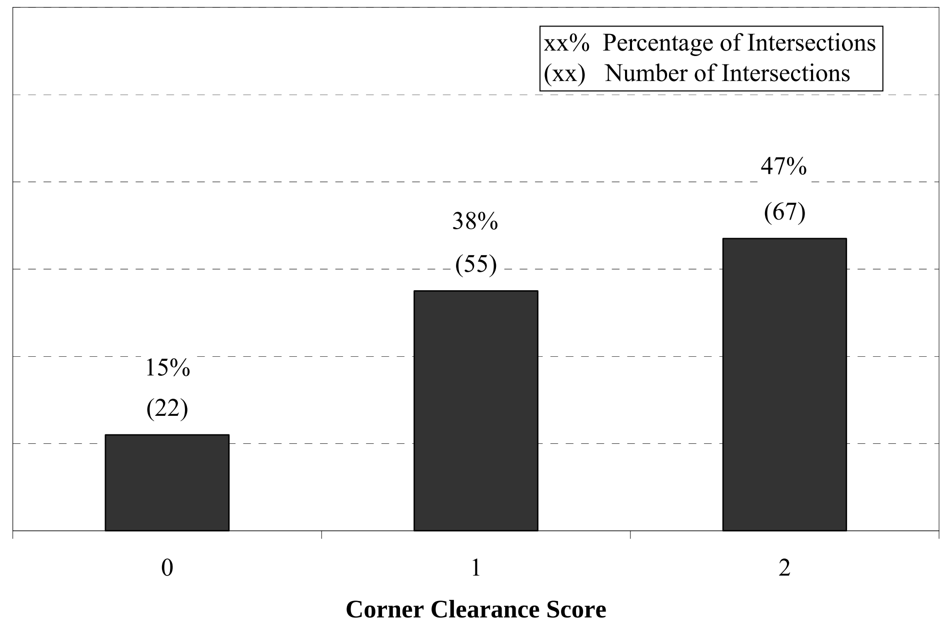

Figure 4.5 Distribution of study intersections by corner clearance score.

4.2.1.4 Access and Conflict Point Densities

As discussed in Section 2.1.2.2, land use has been found to be an important factor in the relationship between access density and roadway safety. Because residential accesses tend to have closer spacing and lower driveway volumes, a given residential access density will not have as severe an impact on safety as an equivalent access density of commercial driveways (Schultz and Braley 2007; Schultz and Lewis 2006). To account for the relative impacts of driveway land use in the linear regression model, access density was split into two variables: commercial access density and residential access density. All accesses at a study intersection were grouped into commercial or residential categories utilizing the data obtained from the method described in Section 3.2.5.5. The intersection functional area access density was then calculated for each land use type. Following the same process, commercial conflict point density and residential conflict point density were also obtained.

Figure 4.6 compares the distributions of commercial access density and residential access densities among study intersections. As can be seen from the figure, residential access densities tend to be much smaller than commercial access densities.

Only 11 of 144 study intersections feature a residential access density greater than 5 accesses per 1,000 feet.

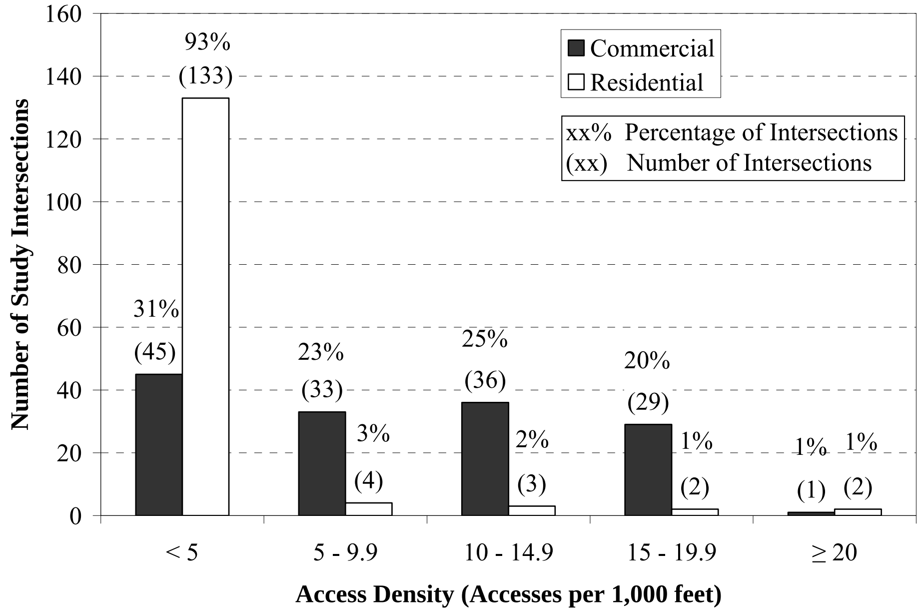

Figure 4.6 Distribution of study intersections by access density and land use.

Figure 4.7 compares the commercial conflict point density and residential conflict point density distributions among study intersections. The conflict point density distribution is similar to the access density distribution in that the residential densities are heavily weighted on the low end of the scale. Only 14 of 144 sites feature a residential conflict point density greater than 50 conflict points per 1,000 feet.

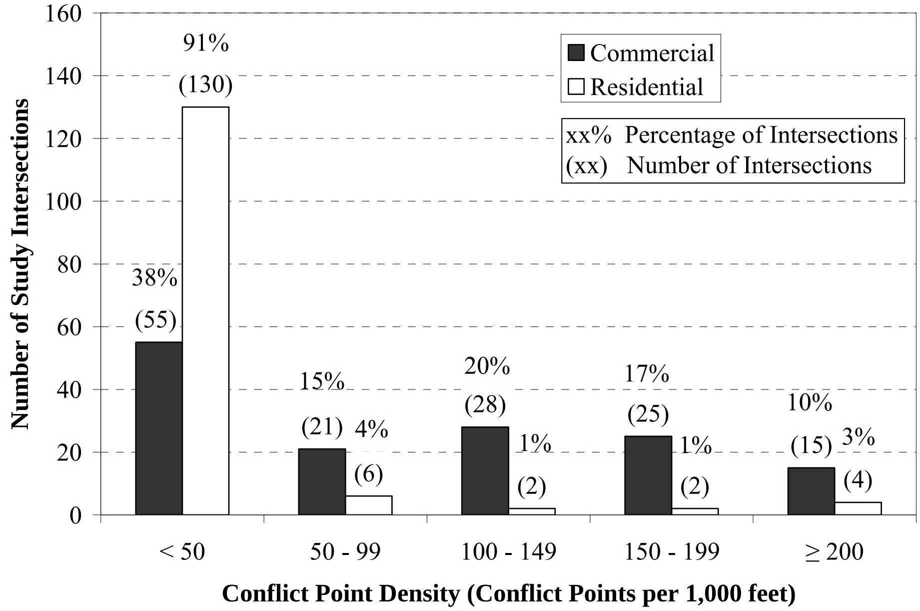

Figure 4.7 Distribution of study intersections conflict point density and land use.

4.2.1.5 Reclassified Variables

As presented in the intersection data collection process in Section 3.2, data distributions for some independent variables produced data categories that contained very few intersections. Small groups do not offer an adequate sample size to truly represent the actual distribution of the variable. Therefore, the variables that featured data categories with very few intersections were reclassified to more evenly redistribute the data. This process was conducted on the speed limit variable and both number of through lane variables.

First, as presented in Section 3.2.2.5, only two of the 144 study intersections feature a posted speed limit of 50 mph. Because of the small sample size of the category, the 50 mph speed limit data were combined with the 45 mph data and reclassified as a 45 mph or greater category. Second, only four study intersections contained one major-street through lane, and only one study intersection featured four minor-street through lanes. Therefore, the one major-street through lane category was combined with the two major-street through-lanes category to form a new category containing intersections with two or fewer major-street through lanes. Likewise, the four minor-street through-lanes category was pooled with the three minor-street through-lanes category and reclassified as a three-or-more minor-street through-lanes category.

4.2.1.6 Indicator Variables

In addition to median type and corner clearance, several more data categories needed to be converted into quantitative variables for regression analysis. These variables include access classification, functional classification, left-turn protection, proximity to freeway interchange, and functional area overlap. Each variable was converted into one or more indicator variables. Indicator variables hold either a value of 0 or 1 corresponding to the absence or presence of an attribute (Ramsey and Schafer 2002).

Because only two functional classification types were present within the dataset, principal arterial roadways and minor arterial roadways, the functional classification indicator variable was used to flag intersections on a principal arterial roadway. For example, an intersection on a principal arterial roadway received a value of 1 for the functional classification indicator variable while an intersection on a minor arterial roadway received a value of 0. Since four access classification categories existed within the dataset, the access classification variable was divided into four indicator variables corresponding with the UDOT access classification groups (UDOT 2006a). An intersection received a value of 1 for its corresponding access classification indicator variable and a value of 0 for all other access classification indicator variables.

Left-turn protection was also converted into an indicator variable. Preliminary examination of the data showed that crash totals were similar for intersections with protected-permitted left-turn phasing and for intersections with protected left-turn phasing. However, both protected-permitted left-turn intersection crash totals and protected left-turn intersection crash totals were different than crash totals for intersections with permitted left-turn phasing. Figures 4.8 and 4.9 illustrate the left-turn phasing relationship for major streets and minor streets, respectively.

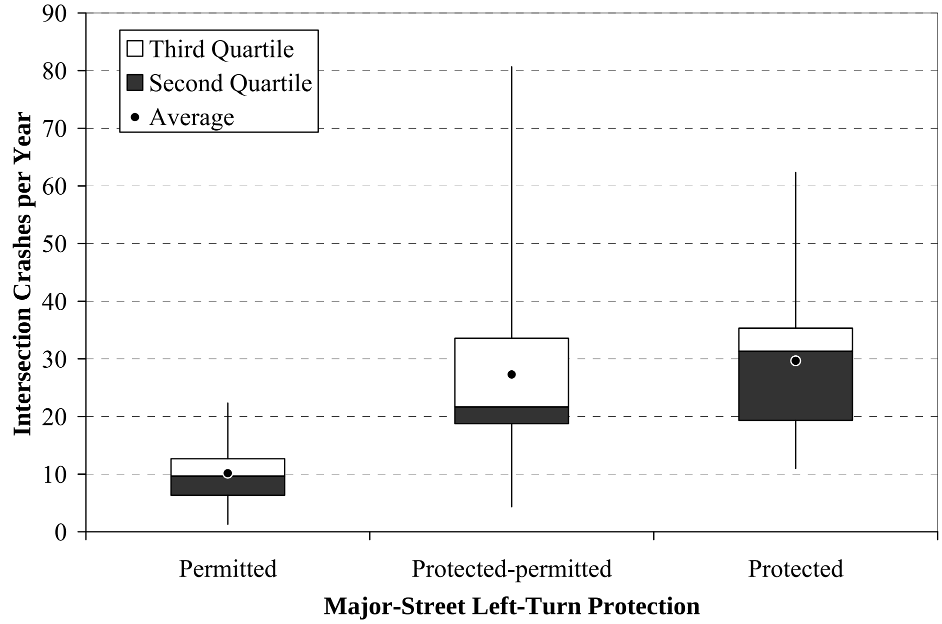

Figure 4.8 Box plot of major-street left-turn protection and intersection crashes.

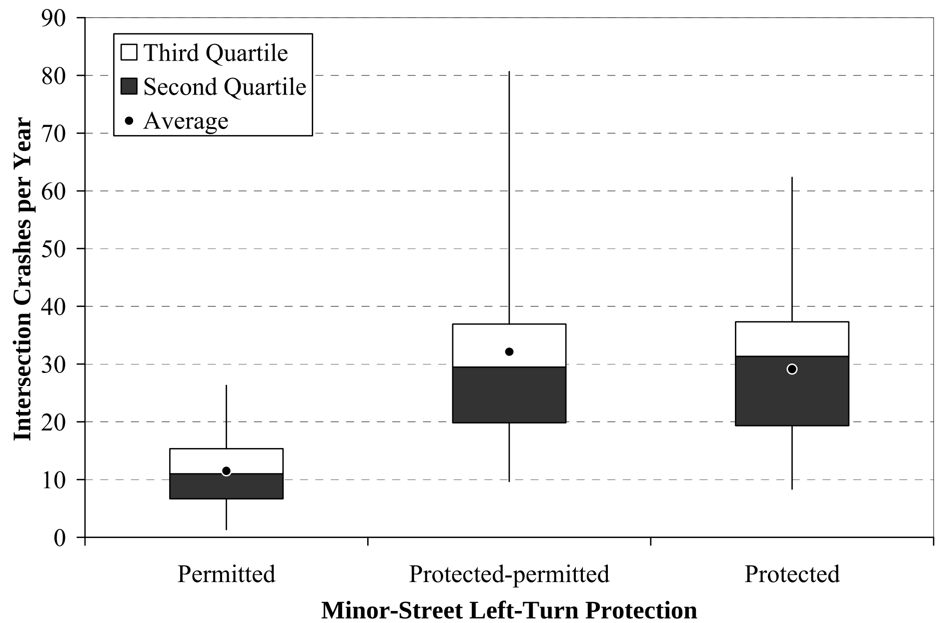

Figure 4.9 Box plot of minor-street left-turn protection and intersection crashes.

In consequence of these left-turn protection phasing relationships, the left-turn protection variable was consolidated into two groups: 1) no left-turn protection (permitted left-turn phasing only) and 2) some left-turn protection (protected-permitted or protected left-turn phasing). Intersections with no left-turn protection were assigned a value of 0, and intersections with some left-turn protection received a value of 1. This process was conducted for both the major-street left-turn protection variable and the minor-street left-turn protection variable.

To verify the assumption that approaches with some left-turn protection phasing feature different crash patterns than those with no left-turn protection, a Tukey-Kramer multiple-comparison test was conducted on the study data. The Tukey-Kramer procedure evaluates all pairwise differences against the means of each group (Hintze 2007). Table 4.1 presents the results from the Tukey-Kramer procedure. As can be seen from Table 4.1, both protected-permitted and protected phasing intersection crash totals were different than permitted left-turn phasing intersection crash totals. Furthermore, protected-permitted left-turn phasing intersection crash totals were not statistically different than protected phasing intersection crash totals. These results were consistent for both major streets and minor streets.

Finally, an intersection that featured an overlapping functional area with an adjacent intersection received a value of 1 for the functional area overlap indicator variable. An intersection that was in proximity to a freeway interchange, as defined in Section 3.2.2.6, received a value of 1 for the proximity to a freeway indicator variable.

4.2.1.7 Identification of Access-Related and Non-Access-Related Variables

Non-access-related variables were defined as variables that do not describe the location or type of accesses within the intersection functional area. A listing and description of the non-access-related variables is contained in Table 4.2. All other independent variables were then defined as access-related variables. A listing and description of each access-related variable is contained in Table 4.3.

Table 4.1 Tukey-Kramer Multiple-Comparison Test Results

Major Street					
Left-Turn Phasing	**Count**	**Average Crashes**	**Different from Groups**	**Test Parameters**	
Permitted	69	10.2	Protected-permitted, Protected	Degrees of Freedom	141
Protected-permitted	54	27.3	Permitted	Mean Square Error	124.8
Protected	21	29.7	Permitted	Critical Value	3.35
Minor Street					
Left-Turn Phasing	**Count**	**Average Crashes**	**Different from Groups**	**Test Parameters**	
Permitted	85	11.5	Protected-permitted, Protected	Degrees of Freedom	141
Protected-permitted	34	32.1	Permitted	Mean Square Error	112.7
Protected	25	29.1	Permitted	Critical Value	3.35

Table 4.2 Description of Non-Access-Related Independent Variables

Non-Access-Related Variables	Description
AADT	Average AADT per major-street approach
Speed	Posted speed limit at intersection
Major-Street Left-Turn (LT) Protection	1 = major street has protected-permitted or protected left-turn phasing; 0 = major street has permitted left-turn phasing
Minor-Street LT Protection	1 = minor street has protected-permitted or protected left-turn phasing; 0 = minor street has permitted left-turn phasing
Major-Street Through Lanes	Number of through lanes on the major-street approaches
Minor-Street Through Lanes	Number of through lanes on the minor-street approaches
Functional Area (FA) Length	Total length of the intersection functional area
FA Overlap	1= the functional area overlaps with the functional area of an adjacent intersection; 0 = otherwise
Freeway	1 = the intersection is in proximity to a freeway interchange; 0 = otherwise
Functional Class	1 = the roadway is a principal arterial; 0 = the roadway is a minor arterial

Table 4.3 Description of Access-Related Independent Variables

Access-Related Variables	Description
Commercial Access Density	Density of commercial accesses within the functional area
Residential Access Density	Density of residential accesses within the functional area
Commercial Conflict Point (CP) Density	Density of commercial conflict points within the functional area
Residential CP Density	Density of residential conflict points within the functional area
Median Score	Number of major-street approaches featuring a raised median
Corner Clearance Score	Number of approach corner clearances in violation of UDOT corner clearance standards[1]
Access Cat 4	1 = the roadway is UDOT Access Category 4 (Regional Rural)[1]; 0 = otherwise
Access Cat 5	1 = the roadway is UDOT Access Category 5 (Regional Priority Urban)[1]; 0 = otherwise
Access Cat 6	1 = the roadway is UDOT Access Category 6 (Regional Urban)[1]; 0 = otherwise
Access Cat 7	1 = the roadway is UDOT Access Category 7 (Community Rural)[1]; 0 = otherwise

1. UDOT 2006a

4.2.1.8 Correlation Coefficient Analysis and Summary of Independent Variables

Correlation between all independent variables was examined to determine whether any independent variables provided redundant information. Correlation coefficients measure the amount of linear correlation between two quantitative variables and range between values of -1.0 and 1.0. A correlation coefficient of 1.0 represents perfect positive correlation, while a correlation coefficient of -1.0 represents perfect negative correlation. A 0.0 correlation coefficient indicates no correlation between variables. Correlation coefficients were calculated for each independent variable pair and are presented in Table 4.4. Variables with correlation coefficients greater than or equal to 0.8 or less than or equal to -0.8 were flagged as containing redundant information, and one of the variables was removed from the analysis. Table 4.4 displays significant correlation coefficients in bold-face type for emphasis. As can be seen from the table, the Functional Area (FA) Length and Speed variables produce a correlation coefficient of 0.80. This is expected since speed is the primary input in the functional distance calculation. The FA Length variable was excluded from the statistical analysis because it contained more outliers than the Speed variable. High correlation was also observed among the density variables. The Commercial Access Density variable and the Commercial Conflict Point (CP) Density variable featured a correlation coefficient of 0.85, while the Residential Access Density variable and the Residential CP Density variable had a correlation coefficient of 0.99. Since access density is the more commonly used measurement within the literature, the conflict point density variables were removed from the statistical analyses. Table 4.5 summarizes the final listing of independent variables utilized in the statistical analyseis.

Table 4.4 Correlation Coefficients of Independent Variables

	AADT	Speed	Major-Street LT Protection	Minor-Street LT Protection	Major-Street Through Lanes	Minor-Street Through Lanes	FA Length	FA Overlap	Freeway	Functional Class	Commercial Access Density	Commercial CP Density	Residential Access Density	Residential CP Density	Median Score	Corner Clearance Score	Access Cat 4	Access Cat 5	Access Cat 6	Access Cat 7
AADT	1.00																			
Speed	0.18	1.00																		
Major-Street LT Protection	0.26	0.41	1.00																	
Minor-Street LT Protection	0.16	0.42	0.77	1.00																
Major-Street Through Lanes	0.37	0.29	0.31	0.22	1.00															
Minor-Street Through Lanes	0.09	0.16	0.44	0.53	0.17	1.00														
Functional Area Length	0.21	**0.80**	0.50	0.54	0.39	0.34	1.00													
Functional Area Overlap	0.35	0.03	0.01	-0.00	0.02	-0.03	0.08	1.00												
Freeway	0.13	0.17	0.22	0.27	0.16	0.09	0.38	0.35	1.00											
Functional Class	-0.07	-0.21	-0.35	-0.23	-0.21	-0.16	-0.27	0.02	0.10	1.00										
Commercial Access Density	-0.15	-0.32	-0.05	-0.07	-0.01	0.05	-0.25	-0.17	-0.19	-0.07	1.00									
Commercial CP Density	-0.16	-0.36	-0.19	-0.18	-0.13	-0.12	-0.40	-0.14	-0.13	0.14	**0.85**	1.00								
Residential Access Density	0.13	0.01	-0.25	-0.25	-0.15	-0.19	-0.11	0.04	-0.08	0.08	-0.25	-0.21	1.00							
Residential CP Density	0.11	0.01	-0.27	-0.25	-0.15	-0.20	-0.12	0.01	-0.08	0.11	-0.25	-0.19	**0.99**	1.00						
Median Score	0.17	0.33	0.49	0.48	0.50	0.43	0.53	-0.07	0.05	-0.44	0.00	-0.36	-0.14	-0.16	1.00					
Corner Clearance Score	-0.00	-0.05	0.03	0.05	0.05	0.08	-0.03	-0.14	-0.14	-0.25	0.57	0.45	0.09	0.09	0.13	1.00				
Access Cat 4	0.03	0.16	-0.01	-0.06	-0.08	-0.10	0.14	0.11	-0.04	0.08	-0.10	-0.06	-0.03	-0.04	-0.09	0.04	1.00			
Access Cat 5	0.15	0.69	0.51	0.44	0.41	0.29	0.59	-0.05	0.18	-0.38	-0.07	-0.20	-0.18	-0.19	0.47	0.13	-0.15	1.00		
Access Cat 6	-0.13	-0.79	-0.36	-0.34	-0.17	-0.12	-0.66	0.10	-0.11	0.35	0.16	0.25	-0.03	-0.03	-0.29	-0.06	-0.13	-0.71	1.00	
Access Cat7	-0.05	0.04	-0.22	-0.13	-0.32	-0.20	0.02	-0.12	-0.09	0.02	-0.08	-0.04	0.31	0.32	-0.22	-0.12	-0.07	-0.36	-0.32	1.00

Table 4.5 Summary of Independent Variables

	Variable	Units	Range	Average
Non-Access-Related Variables	AADT	thousands of vpd	5.200 to 54.723	30.258
	Speed	mph	30, 35, 40, ≥ 45	38
	Major-Street LT Protection	--	0 or 1	--
	Minor-Street LT Protection	--	0 or 1	--
	Major-Street Through Lanes	lanes	≤ 2 or ≥3	2.3
	Minor-Street Through Lanes	lanes	1, 2, ≥3	1.3
	FA Overlap	--	0 or 1	--
	Freeway	--	0 or 1	--
	Functional Class	--	0 or 1	--
Access-Related Variables	Commercial Access Density	accesses / 1,000 feet	0.0 to 20	8.9
	Residential Access Density	accesses / 1,000 feet	0.0 to 25	1.5
	Median Score	--	0, 1, 2	0.4
	Corner Clearance Score	--	0, 1, 2	1.3
	Access Cat 4	--	0 or 1	--
	Access Cat 5	--	0 or 1	--
	Access Cat 6	--	0 or 1	--
	Access Cat 7	--	0 or 1	--

4.2.2 Dependent Variables

Dependent variables were selected from the results of the crash data collection process, as described in Section 3.3. As with the independent variable data, the dependent variable data required refinement in order to be suitable for statistical analysis.

First, since crash total data, severity cost data, and crash type data were obtained as three-year totals, each value was divided by three to produce one-year averages.

Second, because of the concentrated size of intersection functional areas, some crash types did not exhibit sufficient frequency to merit a meaningful statistical analysis. As presented in Table 4.6, of the six crash types identified in Section 3.3.4, only right-angle crashes and rear-end crashes regularly occurred at study site intersections. All other crash types averaged approximately 1 crash per year or less. Since stepwise variable selection and multiple linear regression results from these data would be unreliable, only the right-angle crash and rear-end crash types were included in the statistical analyses. Furthermore, right-angle and rear-end crashes are the crash types of most interest as they are correlated with crash severity. Right-angle crashes tend to be one of the most severe crashes, while rear-end crashes tend to be one of the least severe.

Table 4.6 Frequency of Intersection Crashes by Crash Type

Crash Type	Crashes Per Year within Intersection Functional Areas		
	Minimum	Maximum	Average
Right Angle	0.67	43.3	7.89
Rear End	0.00	31.2	8.34
Side Swipe	0.00	7.67	0.968
Single Vehicle	0.00	7.33	1.10
Head On	0.00	0.67	0.102
Other	0.00	5.67	1.05

Finally, initial investigation of the remaining dependent variable data revealed skewed distributions and the presence of outliers. Since multiple linear regression operates with the assumption of constant variation among variables, a natural log transformation was conducted on each dependent variable to reduce the effect of the outliers and provide the data with more normal distributions. Figures 4.10 through 4.14 compare the dependent variable distributions before and after the natural log transformation. Table 4.7 summarizes the dependent variables used in the statistical analyses.

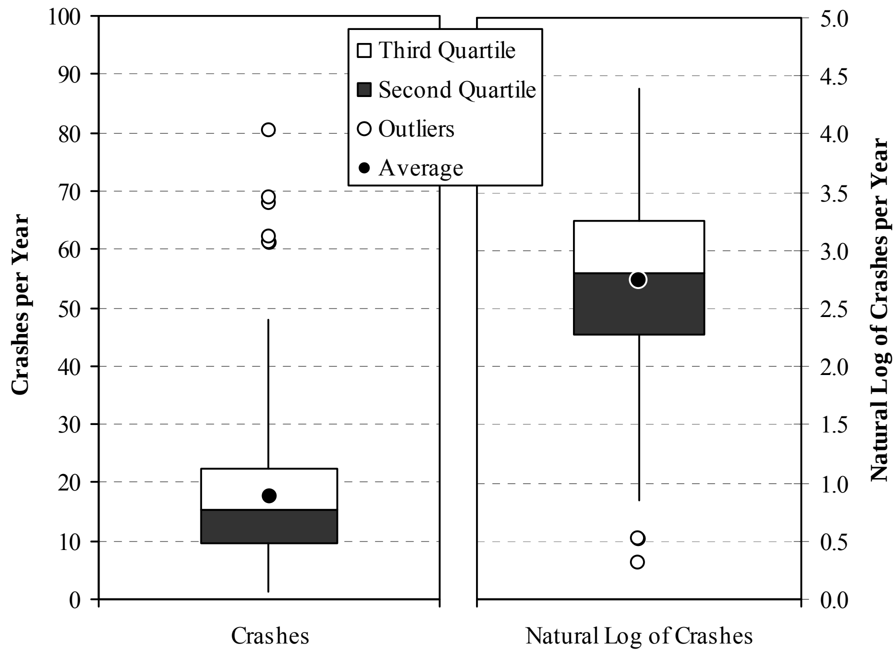

Figure 4.10 Comparison of crash-totals distributions.

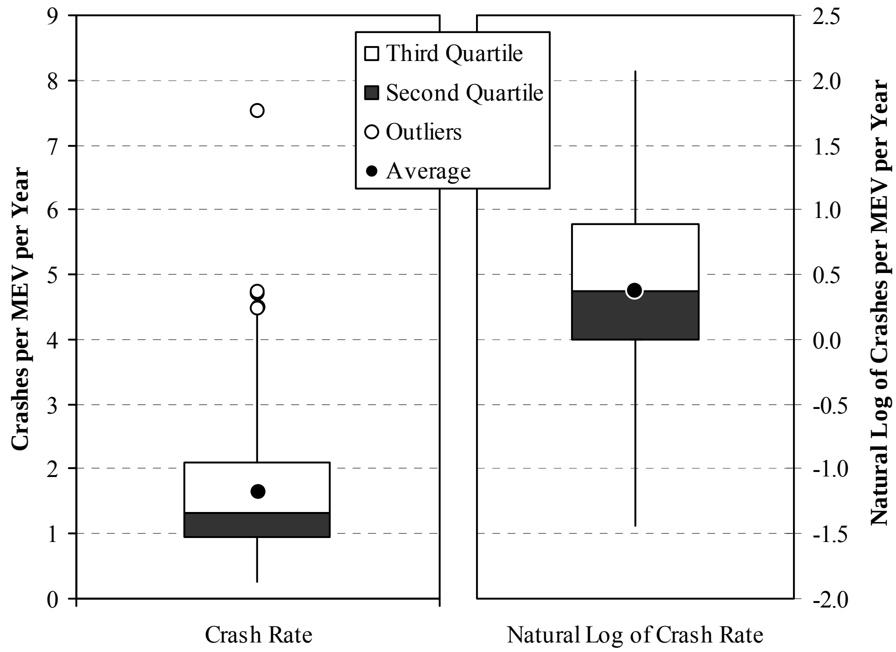

Figure 4.11 Comparison of crash-rate distributions.

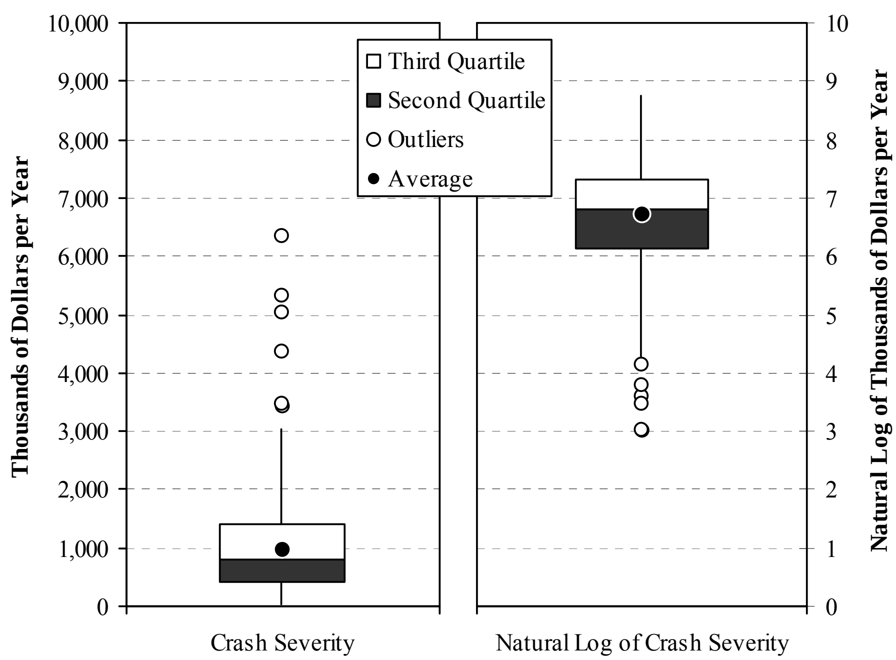

Figure 4.12 Comparison of crash-severity distributions.

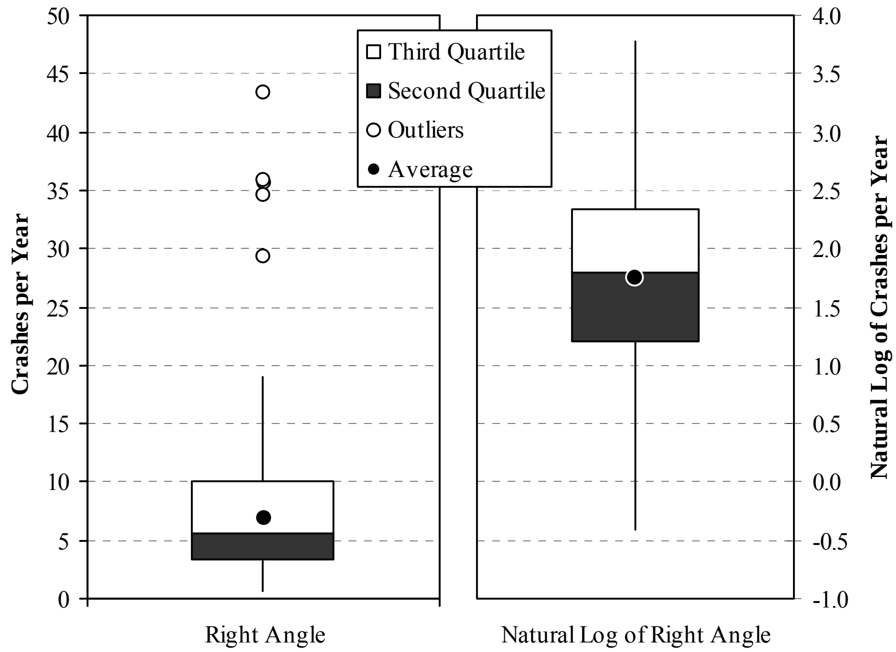

Figure 4.13 Comparison of right-angle-crash distributions.

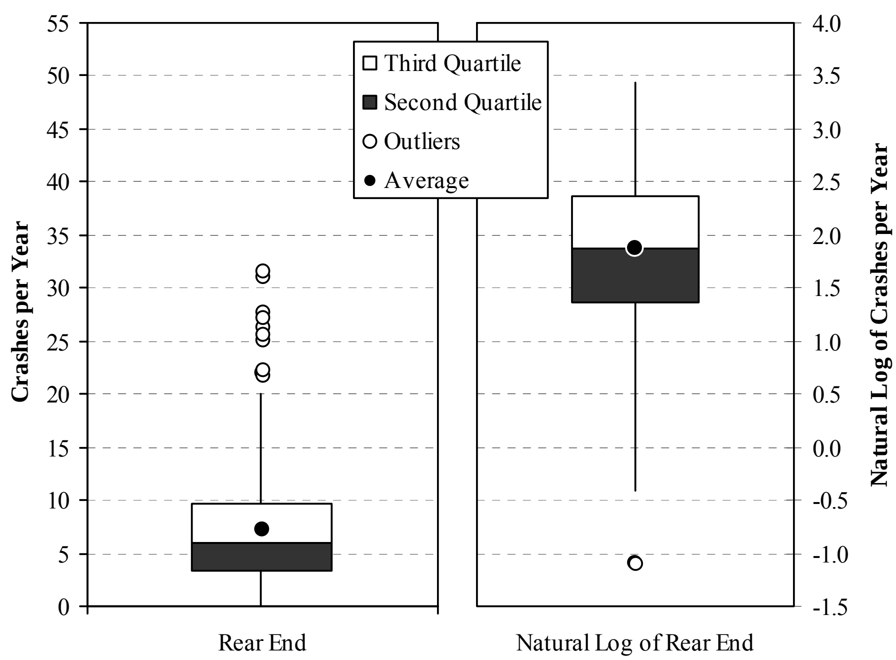

Figure 4.14 Comparison of rear-end-crash distributions.

Table 4.7 Summary of Dependent Variables

Variable	Description	Pre-Natural Log Transformation			Post-Natural Log Transformation	
		Units	Range	Avg	Range	Avg
Crashes Totals	Total crashes in the functional area	crashes / year	1.33 to 80.7	19.4	0.288 to 4.39	2.71
Crash Rate	Functional area crash rate according to major-street volumes only	crashes / year / MMSEV	0.247 to 7.51	1.75	-1.40 to 2.02	0.37
Crash Severity	Total costs of crashes in the functional area according to severity	thousands of dollars / year	19.9 to 6,305.2	1,109.0	3.00 to 8.75	6.61
Right Angle	Right-angle crashes in the functional area	crashes / year	0.667 to 43.3	7.89	-0.406 to 3.77	1.75
Rear End	Rear-end crashes in the functional area	crashes / year	0.00 to 31.3	8.34	-1.10 to 3.44	1.83

4.3 Crash Totals

As outlined in Section 4.1, stepwise variable selection was conducted on the non-access-related variables for the natural log of the number of crashes per year occurring in the intersection functional area. The following non-access-related variables were significant at the 95th percentile level:

- AADT
- Minor-Street Through Lanes
- Major-Street LT Protection
- Minor-Street LT Protection

Manually inserting the above non-access-related variables into the output variable set, the second stepwise selection iteration was then conducted including the access-related variables as potential model variables. In addition to the non-access-related variables, the following access-related variable was selected as a significant factor in describing functional area crash totals at the 95th percentile significance level:

- Commercial Access Density

A multiple linear regression model was formed with the four non-access-related and one access-related variables. Table 4.8 presents the regression coefficients, standard errors, *t*-statistics, *p*-values, and the *R*-squared term for the multiple linear regression model.

Table 4.8 Multiple Linear Regression Model for Crash Total Variable

Variable		Regression Coefficient	Standard Error	*t*-statistic	*p*-value
	(Constant)	0.880	0.157	5.63	< 0.01
Non-Access-Related	AADT	0.0323	0.0040	8.09	< 0.01
	Minor-Street Through Lanes	0.185	0.0714	2.59	0.01
	Major-Street LT Protection	0.371	0.112	3.30	< 0.01
	Minor-Street LT Protection	0.539	0.119	4.55	< 0.01
Access-Related	Commercial Access Density	0.0218	0.0059	3.68	< 0.01
R-squared = 0.70					

Equation 4.2 shows the multiple regression model in mathematical form while graphical relationships between variables are shown in Appendix I. As discussed in

Section 4.1, the regression equations presented in this study are not suitable for prediction purposes.

$$\begin{aligned}\ln(\textit{crashes per year}) = 0.880 + 0.0323X_1 + 0.185X_2 + 0.371X_3 \\ + 0.539X_4 + 0.0218X_5\end{aligned} \tag{4.2}$$

where: X_1 = AADT (thousands of vpd),

X_2 = Minor-Street Through Lanes,

X_3 = Major-Street LT Protection,

X_4 = Minor-Street LT Protection, and

X_5 = Commercial Access Density (accesses per 1,000 feet).

As seen in Table 4.8, the Commercial Access Density variable was identified as being significantly related to functional area crashes even after non-access-related factors are considered. As previously discussed in Section 4.1, because the stepwise selection was conducted in two iterations, the second iteration functions as a test for significance on the association between access-related factors and the dependent crash variables after accounting for non-access-related factors. Hence, the access-related variables included in the regression model may be interpreted as factors that have a significant correlation with the dependent crash variable. Thus, for study site intersections, commercial access density has a significant association with functional area crashes.

Conclusions may also be formed regarding the signs of the access-related regression coefficients. Although the exact value of the regression coefficients can change with the particular variable selection set, the signs of the regression coefficients are expected to remain constant for most models. Thus, the magnitude of the association between independent and dependent variables may be evaluated with reasonable certainty as being either positive or negative. In essence, an increase in the independent variable may be determined as to whether it is associated with either a positive change or a negative change in the dependent crash variable. As seen in Table 4.8, because the sign of the Commercial Access Density regression coefficient is positive, an increase in commercial access density is associated with an increase in the natural log of functional area crashes.

Although conclusions are formed regarding the selection of access-related variables and the signs of their regression coefficients, caution must be exercised when applying the same techniques to the non-access-related variables. First, rather than establishing a single, definitive model for the data, variable selection techniques, such as stepwise selection, can output more than one valid regression model. The particular set of selected non-access-related variables is likely one of several useful sets for describing the data on hand (Ramsey and Schafer 2002). Therefore, the inclusion or exclusion of a particular non-access-related variable cannot be viewed as representative of a law of nature or a cause-and-effect relationship. Second, a regression coefficient is defined as the numeric association a unit change in the independent variable has with the average value of the dependent variable, assuming all other variables are held constant. Because of pre-existing relationships among independent variables, the ability to change one variable within a large dataset while holding all others constant becomes unrealistic (Ramsey and Schafer 2002). For example, at signalized intersections, left-turn phasing is associated with traffic volumes. Assuming that a vast increase in AADT would not be accompanied by a change in the type of left-turn protection is not reasonable. As with access-related variables, interpretation of regression coefficients is limited to the examination of whether the coefficient is positive or negative, since this is expected to remain constant from model to model. Additional caution regarding a strict interpretation of the regression coefficient is also supported by the fact that the Crash Totals variable has undergone a natural log transformation. Evaluating the true relationship between independent variables and a transformed dependent variable requires a back-transformation of the dependent variable.

In lieu of these cautions, the following conclusions can be made regarding the non-access-related variables chosen for the crash totals model. Table 4.8 illustrates that the four non-access-related variables inputted into the model each had positive regression coefficients. For the AADT and the Minor-Street Through Lanes variables, the relationship is intuitive. Increased major-street and minor-street traffic volumes is expected to correlate with increased crashes. The left-turn variables, however, represent a more complex relationship. As discussed in Section 3.2.2.4, the left-turn protection data were obtained as indicators of turning movement patterns. Intersections with left-turn protection were assumed to experience greater left-turn volumes, which lead to increased turning traffic conflicts. Consequently intersections with left-turn protection are assumed to have more crashes. These assumptions are supported by the UDOT warranting process (UDOT 2006d) which requires intersections to feature specific left-turn volumes, left-turn crash rates, or insufficient left-turn storage in order to warrant left-turn protection. As presented in Section 4.2.1.6, the level of left-turn protection increased with minor-street volumes for study site intersections. Thus, although the positive left-turn variable regression coefficient appears to indicate that left-turn protection is associated with increased crashes, the factors represented by the left-turn variables are more likely the root of the association. Namely, the positive left-turn variable regression coefficients indicate that the increase in left-turn volumes, left-turn crashes, or minor-street traffic volumes are associated with increases in functional area crashes.

4.4 Crash Rate

Stepwise variable selection was conducted on the non-access-related variables for the natural log of the adjusted intersection crash rate. The following non-access-related variables were significant at the 95th percentile level:

- Minor-Street Through Lanes
- Major-Street LT Protection
- Minor-Street LT Protection

Manually inserting the above non-access-related variables into the final output, the second stepwise selection iteration was conducted including the access-related variables as potential model variables. In addition to the above variables, the following access-related variable was selected as a significant factor in describing functional area crash rates at the 95th percentile significance level:

- Commercial Access Density

A multiple linear regression model was formed with the three non-access-related and one access-related variables. Table 4.9 presents the regression coefficients, standard errors, *t*-statistics, *p*-values, and the *R*-squared term for the multiple linear regression model.

Table 4.9 Multiple Linear Regression Model for Crash Rate Variable

Variable		Regression Coefficient	Standard Error	*t*-statistic	*p*-value
	(Constant)	-0.454	0.101	-4.50	< 0.01
Non-Access-Related	Minor-Street Through Lanes	0.176	0.0707	2.49	0.01
	Major-Street LT Protection	0.307	0.108	2.83	< 0.01
	Minor-Street LT Protection	0.552	0.117	4.71	< 0.01
Access-Related	Commercial Access Density	0.0224	0.0058	3.87	< 0.01
R-squared = 0.58					

Equation 4.3 shows the multiple regression model in mathematical form while graphical relationships between variables are shown in Appendix J. As discussed in Section 4.1, the regression equations presented in this study are not suitable for prediction purposes.

$$\begin{aligned}\ln(crash\,rate) = & -0.454 + 0.176X_1 + 0.307X_2 + 0.552X_3 \\ & + 0.0224X_4\end{aligned} \tag{4.3}$$

where: X_1 = Minor-Street Through Lanes,

X_2 = Major-Street LT Protection,

X_3 = Minor-Street LT Protection, and

X_4 = Commercial Access Density (accesses per 1,000 feet).

As seen in Table 4.9, the multiple linear regression model for the natural log of the adjusted functional area crash rate includes three non-access-related variables and one access-related variable. The Commercial Access Density variable was identified as being significantly related to adjusted functional area crash rates even after non-access-related factors are considered. Thus, for study site intersections, commercial access density has a significant association with adjusted functional area crash rates. Because the sign of the Commercial Access Density regression coefficient is positive, an increase in commercial access density is associated with an increase in adjusted functional area crash rates.

Table 4.9 shows that the signs of each non-access-related variable regression coefficient are also positive, meaning that each of the variables has a positive association with adjusted functional area crash rates. As with the crash totals analysis in Section 4.3, this positive regression coefficient is intuitive for the Minor-Street Through Lanes variable but not necessarily for the left-turn variables. As discussed in Section 4.3, rather than representing the left-turn protection itself, the positive regression coefficients likely represent the road conditions that create the need for left-turn protection, such as heavy left-turn volumes, high left-turn crash rates, or insufficient left-turn storage.

4.5 Crash Severity

Stepwise variable selection was conducted on the non-access-related variables for the natural log of severity costs per year resulting from crashes occurring in the intersection functional area. The following non-access-related variables were significant at the 95th percentile level:

- AADT
- Minor-Street Through Lanes
- Minor-Street LT Protection
- Speed Limit

Manually inserting the above non-access-related variables into the final output, the second stepwise selection iteration was conducted including the access-related variables as potential model variables. In addition to the above variables, the following access-related variable was selected as a significant factor in describing functional area crash severity costs at the 95th percentile significance level:

- Corner Clearance Score

A multiple linear regression model was formed with the four non-access-related and one access-related variables. Table 4.10 presents the regression coefficients, standard errors, *t*-statistics, *p*-values, and the *R*-squared term for the multiple linear regression model.

Table 4.10 Multiple Linear Regression Model for Crash Severity Variable

Variable		Regression Coefficient	Standard Error	*t*-statistic	*p*-value
	(Constant)	2.94	0.606	4.86	< 0.01
Non-Access-Related	AADT	0.0436	0.0069	6.33	< 0.01
	Minor-Street Through Lanes	0.267	0.126	2.11	0.04
	Minor-Street LT Protection	0.531	0.162	3.28	< 0.01
	Speed Limit	0.0386	0.0150	2.57	0.01
Access-Related	Corner Clearance Score	0.233	0.0858	2.71	< 0.01
R-squared = 0.48					

Equation 4.4 shows the multiple regression model in mathematical form while graphical relationships between variables are shown in Appendix K. As discussed in Section 4.1, the regression equations presented in this study are not suitable for prediction purposes.

$$\ln(crash\,severity) = 2.94 + 0.0436X_1 + 0.267X_2 + 0.531X_3 + 0.0386X_4 + 0.233X_5 \tag{4.4}$$

where: X_1 = AADT (thousands of vpd),

X_2 = Minor-Street Through Lanes,

X_3 = Minor-Street LT Protection,

X_4 = Speed Limit (mph), and

X_5 = Corner Clearance Score.

As seen in Table 4.10, the multiple linear regression model for the natural log of the functional area crash severity costs includes several non-access-related variables and one access-related variable. The Corner Clearance Score variable was identified as being significantly related to functional area crash severity costs even after non-access-related factors are considered. Thus, for study site intersections, violation of UDOT corner clearance standards has a significant association with functional crash severity costs. Because the sign of the Corner Clearance Score regression coefficient is positive, an increase in the number of approaches in violation of UDOT corner clearance standards is associated with an increase in functional area crash severity costs.

Table 4.10 shows that the signs of each non-access-related variable regression coefficient are also positive, meaning that each of the variables has a positive association with functional area crash severity costs. This positive regression coefficient is intuitive for the AADT variable, the Minor-Street Through Lanes variable, and the Speed Limit variable, but not necessarily for the Minor-Street LT Protection variable. As discussed in Section 4.3, rather than representing the presence of left-turn protection itself, the positive regression coefficient likely represents the road conditions that create the need for left-turn protection, such as heavy left-turn volumes, high left-turn crash rates, or insufficient left-turn storage.

4.6 Crash Type

Because of the concentrated size of intersection functional areas, some crash types did not exhibit sufficient frequency to merit a meaningful statistical analysis. Of the six crash types identified in Section 3.3.4, only right-angle crashes and rear-end crashes featured an even distribution of crash frequency among study intersections. As can be seen from Table 4.6 in Section 4.2.2, all other crash types average approximately one crash per year or less. Stepwise variable selection and multiple linear regression results from these data would be unreliable. Therefore, only the right-angle crash and rear-end crash types were included in the statistical analysis. Furthermore, right-angle and rear-end crashes are the crash types of most interest as they are correlated with crash severity. Right-angle crashes tend to be one of the most severe crashes, while rear-end crashes tend to be one of the least severe.

4.6.1 Right Angle

Stepwise variable selection was conducted on the non-access-related variables for the natural log of the number of right-angle crashes per year occurring in the intersection functional area. The following non-access-related variables were significant at the 95^{th} percentile level:

- AADT
- Major-Street LT Protection
- Minor-Street LT Protection

Manually inserting the above non-access-related variables into the final output, the second stepwise selection iteration was conducted including the access-related variables as potential model variables. In addition to the above variables, the following access-related variable was selected as a significant factor in describing functional area right-angle crashes at the 95^{th} percentile significance level:

- Corner Clearance Score

A multiple linear regression model was formed with the three non-access-related and one access-related variables. Table 4.11 presents the regression coefficients, standard errors, *t*-statistics, *p*-values, and the *R*-squared term for the multiple linear regression model.

Table 4.11 Multiple Linear Regression Model for Right Angle Variable

Variable		Regression Coefficient	Standard Error	*t*-statistic	*p*-value
	(Constant)	0.434	0.185	2.34	0.02
Non-Access-Related	AADT	0.0170	0.0054	3.17	< 0.01
	Major-Street LT Protection	0.666	0.152	4.37	< 0.01
	Minor-Street LT Protection	0.428	0.151	2.83	< 0.01
Access-Related	Corner Clearance Score	0.210	0.0656	3.19	< 0.01
R-squared = 0.53					

Equation 4.5 shows the multiple regression model in mathematical form while graphical relationships between variables are shown in Appendix L. As discussed in Section 4.1, the regression equations presented in this study are not suitable for prediction purposes.

$$\ln(right\ angle) = 0.4340 + 0.0170X_1 + 0.666X_2 + 0.428X_3 + 0.210X_4 \quad (4.5)$$

where: X_1 = AADT,

X_2 = Major-Street LT Protection,

X_3 = Minor-Street LT Protection, and

X_4 = Corner Clearance Score.

As seen in Table 4.11, the multiple linear regression model for the natural log of the functional area right-angle crashes includes three non-access-related variables and one access-related variable. The Corner Clearance Score variable was identified as being significantly related to functional area right-angle crashes even after non-access-related factors are considered. Thus, for study site intersections, violation of UDOT corner clearance standards has a significant association with functional area right-angle crashes. Because the sign of the Corner Clearance Score regression coefficient is positive, an increase in the number of approaches in violation of UDOT corner clearance standards is associated with an increase in functional area right-angle crashes.

Table 4.11 shows that the signs of each non-access-related variable regression coefficient are also positive, meaning that each of the variables has a positive association with functional area right-angle crashes. This positive regression coefficient is intuitive for the AADT variable but not necessarily for the left-turn variables. As discussed in Section 4.3, rather than representing the presence of left-turn protection itself, the positive regression coefficients likely represents the road conditions that create the need for left-turn protection, such as heavy left-turn volumes, high left-turn crash rates, or insufficient left-turn storage.

4.6.2 Rear End

Because the natural log of zero is undefined, two study sites that featured no rear-end crashes could not undergo natural log transformation. Also, initial statistical analysis revealed that two additional study intersections skewed the distribution of the multiple linear regression model. These two intersections both feature only one rear-end crash for the three-year study period and also appear as outliers even after natural log transformation, as displayed in Figure 4.14 in Section 4.2.2. Consequently, all four of the above-mentioned intersections were removed from analysis and the scope of the rear-end crash analysis was limited to intersections with two or more rear-end crashes during the study period. For the remaining data, stepwise variable selection was conducted on the non-access-related variables for the natural log of the number of rear-end crashes per year occurring in the intersection functional area. From the remaining data, the following non-access-related variables were significant at the 95th percentile level:

- AADT
- Minor-Street Through Lanes
- Minor-Street LT Protection

Manually inserting the above non-access-related variables into the final output, the second stepwise selection iteration was conducted including the access-related variables as potential model. In addition to the above variables, the following access-related variables were selected as significant factors in describing functional area rear-end crashes at the 95th percentile significance level:

- Commercial Access Density
- Median Score

A multiple linear regression model was formed with the three non-access-related and two access-related variables. Table 4.12 presents the regression coefficients, standard errors, *t*-statistics, *p*-values, and the *R*-squared term for the multiple linear regression model.

Table 4.12 Multiple Linear Regression Model for Rear End Variable

Variable		**Regression Coefficient**	**Standard Error**	***t*-statistic**	***p*-value**
	(Constant)	-0.173	0.191	-0.910	0.37
Non-Access-Related	AADT	0.0414	0.0048	8.72	< 0.01
	Minor-Street Through Lanes	0.212	0.0867	2.44	0.02
	Minor-Street LT Protection	0.582	0.105	5.53	< 0.01
Access-Related	Commercial Access Density	0.0196	0.0070	2.79	< 0.01
	Median Score	0.165	0.0647	2.56	0.01
R-squared = 0.63					

Equation 4.6 shows the multiple regression model in mathematical form while graphical relationships between variables are shown in Appendix M. As discussed in Section 4.1, the regression equations presented in this study are not suitable for prediction purposes.

$$\ln(rear\ end) = -0.173 + 0.0414X_1 + 0.212X_2 + 0.582X_3 + 0.0196X_4 + 0.165X_5 \tag{4.6}$$

where: X_1 = AADT,

X_2 = Minor-Street Through Lanes,

X_3 = Minor-Street LT Protection,

X_4 = Commercial Access Density, and

X_5 = Median Score.

As seen in Table 4.12, the multiple linear regression model for the natural log of the functional area rear-end crashes includes three non-access-related variables and two access-related variables. The Commercial Access Density variable and the Median Score variable were identified as being significantly related to functional area rear-end crashes even after non-access-related factors are considered. Thus, for study site intersections, commercial access density and the presence of raised medians on major-street approaches have a significant association with functional area rear-end crashes. Because the sign of the both access-related regression coefficients are positive, increases in commercial access density as well as the presence of raised medians on major-street approaches are associated with an increase in functional area rear-end crashes.

As one of the least severe crash types, rear-end crashes have an association with severity levels. Previous research has found that, although the installation of raised medians along a roadway may be accompanied by increased rear-end crashes, the overall crash severity decreases (Schultz and Lewis 2006). Because raised medians prohibit egressing and ingressing left-turns from accesses, fewer opportunities for right-angle crashes, which are generally some of the most severe crashes, exist. Consequently, a possible increase in rear-end crashes accompanying a raised median may be offset by the reduction in right-angle crashes.

Table 4.12 shows that the signs of each non-access-related variable regression coefficient are also positive, meaning that each of the variables has a positive association with functional area rear-end crashes. This positive regression coefficient is intuitive for the AADT variable and the Minor-Street Through Lanes variable, but not necessarily for the Minor-Street LT Protection variable. As discussed in Section 4.3, rather than representing the left-turn protection itself, the positive regression coefficient likely represents the road conditions that create the need for left-turn protection, such as heavy left-turn volumes, high left-turn crash rates, or insufficient left-turn storage.

4.7 Reference Corridor Analysis

Analysis of 15 reference site intersections was conducted in order to compare crash patterns at locations that prohibit all unsignalized access with the crash patterns evident in the rest of the study data. As previously mentioned in Section 3.1, reference site intersections were selected from two restricted access corridors as identified in cooperation with UDOT personnel. Both corridors feature the UDOT access classification of "System Priority Urban," meaning they do not allow any unsignalized access (UDOT 2006a). Appendix N contains a full listing of reference site raw data.

Because of the unique nature of the reference data, a direct comparison between reference site crash patterns and study site crash patterns was not appropriate. In addition to their restrictions on unsignalized access, reference sites were found to differ from the study sites in several other ways. First, all 15 reference sites had posted speed limits between 50 mph to 60 mph, whereas study site speed limits only exceeded 45 mph at two locations. Second, every reference site featured raised medians on both major-street approaches. In comparison, three-quarters of study sites had no raised median on either approach. Third, some level of left-turn protection (protected-permitted or protected) was evident on nearly all reference site major-street and minor-street approaches. Finally, reference site major streets tended to be larger than the study site major streets. At 13 out of 15 reference sites, the major street averaged three through lanes, whereas only 26 percent of study sites averaged three or more major-street through lanes. In essence, the reference sites represented a population of intersections dissimilar to the study site intersections in more ways than just the restriction of unsignalized accesses. Thus, differences in crash patterns arising from a direct comparison may not be attributable to the discrepancies in accesses alone. The disparities in speed limit, median, left-turn protection, or roadway size could also be contributing to the difference in crash patterns.

Since a direct comparison between reference site and study site crash patterns was not an appropriate analytical technique, a methodology that considered all differing intersection characteristics was needed. Therefore, stepwise variable selection and multiple linear regression analysis were again employed to investigate the factors that significantly describe the dependent crash variables for study and reference intersections.

For the first step in the analysis process, the reference site data were combined with the study data to form a single dataset. The variables within the dataset were then adjusted slightly to accommodate the new distribution of data. For example, because of the high speed limits of the reference sites, consolidating the 45 mph and 50 mph speed distribution categories was no longer necessary, as was done for the study site analysis. For the reference site analysis, each speed limit category remained unaltered. Also, a new indicator variable was created to distinguish between reference sites and study sites. For this indicator variable, a reference site received a value of 1, while a study site received a value of 0.

The second step in the analysis process was to conduct stepwise variable selection on the non-access-related variables to determine the non-access-related factors that describe the dependent crash variables for both study and reference sites. The selected non-access-related variables were then used to form a multiple linear regression model. In this way, the dissimilar characteristics between reference sites and study sites, such as speed limit, median type, type of left-turn protection, and roadway size are accounted for within the results of the stepwise selection and the multiple linear regression model. Finally, the reference variable was inserted into the multiple linear regression model to determine whether it added any significant descriptive power. If the reference variable was significant at the 95th percentile level, then reference sites did feature significantly different crash patterns than the study sites even after all non-access variables were accounted for. Unlike the study site analysis, stepwise selection was not run on the access-related variables in the reference analysis because reference sites did not have any accesses. Hence, the access-related variables would not add additional meaning to the model.

As with the study sites, statistical analyses were conducted on natural-log-transformed dependent variables. Initial investigation of the multiple linear regression models revealed one outlier and two outliers were influential to the results of the right-angle and rear-end analyses, respectively. The outliers were removed, and the analysis was rerun. In consequence of the removed outliers, the scope of the results was reduced to the range of values expressed in the remaining data points. Table 4.13 summarizes the variables selected from the stepwise selection process for each of the dependent crash variables. Within Table 4.13, the regression coefficients and *p*-values from the multiple linear regression model for each independent variable, including the inserted reference variable, are also shown. Reference variable *p*-values significant at the 95th percentile level are presented in bold-face type for emphasis.

As can be seen from Table 4.13, the reference variable was significant at the 95th percentile level for the following dependent crash variables:

- Crashes
- Crash Rate
- Right Angle
- Rear End

At the 85th percentile level, the Crash Severity dependent variable was significant as well. The reference variable regression coefficient was negative in each regression model, suggesting that the reference intersections are associated with significantly lower crash variables than non-reference intersections. Thus, the restriction of all unsignalized accesses is correlated with fewer total crashes, lower crash rates, fewer right-angle crashes, and fewer rear-end crashes within the data acquired for this study.

Table 4.13 Reference Analysis Multiple Linear Regression Models

	Crash Totals		Crash Rate		Crash Severity		Right Angle		Rear End	
Variable	**Regression Coefficient**	***p*-value**	**Regression Coefficient**	***p*-value**	**Regression Coefficient**	***p*-value**	**Regression Coefficient**	***p*-value**	**Regression Coefficient**	***p*-value**
(Constant)	1.23	< 0.01	-0.241	< 0.01	4.97	< 0.01	1.14	< 0.01	0.0396	0.80
AADT	0.0272	< 0.01			0.0428	< 0.01			0.0400	< 0.01
Major-Street LT Protection	0.381	< 0.01	0.289	< 0.01			1.05	< 0.01		
Minor-Street LT Protection	0.522	< 0.01	0.536	< 0.01	0.847	< 0.01			0.652	< 0.01
Minor-Street Through Lanes	0.184	< 0.01	0.180	< 0.01					0.244	< 0.01
FA Overlap							0.254	0.03		
Reference	-0.318	**0.01**	-0.307	**0.01**	-0.351	0.12	-0.936	**< 0.01**	-0.510	**< 0.01**

4.8 Summary of Intersection Analysis

Because intersection crashes can be influenced by a number of factors, a statistical methodology was used to separate the effects of access-related factors from non-access-related factors. The impact of the access-related factors was examined against the total number of functional area crashes, an intersection functional area crash rate, the functional area crash severity costs, and the total right-angle crashes and rear-end crashes within intersection functional areas.

Table 4.14 summarizes the access-related factors that were correlated with dependent crash variables at a 95 percent significance level even after accounting for non-access-related factors. Within Table 4.14, the sign of the resulting regression coefficient is given for each statistically significant access-related variable. A "+" symbol indicates a positive regression coefficient while a blank cell indicates that an access-related variable was not selected for the multiple linear regression model. As can be seen in Table 4.14, every dependent variable was significantly associated with at least one access-related variable. The Crash Total and Crash Rate variables were positively correlated with the Commercial Access Density variable, the Crash Severity and Right Angle variables were positively correlated with the Corner Clearance Score variable, and the Rear End variable was positively correlated with the Commercial Access Density variable and the Median Score variable.

Table 4.14 Summary of Significant Access-Related Variables

	Dependent Crash Variable				
Access-Related Variable	**Crash Totals**	**Crash Rate**	**Crash Severity**	**Right Angle**	**Rear End**
Commercial Access Density	+	+			+
Corner Clearance Score			+	+	
Median Score					+

"+" denotes positive correlation.

An important consideration is that the absence of an access-related variable in the final regression model of a crash variable does not necessarily imply that that particular access-related factor has no significant relationship with the crash variable. The stepwise variable selection process iteratively selects only those independent variables most correlated with the dependent variable at the same time as examining the overall descriptive power a potential variable adds to the model. Once one variable is selected, a second significant variable may not be selected simply because it is too correlated with the first variable to add additional meaning to the model.

For example, in this statistical analysis, the Commercial Access Density and Corner Clearance Score had sufficient correlation that the inclusion of one variable led to the exclusion of the other. As can be seen from Table 4.14, when the Commercial Access Density variable was selected first, the Corner Clearance Score variable was never selected. Conversely, when the Corner Clearance Score variable was selected first, the Commercial Access Density variable was never selected. These results do not mean that for models in which Commercial Access Density was deemed significant that Corner Clearance Score was not significant. Instead, in these instances, Commercial Access Density was slightly more significant than Corner Clearance Score, and the inclusion of Commercial Access Density in the model omitted the need for Corner Clearance Score because of the relationship between the two variables. This relationship was verified by re-analyzing the data while removing the Commercial Access Density or Corner Clearance Score variables for models in which they were selected first. With the exception of the Rear End crashes variable, whenever the Commercial Access Density was removed, Corner Clearance Score was chosen in its place and vice versa.

The association between Corner Clearance Score and intersection functional area crash patterns supports the importance of adherence to UDOT corner clearance criteria. Study intersections that have approach-side accesses in violation of UDOT corner clearance standards were found to have more severe crashes and more right-angle crashes. Also, when considering the correlation between the Commercial Access Density and Corner Clearance variables, as discussed above, UDOT corner clearance violation is likewise associated with increased crash totals and crash rates.

The Median Score variable was shown to have a positive association with functional area rear-end crashes. This finding is consistent with previous research that indicates that the presence of raised medians is associated with slightly increased rear-end crashes on roadway corridors (Schultz and Braley 2007; Schultz and Lewis 2006). However, as noted in the previous research, rear-end crashes are generally less severe than other crash types. Consequently, raised medians may contribute to more rear-end crashes in intersection functional areas, but this should be considered with regards to the complete effect of a raised median. Since raised medians prohibit ingressing and egressing left-turns at accesses, intersection approaches with raised medians are expected to exhibit increased rear-end crashes at the expense of fewer right-angle crashes.

The reference site analysis results coincided with the study site analysis in that fewer accesses in intersection functional areas were associated with lower crash totals. From the results of the reference site analysis, sites that prohibited all unsignalized access were found to have fewer crashes than the sites that did permit unsignalized access. Since the reference intersections prohibited all unsignalized access, they represented sites that had no accesses within the functional area.

This chapter has presented the statistical analyses conducted on the data collected from the study and reference sites. Relationships between crash patterns and access-related factors were examined. The next chapter summarizes the conclusions and recommendations of this research.

5 Conclusions and Recommendations

The preceding chapters presented an analysis of the relationship between crashes and access location at major-arterial crossroads. Chapter 2 provided a literature review of access management and intersection principles pertinent to the study. The application of access management techniques at signalized intersections was discussed, as well as the implementation of these techniques in the state of Utah. Next, the definition of an intersection functional area was examined. Finally, UDOT intersection crash analysis methods were reviewed. Chapter 3 outlined the data collection process undertaken to acquire information for the statistical analyses. Data were obtained from 144 study intersections and 15 reference intersections across the state of Utah. The data collection process focused on the intersection, access, and crash characteristics of each site. Chapter 4 presented the results of the statistical analyses conducted to evaluate access location and intersection crash relationships. The data from Chapter 3 were divided into independent and dependent variables. Then, using statistical tools, the impacts of access location and spacing on intersection crash patterns were isolated from other intersection characteristics.

This chapter summarizes the findings of the report. Section 5.1 presents the analysis conclusions, Section 5.2 outlines the recommendations, and Section 5.3 highlights areas of the research results that would benefit from further study.

5.1 Conclusions

The purpose of this research was to analyze the relationship between access location and safety at major-arterial crossroads. The results of the statistical analysis showed that access density, access location, and access type had a significant impact on safety within intersection functional areas. The existence of accesses within the functional area of study sites were correlated with increased crashes and crash severity costs. In particular, an increase in commercial access density was associated with increases in crash totals, crash rates, and rear-end crashes in intersection functional areas. The analysis also showed that when UDOT corner clearance standards were observed, study site intersections exhibited fewer right-angle crashes and lower crash severity costs. Finally, the presence of raised medians on major-street approaches was associated with increased rear-end crashes. Previous research has shown that increases in rear-end crashes, when accompanying a raised median, are correlated with crash severity reductions due to the decreased opportunities for right-angle crashes (Schultz and Lewis 2006).

Additional research conclusions were determined from the reference site analysis. In the reference site analysis, intersections that allowed unsignalized accesses on their major-street approaches were compared against a group of reference intersections that do not permit any unsignalized access on the major-street approaches. The reference group intersections were found to have lower crash totals, crash rates, right-angle crash totals, and rear-end crash totals.

5.2 Recommendations

Intersection functional areas represent a sensitive component of the traffic system due to the numerous conflicting vehicle movements. When accesses are located within the functional area, additional conflicts are introduced into the traffic stream complicating vehicle maneuvers. UDOT personnel should continue to preserve the functional areas of major intersections by adhering to the access spacing and setback standards developed within Administrative Rule R930-6, *Accommodation of Utilities and the Control and Protection of State Highway Rights of Way* (UDOT 2006a). This research has found that major intersections within the state of Utah exhibited fewer crashes and less severe crashes when functional areas featured reduced commercial access densities and when UDOT corner clearance standards were maintained.

5.3 Future Research

Further research is needed to investigate the true relationship between left-turn protection and functional area crashes. In this study, the existence of left-turn protection at major signalized intersections was found to be associated with increased functional area crashes. Although left-turn protection appears to be linked to decreased safety, the true relationship of left-turn protection and functional area crashes remains unclear for several reasons. First, because the statistical analyses were conducted as an observational study, not a randomized experiment, cause-and-effect relationships may not be assumed. Second, the intent of the left-turn protection variable was to act as an indicator for other roadway characteristics (e.g., turning volumes, minor-street AADT). Any supposed relationship between left-turn protection and crashes may actually represent a relationship with the characteristics for which left-turn protection is an indicator. Finally, because UDOT utilizes left-turn volume and crash rate warrants for the installation of left-turn protection, all intersections with left-turn protection automatically feature characteristics that fulfill those warrants. In other words, such intersections meet certain volumes or crash-rate levels before the left-turn protection is installed. In summary, from this research, the question whether left-turn protection at intersections is the cause of increased crashes or whether left-turn protection is installed at the intersections that already have high crash levels remains uncertain. In order to more accurately ascertain the impact of left-turn protection on functional area crashes, a before-and-after study of left-turn protection is recommended. Such a study may be able identify fluctuations in crash rates at intersections where all other factors are held constant.

The relationship between rear-end crashes and the presence of raised medians should also be investigated further. In this research, as well as previous research (Schultz and Braley 2007; Schultz and Lewis 2006), slight increases in rear-end crashes have been observed at Utah corridors and intersections featuring raised medians. However, in many cases, overall crash severity has been found to be reduced. If an increase in rear-end crashes at raised median sites is accompanied by a complimentary decrease in a more severe crash type, such as right-angle crashes, then the overall increase in crashes increase may actually represent improved safety. Additional research is needed to evaluate the true impact of raised medians on crash types and crash severity.

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Appendix A. Study Site Locations

Table A.1 Study Site Locations

Study ID	Route Num	Major Street	Minor Street	Functional Class	Access Class
2	0265	University Pkwy	Sandhill Road	Arterial	Regional Priority Urban
3	0265	University Pkwy	400 West	Arterial	Regional Priority Urban
4	0265	University Pkwy	Main St	Arterial	Regional Priority Urban
5	0265	University Pkwy	200 East	Arterial	Regional Priority Urban
6	0265	University Pkwy	State St	Arterial	Regional Priority Urban
7	0265	University Pkwy	800 East	Arterial	Regional Priority Urban
9	0265	University Pkwy	Freedom Blvd	Arterial	Regional Priority Urban
10	0265	University Pkwy	University Ave	Arterial	Regional Priority Urban
11	0189	University Ave	East Bay Blvd	Arterial	Regional Priority Urban
12	0189	University Ave	1200 South	Arterial	Regional Priority Urban
13	0189	University Ave	900 South	Arterial	Regional Priority Urban
14	0189	University Ave	300 South	Arterial	Regional Urban
15	0189	University Ave	100 South	Arterial	Regional Urban
16	0189	University Ave	Center Street	Arterial	Regional Urban
17	0189	University Ave	100 North	Arterial	Regional Urban
18	0189	University Ave	200 North	Arterial	Regional Urban
19	0189	University Ave	500 North	Arterial	Regional Urban
20	0189	University Ave	700 North	Arterial	Regional Urban
21	0189	University Ave	800 North	Arterial	Regional Urban
24	0189	University Ave	Bulldog Blvd	Arterial	Regional Urban
32	0052	800 North	1200 West	Arterial	Regional Priority Urban
33	0052	800 North	900 West	Arterial	Regional Priority Urban
34	0052	800 North	800 West	Arterial	Regional Priority Urban
35	0052	800 North	State Street	Arterial	Regional Priority Urban
36	0052	800 North	Main St	Arterial	Regional Priority Urban
37	0052	800 North	400 East	Arterial	Regional Priority Urban
38	0052	800 North	800 East	Arterial	Regional Priority Urban
39	0052	800 North	Palisade Drive	Arterial	Regional Priority Urban
40	0203	Harrison Blvd	5700 South	Arterial	Regional Rural
41	0203	Harrison Blvd	5600 South	Arterial	Regional Rural
42	0203	Harrison Blvd	Edgewood Dr	Arterial	Regional Rural
43	0203	Harrison Blvd	4600 South	Arterial	Regional Rural
44	0203	Harrison Blvd	Country Hills Dr	Arterial	Community Rural
45	0203	Harrison Blvd	3850 South	Arterial	Community Rural

Table A.1 (cont.)

Study ID	Route Num	Major Street	Minor Street	Functional Class	Access Class
46	0203	Harrison Blvd	36th Street	Arterial	Community Rural
47	0203	Harrison Blvd	32nd Street	Arterial	Community Rural
49	0203	Harrison Blvd	28th Street	Arterial	Community Rural
50	0203	Harrison Blvd	26th Street	Arterial	Community Rural
51	0203	Harrison Blvd	24th Street	Arterial	Community Rural
52	0203	Harrison Blvd	22nd Street	Arterial	Community Rural
53	0203	Harrison Blvd	20th Street	Arterial	Community Rural
54	0204	Wall Avenue	36th Street	Minor Arterial	Community Rural
57	0204	Wall Avenue	29th Street	Minor Arterial	Community Rural
62	0204	Wall Avenue	12th Street	Minor Arterial	Community Rural
65	0089	Washington Blvd	40th Street	Arterial	Regional Priority Urban
66	0089	Washington Blvd	36th Street	Arterial	Regional Priority Urban
68	0089	Washington Blvd	34th Street	Arterial	Regional Priority Urban
69	0089	Washington Blvd	32nd Street	Arterial	Regional Priority Urban
72	0089	Washington Blvd	29th Street	Arterial	Regional Urban
73	0089	Washington Blvd	28th Street	Arterial	Regional Urban
74	0089	Washington Blvd	27th Street	Arterial	Regional Urban
75	0089	Washington Blvd	26th Street	Arterial	Regional Urban
76	0089	Washington Blvd	25th Street	Arterial	Regional Urban
77	0089	Washington Blvd	24th Street	Arterial	Regional Urban
79	0089	Washington Blvd	22nd Street	Arterial	Regional Urban
80	0089	Washington Blvd	21st Street	Arterial	Regional Urban
81	0089	Washington Blvd	20th Street	Arterial	Regional Urban
82	0089	Washington Blvd	17th Street	Arterial	Regional Urban
83	0089	Washington Blvd	12th Street	Arterial	Regional Priority Urban
84	0089	Washington Blvd	7th Street	Arterial	Regional Priority Urban
85	0013	Main Street	990 South	Arterial	Regional Urban
86	0013	Main Street	700 South	Arterial	Regional Urban
87	0013	Main Street	200 South	Arterial	Regional Urban
88	0013	Main Street	100 South	Arterial	Regional Urban
90	0013	Main Street	100 North	Arterial	Regional Urban
91	0089	Main Street	2600 South	Minor Arterial	Regional Priority Urban
92	0089	500 West	1500 South	Minor Arterial	Regional Priority Urban
93	0089	500 West	500 South	Minor Arterial	Regional Priority Urban
94	0089	500 West	400 North	Minor Arterial	Regional Priority Urban
95	0040	Main Street	500 North	Arterial	Community Rural
96	0040	Main Street	100 South	Arterial	Community Rural
97	0040	Main Street	600 South	Arterial	Community Rural
98	0040	Main Street	1200 South	Arterial	Community Rural
99	0036	Main Street	Vine Street	Arterial	Regional Urban
100	0036	Main Street	200 North	Arterial	Regional Urban
101	0036	Main Street	400 North	Arterial	Regional Urban
102	0036	Main Street	600 North	Arterial	Regional Urban
103	0036	Main Street	1280 North	Arterial	Regional Priority Urban
106	0130	Main Street	200 South	Arterial	Regional Urban
107	0130	Main Street	Center Street	Arterial	Regional Urban
108	0130	Main Street	200 North	Arterial	Regional Urban
112	0056	200 North	Airport Road	Arterial	Regional Priority Urban
113	0056	200 North	800 West	Arterial	Regional Urban

Table A.1 (cont.)

Study ID	Route Num	Major Street	Minor Street	Functional Class	Access Class
114	0056	200 North	300 West	Arterial	Regional Urban
115	0055	100 North	Carbon Avenue	Arterial	Regional Urban
116	0055	Main Street	700 East	Arterial	Regional Urban
117	0034	St George Blvd	300 West	Arterial	Regional Urban
118	0034	St George Blvd	Main St	Arterial	Regional Urban
119	0034	St George Blvd	200 East	Arterial	Regional Urban
120	0034	St George Blvd	400 East	Arterial	Regional Urban
121	0034	St George Blvd	700 East	Arterial	Regional Urban
122	0034	St George Blvd	1000 East	Arterial	Regional Urban
123	0018	Bluff Street	Black Ridge Dr	Arterial	Regional Priority Urban
126	0018	Bluff Street	St George Blvd	Arterial	Regional Urban
130	0009	State Street	1150 West	Arterial	Community Rural
131	0009	State Street	700 West	Arterial	Community Rural
132	0009	State Street	300 West	Arterial	Community Rural
133	0009	State Street	Main St	Arterial	Community Rural
134	0040	Main Street	1000 West	Arterial	Regional Urban
135	0040	Main Street	500 West	Arterial	Regional Urban
136	0040	Main Street	100 West	Arterial	Regional Urban
137	0040	Main Street	Vernal Ave	Arterial	Regional Urban
138	0040	Main Street	500 East	Arterial	Regional Priority Urban
139	0171	3500 South	5600 West	Arterial	Regional Priority Urban
140	0171	3500 South	4800 West	Arterial	Regional Priority Urban
141	0171	3500 South	4400 West	Arterial	Regional Priority Urban
142	0171	3500 South	4000 West	Arterial	Regional Priority Urban
143	0171	3500 South	Bangerter Hwy	Arterial	Regional Priority Urban
144	0171	3500 South	3600 West	Arterial	Regional Priority Urban
145	0171	3500 South	3200 West	Arterial	Regional Priority Urban
146	0171	3500 South	Constitution Blvd	Arterial	Regional Priority Urban
147	0171	3500 South	Decker Lake Dr	Arterial	Regional Priority Urban
148	0171	3500 South	1940 West	Arterial	Regional Priority Urban
149	0171	3500 South	Redwood Rd	Arterial	Regional Priority Urban
150	0171	3300 South	900 West	Arterial	Regional Priority Urban
151	0171	3300 South	West Temple	Arterial	Regional Urban
152	0171	3300 South	Main St	Arterial	Regional Urban
153	0171	3300 South	State St	Arterial	Regional Urban
154	0171	3300 South	200 East	Arterial	Regional Urban
155	0171	3300 South	300 East	Arterial	Regional Urban
157	0171	3300 South	700 East	Arterial	Regional Urban
158	0171	3300 South	900 East	Arterial	Regional Urban
159	0171	3300 South	1100 East	Arterial	Regional Urban
160	0171	3300 South	1300 East	Arterial	Regional Urban
161	0068	Redwood Rd	9000 South	Arterial	Regional Priority Urban
162	0068	Redwood Rd	7800 South	Arterial	Regional Priority Urban
163	0068	Redwood Rd	7000 South	Arterial	Regional Priority Urban
164	0068	Redwood Rd	4700 South	Minor Arterial	Regional Priority Urban
165	0068	Redwood Rd	Conifer Way	Minor Arterial	Regional Priority Urban
166	0068	Redwood Rd	4450 South	Minor Arterial	Regional Priority Urban
167	0068	Redwood Rd	4200 South	Minor Arterial	Regional Priority Urban
168	0068	Redwood Rd	4100 South	Minor Arterial	Regional Priority Urban

Table A.1 (cont.)

Study ID	Route Num	Major Street	Minor Street	Functional Class	Access Class
170	0089	State Street	10600 South	Minor Arterial	Regional Priority Urban
171	0089	State Street	10200 South	Minor Arterial	Regional Priority Urban
172	0089	State Street	10000 South	Minor Arterial	Regional Priority Urban
173	0089	State Street	9400 South	Minor Arterial	Regional Priority Urban
174	0089	State Street	9000 South	Minor Arterial	Regional Priority Urban
175	0089	State Street	7200 South	Minor Arterial	Regional Priority Urban
176	0089	State Street	6100 South	Minor Arterial	Regional Priority Urban
177	0089	State Street	5900 South	Minor Arterial	Regional Priority Urban
178	0089	State Street	5300 South	Minor Arterial	Regional Priority Urban
179	0089	State Street	Vine Street	Minor Arterial	Regional Priority Urban
180	0089	State Street	4800 South	Minor Arterial	Regional Priority Urban
181	0089	State Street	4500 South	Minor Arterial	Regional Priority Urban

Appendix B. Study Site Characteristics

Table B.1 Study Site Characteristics

Study ID	UDOT Region	Speed Limit	Major-Street Left Turn	Minor-Street Left Turn	Median	Analysis Years
2	3	45	Protected	Protected	Barrier/TWLTL	03-05
3	3	45	Protected-permitted	Protected-permitted	TWLTL/Barrier	03-05
4	3	45	Protected-permitted	Protected-permitted	Barrier	03-05
5	3	45	Protected-permitted	Protected-permitted	Barrier	03-05
6	3	45	Protected	Protected	Barrier	02-04
7	3	45	Protected	Protected	TWLTL/Barrier	03-05
9	3	40	Protected-permitted	Protected	Barrier/TWLTL	03-05
10	3	40/30	Protected-permitted	Protected-permitted	TWLTL	03-05
11	3	45	Protected	Protected	TWLTL	03-05
12	3	40	Protected-permitted	Permitted	TWLTL	03-05
13	3	40	Protected-permitted	Permitted	TWLTL	03-05
14	3	35	Protected-permitted	Protected-permitted	TWLTL	02-04
15	3	35	Permitted	Permitted	TWLTL	03-05
16	3	35	Permitted	Permitted	TWLTL	03-05
17	3	35	Permitted	Permitted	TWLTL	03-05
18	3	35	Permitted	Permitted	TWLTL	03-05
19	3	35	Permitted	Permitted	TWLTL	03-05
20	3	35	Permitted	Permitted	TWLTL	03-05
21	3	35	Permitted	Permitted	TWLTL	03-05
24	3	35	Protected-permitted	Protected-permitted	TWLTL	03-05
32	3	45	Protected-permitted	Protected-permitted	TWLTL	03-05
33	3	45	Permitted	Permitted	TWLTL	03-05
34	3	45	Permitted	Permitted	TWLTL	03-05
35	3	40	Protected	Protected	Barrier	02-04
36	3	40	Permitted	Permitted	TWLTL	03-05
37	3	40	Permitted	Permitted	TWLTL	03-05
38	3	45	Protected-permitted	Protected	TWLTL	03-05
39	3	45	Permitted	Permitted	TWLTL	03-05
40	1	50	Permitted	Permitted	TWLTL	02-04
41	1	50	Protected-permitted	Protected-permitted	TWLTL	02-04
42	1	40	Permitted	Permitted	TWLTL	02-04
43	1	40	Protected-permitted	Permitted	TWLTL	02-04
44	1	40	Protected-permitted	Protected-permitted	TWLTL	02-04
45	1	40	Permitted	Permitted	TWLTL	02-04

Table B.1 (cont.)

Study ID	UDOT Region	Speed Limit	Major-Street Left Turn	Minor-Street Left Turn	Median	Analysis Years
46	1	40	Protected-permitted	Protected-permitted	TWLTL	02-04
47	1	40	Permitted	Permitted	TWLTL	02-04
49	1	40	Permitted	Permitted	TWLTL	02-04
50	1	40	Permitted	Permitted	TWLTL	02-04
51	1	40	Permitted	Permitted	TWLTL	02-04
52	1	40	Permitted	Permitted	TWLTL	02-04
53	1	40	Permitted	Permitted	TWLTL	02-04
54	1	40	Protected-permitted	Protected-permitted	TWLTL	02-04
57	1	40	Protected-permitted	Permitted	TWLTL	02-04
62	1	40	Protected	Protected	TWLTL	02-04
65	1	40	Protected-permitted	Protected-permitted	TWLTL	02-04
66	1	40	Protected-permitted	Permitted	TWLTL	02-04
68	1	40	Permitted	Permitted	TWLTL	02-04
69	1	40	Permitted	Permitted	TWLTL	02-04
72	1	35	Permitted	Permitted	TWLTL	02-04
73	1	35	Permitted	Permitted	TWLTL	02-04
74	1	35	Permitted	Permitted	TWLTL	02-04
75	1	35	Permitted	Permitted	TWLTL	02-04
76	1	35	Permitted	Permitted	TWLTL	02-04
77	1	35	Permitted	Permitted	TWLTL	02-04
79	1	35	Permitted	Permitted	TWLTL	02-04
80	1	35	Permitted	Permitted	TWLTL	02-04
81	1	35	Permitted	Permitted	TWLTL	02-04
82	1	35	Permitted	Permitted	TWLTL	02-04
83	1	40	Protected-permitted	Protected-permitted	TWLTL	02-04
84	1	40	Permitted	Permitted	TWLTL	02-04
85	1	35	Permitted	Permitted	TWLTL	03-05
86	1	35	Permitted	Permitted	TWLTL	03-05
87	1	30	Permitted	Permitted	TWLTL	03-05
88	1	30	Permitted	Permitted	TWLTL	03-05
90	1	30	Permitted	Permitted	TWLTL	03-05
91	1	45	Protected-permitted	Protected-permitted	TWLTL	02-04
92	1	40	Permitted	Permitted	TWLTL	02-04
93	1	40	Protected-permitted	Protected-permitted	Barrier/TWLTL	02-04
94	1	40	Protected-permitted	Protected-permitted	TWLTL	02-04
95	3	35	Permitted	Permitted	TWLTL	03-05
96	3	35	Permitted	Permitted	TWLTL	03-05
97	3	35	Permitted	Permitted	TWLTL	03-05
98	3	40	Permitted	Protected	TWLTL	03-05
99	2	35	Protected-permitted	Protected-permitted	None	03-05
100	2	35	Protected-permitted	Protected-permitted	None/TWLTL	03-05
101	2	35	Protected-permitted	Permitted	TWLTL/Barrier	03-05
102	2	35	Permitted	Permitted	Barrier	03-05
103	2	40	Protected-permitted	Protected-permitted	TWLTL	03-05
106	4	30	Permitted	Permitted	TWLTL	02-04
107	4	30	Protected-permitted	Permitted	TWLTL	02-04
108	4	30	Protected-permitted	Protected-permitted	TWLTL	02-04
112	4	45	Permitted	Permitted	TWLTL	03-05
113	4	35	Permitted	Permitted	TWLTL	03-05

Table B.1 (cont.)

Study ID	UDOT Region	Speed Limit	Major-Street Left Turn	Minor-Street Left Turn	Median	Analysis Years
114	4	35	Permitted	Permitted	TWLTL	03-05
115	4	30	Permitted	Permitted	TWLTL	03-05
116	4	30	Permitted	Permitted	TWLTL	03-05
117	4	30	Permitted	Permitted	TWLTL	02-04
118	4	30	Permitted	Permitted	TWLTL	02-04
119	4	30	Permitted	Permitted	TWLTL	02-04
120	4	30	Permitted	Permitted	TWLTL	02-04
121	4	30	Permitted	Permitted	TWLTL	02-04
122	4	30	Protected-permitted	Protected	TWLTL	02-04
123	4	45	Protected	Protected	TWLTL	02-04
126	4	35	Protected-permitted	Permitted	TWLTL	02-04
130	4	45	Permitted	Permitted	TWLTL	03-05
131	4	40	Permitted	Permitted	TWLTL	03-05
132	4	30	Permitted	Permitted	TWLTL	03-05
133	4	30	Permitted	Permitted	TWLTL	03-05
134	3	35	Permitted	Permitted	TWLTL	02-04
135	3	30	Permitted	Permitted	TWLTL	02-04
136	3	30	Permitted	Permitted	TWLTL	02-04
137	3	30	Protected-permitted	Permitted	TWLTL	02-04
138	3	35	Permitted	Permitted	TWLTL	02-04
139	2	45	Protected-permitted	Protected-permitted	TWLTL	03-05
140	2	45	Protected-permitted	Permitted	TWLTL	03-05
141	2	40	Permitted	Permitted	TWLTL	03-05
142	2	40	Protected-permitted	Protected-permitted	TWLTL	03-05
143	2	40	Protected	Protected	TWLTL	03-05
144	2	40	Protected-permitted	Protected-permitted	TWLTL	03-05
145	2	40	Protected-permitted	Protected-permitted	TWLTL	03-05
146	2	40	Protected	Protected	Barrier	03-05
147	2	40	Protected	Protected-permitted	Barrier/TWLTL	03-05
148	2	40	Protected-permitted	Permitted	TWLTL	03-05
149	2	40	Protected	Protected	Barrier	03-05
150	2	45	Protected-permitted	Protected	TWLTL/Barrier	03-05
151	2	35	Protected-permitted	Permitted	TWLTL	03-05
152	2	35	Protected-permitted	Protected-permitted	TWLTL/Barrier	03-05
153	2	35	Protected-permitted	Protected	Barrier/TWLTL	02-04
154	2	35	Permitted	Permitted	TWLTL	03-05
155	2	35	Permitted	Permitted	TWLTL	03-05
157	2	35	Protected-permitted	Protected	Barrier/TWLTL	03-05
158	2	35	Protected-permitted	Protected-permitted	TWLTL	03-05
159	2	35	Permitted	Permitted	TWLTL	03-05
160	2	35	Protected-permitted	Protected-permitted	Barrier/TWLTL	03-05
161	2	45	Protected	Protected	Barrier	03-05
162	2	45	Protected	Protected	Barrier	03-05
163	2	45	Protected	Protected	Barrier	03-05
164	2	40	Protected-permitted	Protected-permitted	Barrier	03-05
165	2	40	Protected-permitted	Permitted	TWLTL	03-05
166	2	40	Protected-permitted	Permitted	TWLTL	03-05
167	2	40	Permitted	Permitted	TWLTL	03-05
168	2	40	Protected-permitted	Protected-permitted	Barrier	03-05

Table B.1 (cont.)

Study ID	UDOT Region	Speed Limit	Major-Street Left Turn	Minor-Street Left Turn	Median	Analysis Years
170	2	40	Protected	Protected	TWLTL/Barrier	02-04
171	2	40	Protected-permitted	Permitted	Barrier	02-04
172	2	40	Protected-permitted	Protected-permitted	Barrier/TWLTL	02-04
173	2	40	Protected	Protected	Barrier	02-04
174	2	40	Protected	Protected-permitted	Barrier	02-04
175	2	40	Protected-permitted	Protected	Barrier	02-04
176	2	40	Protected-permitted	Permitted	Barrier	02-04
177	2	40	Protected-permitted	Protected-permitted	Barrier	02-04
178	2	40	Protected	Protected-permitted	Barrier	02-04
179	2	40	Protected	Protected	Barrier	02-04
180	2	40	Protected	Permitted	Barrier	02-04
181	2	40	Protected	Protected	Barrier	02-04

Appendix C. Study Site Road Configurations

Table C.1 Study Site Road Configurations

Study ID	Major-Street AADT	Minor-Street AADT	Average Major-Street Through Lanes	Average Minor-Street Through Lanes	Freeway Adjacent
2	38,383	n/a	3.0	2.0	Y
3	38,383	n/a	3.0	1.0	N
4	38,383	4,092	3.0	1.0	N
5	38,383	n/a	3.0	1.0	N
6	42,148	57,164	3.0	3.0	N
7	41,987	n/a	2.0	1.0	N
9	37,852	15,445	2.0	2.0	N
10	28,214	36,690	2.0	2.0	N
11	28,448	5,127	3.0	2.0	Y
12	28,448	n/a	3.0	1.0	N
13	28,448	n/a	2.0	1.0	N
14	35,915	24,608	2.0	2.0	N
15	31,953	n/a	2.0	1.5	N
16	38,303	11,938	2.0	2.0	N
17	44,653	n/a	2.0	2.0	N
18	44,653	n/a	2.0	1.0	N
19	44,264	n/a	2.0	1.0	N
20	43,875	n/a	2.0	1.0	N
21	43,320	n/a	2.0	1.0	N
24	39,931	27,180	2.0	2.5	N
32	27,215	6,298	2.0	2.0	Y
33	27,215	n/a	2.0	1.0	N
34	27,215	n/a	2.0	1.0	N
35	30,569	49,204	2.0	3.0	N
36	33,804	n/a	2.0	1.0	N
37	30,526	6,938	2.0	1.0	N
38	20,940	10,944	2.0	1.0	N
39	14,633	n/a	2.0	1.0	N
40	30,109	n/a	2.0	1.0	N
41	30,109	3,882	2.0	1.0	N
42	33,560	n/a	2.0	1.0	N
43	33,560	n/a	2.0	1.0	N
44	40,524	11,658	2.0	1.0	N
45	47,502	n/a	2.0	1.0	N

Table C.1 (cont.)

Study ID	Major-Street AADT	Minor-Street AADT	Average Major-Street Through Lanes	Average Minor-Street Through Lanes	Freeway Adjacent
46	49,226	10,320	2.0	1.5	N
47	50,934	n/a	2.0	1.0	N
49	36,183	n/a	2.0	1.0	N
50	36,183	n/a	2.0	1.0	N
51	31,087	n/a	2.0	1.0	N
52	25,990	n/a	2.0	1.0	N
53	25,064	n/a	2.0	1.0	N
54	27,585	n/a	2.0	1.5	N
57	28,442	n/a	1.0	1.0	N
62	22,404	26,266	2.0	2.0	N
65	24,700	19,118	2.0	2.0	N
66	27,272	16,265	2.0	2.0	N
68	29,843	n/a	2.0	1.0	N
69	29,843	n/a	2.0	1.0	N
72	31,147	n/a	2.0	1.0	N
73	31,147	n/a	2.0	1.0	N
74	31,147	n/a	2.0	1.0	N
75	31,147	3,335	2.0	1.0	N
76	31,147	n/a	3.0	1.0	N
77	31,437	12,218	3.0	2.0	N
79	31,727	n/a	3.0	1.0	N
80	31,727	n/a	3.0	1.0	N
81	31,409	9,324	3.0	1.0	N
82	31,092	n/a	3.0	1.0	N
83	31,033	25,953	3.0	2.0	N
84	30,975	4,734	3.0	1.0	N
85	16,097	n/a	2.0	1.0	N
86	16,097	4,981	2.0	1.0	N
87	15,445	n/a	2.0	2.0	N
88	14,793	n/a	2.0	1.0	N
90	14,658	728	2.0	1.0	N
91	30,963	22,367	2.0	2.0	N
92	22,418	5,702	2.0	1.0	N
93	27,193	16,169	2.0	2.0	N
94	24,617	23,124	2.0	2.0	N
95	24,265	n/a	2.0	1.0	N
96	24,265	n/a	2.0	1.0	N
97	24,265	n/a	2.0	1.0	N
98	16,105	n/a	2.0	1.0	N
99	26,148	n/a	2.0	1.0	N
100	31,230	n/a	2.0	1.0	N
101	31,400	n/a	2.0	1.0	N
102	29,853	n/a	2.0	1.0	N
103	28,307	n/a	2.0	1.0	N
106	32,808	n/a	2.0	1.0	N
107	31,416	n/a	2.0	2.0	N
108	25,444	5,071	2.0	2.0	N
112	5,200	6,222	2.0	1.0	N
113	8,493	3,409	2.0	1.0	N

Table C.1 (cont.)

Study ID	Major-Street AADT	Minor-Street AADT	Average Major-Street Through Lanes	Average Minor-Street Through Lanes	Freeway Adjacent
114	8,493	4,272	2.0	1.0	N
115	10,847	7,869	1.0	1.0	N
116	10,349	n/a	2.0	1.0	N
117	20,725	n/a	2.0	1.0	N
118	26,469	8,890	2.0	1.0	N
119	32,213	3,850	2.0	1.0	N
120	32,213	n/a	2.0	2.0	N
121	34,166	n/a	2.0	1.0	N
122	35,334	n/a	2.0	1.0	Y
123	25,953	7,816	2.0	1.5	Y
126	41,925	10,998	2.0	1.0	N
130	18,193	n/a	2.0	1.0	N
131	18,193	n/a	2.0	1.0	N
132	18,193	n/a	1.0	1.0	N
133	18,830	n/a	1.0	1.0	N
134	22,586	n/a	2.0	1.0	N
135	22,586	5,905	2.0	1.0	N
136	22,586	n/a	2.0	1.0	N
137	24,791	2,911	2.0	1.0	N
138	26,995	n/a	2.0	1.0	N
139	22,252	25,105	2.0	2.0	N
140	27,474	10,428	2.0	1.0	N
141	28,868	n/a	2.0	1.0	N
142	30,908	n/a	2.0	1.0	N
143	36,400	50,158	2.0	3.0	N
144	39,469	7,692	2.0	1.0	N
145	37,264	8,776	2.0	1.0	N
146	41,773	19,564	3.0	2.0	Y
147	50,638	5,959	3.0	1.0	Y
148	50,638	n/a	3.0	1.0	N
149	42,155	44,480	3.0	3.0	N
150	31,148	n/a	3.0	1.5	N
151	34,635	n/a	3.0	1.0	N
152	34,635	9,350	3.0	1.0	N
153	33,855	28,476	2.0	3.0	N
154	32,370	n/a	2.0	1.0	N
155	32,370	10,390	2.0	1.0	N
157	26,409	41,116	2.0	4.0	N
158	23,647	13,459	2.0	1.0	N
159	23,580	n/a	2.0	1.0	N
160	22,097	19,621	2.0	2.0	N
161	22,039	n/a	3.0	3.0	N
162	28,212	n/a	3.0	2.0	N
163	31,276	28,194	3.0	2.0	N
164	54,723	31,826	3.0	2.0	N
165	46,757	n/a	3.0	1.0	N
166	46,757	n/a	3.0	1.0	N
167	46,757	n/a	3.0	1.0	N
168	45,629	31,394	3.0	2.0	N

Table C.1 (cont.)

Study ID	Major-Street AADT	Minor-Street AADT	Average Major-Street Through Lanes	Average Minor-Street Through Lanes	Freeway Adjacent
170	27,950	n/a	2.0	2.0	N
171	30,677	n/a	3.0	1.0	N
172	30,677	n/a	2.0	2.0	N
173	26,663	19,725	2.0	1.5	N
174	23,016	37,528	2.0	2.0	N
175	27,491	n/a	2.0	2.0	N
176	28,392	n/a	3.0	1.0	N
177	28,281	11,824	3.0	1.0	N
178	27,815	18,764	3.0	2.0	N
179	27,460	n/a	3.0	1.0	N
180	28,710	10,654	3.0	1.0	N
181	28,868	33,843	3.0	3.0	N

Appendix D. Study Site Geometry

Table D.1 Study Site Geometry

Study ID	Increasing Milepost Approach			Decreasing Milepost Approach		
	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length
2	200	170	750	2,430	260	260
3	950	100	100	750	100	200
4	600	80	220	540	100	240
5	600	90	160	790	140	220
6	520	120	350	670	200	200
7	1,840	150	150	7,640	280	180
9	1,460	220	190	480	100	230
10	920	140	140	230	180	80
11	2,220	250	210	40	90	200
12	370	225	100	100	80	100
13	350	n/a	100	250	70	100
14	30	n/a	100	180	70	100
15	430	60	100	430	60	100
16	430	80	100	430	80	100
17	430	80	100	430	80	100
18	90	80	100	90	80	100
19	90	60	100	290	70	100
20	270	70	100	70	70	100
21	130	70	100	90	70	100
24	90	70	100	90	90	100
32	500	100	200	1,470	100	100
33	1,470	n/a	100	630	80	80
34	630	280	100	600	110	100
35	110	130	220	210	110	170
36	230	n/a	100	20	n/a	100
37	110	90	100	220	80	100
38	80	100	100	380	100	100
39	1,260	100	100	340	100	100
40	330	180	70	450	90	100
41	820	180	100	340	170	180
42	100	80	80	120	80	100
43	570	140	100	200	100	100

Table D.1 (cont.)

Study ID	Increasing Milepost Approach			Decreasing Milepost Approach		
	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length
44	30	160	160	430	290	150
45	1,150	n/a	80	90	n/a	430
46	560	240	130	130	110	150
47	700	n/a	130	40	40	100
49	270	n/a	100	70	n/a	100
50	80	n/a	100	110	n/a	100
51	340	n/a	100	140	n/a	100
52	60	n/a	100	150	n/a	100
53	70	100	100	2,870	100	100
54	120	150	280	110	120	240
57	150	100	100	430	100	100
62	110	150	160	230	220	150
65	2,730	130	160	20	90	120
66	140	120	120	150	120	120
68	280	40	110	40	n/a	80
69	190	n/a	110	200	n/a	90
72	140	40	90	170	n/a	120
73	140	90	80	30	n/a	100
74	100	n/a	100	260	n/a	100
75	240	70	100	700	30	100
76	140	60	100	700	n/a	100
77	180	50	100	710	40	100
79	700	60	100	700	100	100
80	70	100	100	540	n/a	100
81	50	120	100	70	100	100
82	190	40	100	70	100	80
83	230	100	100	80	180	100
84	60	140	100	30	100	100
85	220	100	100	50	100	100
86	240	100	100	230	80	100
87	420	n/a	100	270	n/a	80
88	110	n/a	80	760	n/a	90
90	190	100	100	110	n/a	100
91	280	170	130	80	200	180
92	60	n/a	100	240	n/a	170
93	220	n/a	100	120	n/a	100
94	110	n/a	140	110	n/a	130
95	110	130	130	110	100	100
96	430	n/a	100	120	n/a	80
97	110	100	100	580	100	100
98	350	180	90	140	100	100
99	190	n/a	100	330	n/a	100
100	210	100	100	80	n/a	100
101	100	n/a	100	70	n/a	100
102	180	n/a	50	120	n/a	70
103	320	n/a	90	450	90	90
106	100	n/a	100	80	n/a	110

Table D.1 (cont.)

	Increasing Milepost Approach			Decreasing Milepost Approach		
Study ID	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length
107	130	n/a	110	390	n/a	90
108	190	n/a	100	330	100	100
112	1,640	n/a	130	80	80	100
113	100	n/a	90	290	n/a	110
114	200	n/a	100	130	n/a	100
115	130	90	90	190	90	90
116	200	120	60	110	100	60
117	220	n/a	100	250	n/a	100
118	150	n/a	100	360	n/a	100
119	40	n/a	100	160	n/a	100
120	70	n/a	100	150	n/a	110
121	150	n/a	140	230	n/a	140
122	40	n/a	120	770	130	130
123	470	170	200	210	180	180
126	400	300	150	490	n/a	330
130	1,300	n/a	320	80	n/a	360
131	350	n/a	330	200	n/a	300
132	20	290	120	40	n/a	90
133	210	n/a	100	130	n/a	100
134	180	150	150	120	n/a	90
135	120	n/a	90	80	n/a	80
136	110	n/a	80	170	n/a	80
137	450	n/a	80	380	n/a	80
138	430	n/a	100	50	n/a	100
139	50	n/a	180	200	n/a	100
140	130	n/a	110	50	n/a	110
141	60	n/a	90	50	n/a	110
142	80	70	140	50	100	150
143	150	170	210	120	170	190
144	600	190	130	120	110	110
145	50	150	100	40	290	100
146	290	n/a	100	1,000	160	490
147	590	n/a	300	150	n/a	310
148	160	n/a	100	70	n/a	100
149	50	n/a	240	80	n/a	200
150	420	n/a	100	120	n/a	100
151	200	n/a	180	20	n/a	150
152	50	n/a	160	140	n/a	90
153	280	120	170	410	120	120
154	120	n/a	90	80	n/a	90
155	70	n/a	110	90	n/a	160
157	30	n/a	200	80	n/a	230
158	120	n/a	90	30	n/a	120
159	70	n/a	120	60	n/a	120
160	140	n/a	110	70	n/a	140
161	80	330	200	140	260	190
162	120	230	330	210	190	200

Table D.1 (cont.)

	Increasing Milepost Approach			Decreasing Milepost Approach		
Study ID	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length
163	150	230	230	140	200	300
164	130	n/a	200	60	n/a	260
165	210	n/a	100	950	n/a	100
166	240	n/a	100	340	n/a	100
167	50	n/a	100	70	n/a	100
168	30	n/a	260	40	n/a	260
170	120	100	100	630	110	300
171	120	n/a	280	770	n/a	240
172	270	100	170	210	90	100
173	120	100	100	190	100	490
174	90	150	300	160	310	150
175	220	140	230	220	120	360
176	100	n/a	220	40	n/a	410
177	50	n/a	250	70	n/a	570
178	70	n/a	350	1,130	240	240
179	70	90	110	90	190	190
180	30	180	180	60	180	250
181	130	200	290	100	200	580

Appendix E. Study Site Functional Areas

Table E.1 Study Site Functional Areas

Study ID	Increasing Milepost Approach Functional Distance	Increasing Milepost Approach Functional Distance	Physical Area	Functional Area	Functional Area Overlap
2	950	740	140	1,830	Y
3	400	450	105	955	N
4	480	470	100	1,050	N
5	440	460	80	980	N
6	525	450	145	1,120	N
7	425	490	110	1,025	N
9	400	405	150	955	N
10	360	275	120	755	N
11	530	450	135	1,115	N
12	400	340	100	840	N
13	340	340	95	775	N
14	285	285	100	670	N
15	285	285	100	670	Y
16	285	285	135	705	Y
17	285	285	100	670	Y
18	285	285	90	660	Y
19	285	285	95	665	N
20	285	285	100	670	Y
21	285	285	90	660	Y
24	285	285	115	685	Y
32	450	400	115	965	Y
33	400	390	80	870	Y
34	490	405	100	995	Y
35	420	375	160	955	N
36	340	340	90	770	N
37	340	340	85	765	N
38	400	400	100	900	N
39	400	400	115	915	N
40	505	465	100	1,070	Y
41	505	505	145	1,155	Y
42	400	340	90	830	N
43	360	340	100	800	N
44	370	435	100	905	Y

Table E.1 (cont.)

Study ID	Increasing Milepost Approach Functional Distance	Increasing Milepost Approach Functional Distance	Physical Area	Functional Area	Functional Area Overlap
45	330	620	125	1,075	N
46	410	365	145	920	N
47	355	340	80	775	N
49	340	340	100	780	N
50	340	340	100	780	N
51	340	340	100	780	N
52	340	340	100	780	N
53	340	340	90	770	N
54	430	410	110	950	N
57	340	340	80	760	Y
62	370	400	115	885	N
65	370	350	115	835	N
66	350	350	110	810	N
68	345	330	80	755	Y
69	345	335	65	745	Y
72	280	295	80	655	N
73	280	285	100	665	N
74	285	285	100	670	N
75	285	285	100	670	N
76	285	285	100	670	N
77	285	285	110	680	N
79	285	285	100	670	N
80	285	285	100	670	N
81	295	285	95	675	N
82	285	285	100	670	N
83	340	380	125	845	N
84	360	340	80	780	N
85	285	285	70	640	N
86	285	285	80	650	N
87	235	225	110	570	N
88	225	230	80	535	N
90	235	235	80	550	N
91	435	450	140	1,025	N
92	340	375	90	805	N
93	340	340	100	780	N
94	360	355	110	825	N
95	300	285	90	675	N
96	285	275	80	640	N
97	285	285	105	675	N
98	380	340	165	885	N
99	285	285	90	660	N
100	285	285	80	650	N
101	285	285	80	650	N
102	260	270	80	610	N
103	335	335	100	770	N
106	235	240	106	581	N
107	240	230	106	576	N
108	235	235	110	580	N

Table E.1 (cont.)

Study ID	Increasing Milepost Approach Functional Distance	Increasing Milepost Approach Functional Distance	Physical Area	Functional Area	Functional Area Overlap
112	415	400	94	909	N
113	280	290	86	656	N
114	285	285	88	658	N
115	230	230	110	570	N
116	245	260	120	625	N
117	235	235	94	564	N
118	235	235	90	560	N
119	235	235	90	560	N
120	235	240	112	587	N
121	255	255	90	600	N
122	245	330	116	691	Y
123	450	440	132	1,022	Y
126	385	400	114	899	N
130	510	530	80	1,120	N
131	455	440	84	979	N
132	330	230	104	664	N
133	235	235	108	578	N
134	310	280	84	674	N
135	230	225	80	535	N
136	225	225	80	530	Y
137	225	225	100	550	Y
138	285	285	70	640	N
139	440	400	120	960	N
140	405	405	100	910	N
141	335	345	90	770	N
142	360	365	96	821	N
143	395	385	190	970	N
144	385	345	95	825	N
145	365	435	95	895	N
146	340	660	120	1,120	Y
147	650	445	105	1,200	Y
148	340	340	88	768	N
149	410	390	120	920	N
150	400	400	113	913	N
151	325	310	84	719	Y
152	315	280	100	695	Y
153	350	295	142	787	Y
154	320	280	85	685	Y
155	290	315	82	687	Y
157	350	350	140	840	N
158	280	295	90	665	N
159	295	295	78	668	N
160	290	305	103	698	N
161	515	480	167	1,162	N
162	515	460	145	1,120	N
163	520	500	142	1,162	N
164	390	420	123	933	Y
165	340	340	90	770	Y

Table E.1 (cont.)

Study ID	Increasing Milepost Approach Functional Distance	Increasing Milepost Approach Functional Distance	Physical Area	Functional Area	Functional Area Overlap
166	340	340	92	772	N
167	340	340	80	760	Y
168	420	420	128	968	Y
170	340	450	150	940	N
171	430	410	105	945	N
172	375	340	125	840	N
173	340	580	108	1,028	N
174	440	445	130	1,015	N
175	405	490	125	1,020	N
176	400	530	97	1,027	N
177	415	670	110	1,195	N
178	465	410	150	1,025	N
179	345	385	92	822	N
180	380	415	80	875	N
181	435	680	140	1,255	N

Appendix F. Study Site Accesses

Table F.1 Study Site Accesses

	Commercial				Residential			
Study ID	Accesses	Access Density	Conflict Points	Conflict Point Density	Accesses	Access Density	Conflict Points	Conflict Point Density
2	2	1.18	3	1.78	0	0.00	0	0.00
3	0	0.00	0	0.00	0	0.00	0	0.00
4	1	1.05	2	2.11	0	0.00	0	0.00
5	0	0.00	0	0.00	0	0.00	0	0.00
6	2	2.05	4	4.10	0	0.00	0	0.00
7	0	0.00	0	0.00	0	0.00	0	0.00
9	1	1.24	5	6.21	0	0.00	0	0.00
10	0	0.00	0	0.00	0	0.00	0	0.00
11	9	9.18	123	125.51	0	0.00	0	0.00
12	8	10.81	106	143.24	0	0.00	0	0.00
13	10	14.71	123	180.88	0	0.00	0	0.00
14	8	14.04	111	194.74	0	0.00	0	0.00
15	3	5.26	33	57.89	0	0.00	0	0.00
16	0	0.00	0	0.00	0	0.00	0	0.00
17	1	1.75	11	19.30	0	0.00	0	0.00
18	3	5.26	39	68.42	0	0.00	0	0.00
19	3	5.26	55	96.49	0	0.00	0	0.00
20	0	0.00	0	0.00	11	19.30	113	198.25
21	0	0.00	0	0.00	13	22.81	143	250.88
24	8	14.04	100	175.44	0	0.00	0	0.00
32	0	0.00	0	0.00	0	0.00	0	0.00
33	0	0.00	0	0.00	0	0.00	0	0.00
34	0	0.00	0	0.00	0	0.00	0	0.00
35	9	11.32	27	33.96	0	0.00	0	0.00
36	1	1.47	11	16.18	7	10.29	94	138.24
37	0	0.00	0	0.00	4	5.88	70	102.94
38	6	7.50	61	76.25	3	3.75	37	46.25
39	0	0.00	0	0.00	0	0.00	0	0.00
40	3	3.09	33	34.02	0	0.00	0	0.00
41	3	2.97	33	32.67	0	0.00	0	0.00
42	6	8.11	92	124.32	2	2.70	22	29.73
43	5	7.14	55	78.57	0	0.00	0	0.00
44	10	12.42	147	182.61	0	0.00	0	0.00

Table F.1 (cont.)

Study ID	Commercial				Residential			
	Accesses	Access Density	Conflict Points	Conflict Point Density	Accesses	Access Density	Conflict Points	Conflict Point Density
45	2	2.11	14	14.74	2	2.11	22	23.16
46	6	7.74	62	80.00	0	0.00	0	0.00
47	9	12.95	91	130.94	5	7.19	55	79.14
49	1	1.47	5	7.35	12	17.65	162	238.24
50	0	0.00	0	0.00	17	25.00	217	319.12
51	3	4.41	33	48.53	9	13.24	112	164.71
52	0	0.00	0	0.00	10	14.71	143	210.29
53	4	5.88	48	70.59	2	2.94	22	32.35
54	8	9.52	104	123.81	0	0.00	0	0.00
57	0	0.00	0	0.00	4	5.88	44	64.71
62	6	7.79	83	107.79	0	0.00	0	0.00
65	4	5.56	48	66.67	0	0.00	0	0.00
66	14	20.00	142	202.86	0	0.00	0	0.00
68	12	17.78	146	216.30	0	0.00	0	0.00
69	8	11.76	109	160.29	0	0.00	0	0.00
72	10	17.39	111	193.04	0	0.00	0	0.00
73	7	12.39	89	157.52	0	0.00	0	0.00
74	8	14.04	105	184.21	0	0.00	0	0.00
75	2	3.51	24	42.11	0	0.00	0	0.00
76	2	3.51	12	21.05	0	0.00	0	0.00
77	2	3.51	26	45.61	0	0.00	0	0.00
79	3	5.26	39	68.42	0	0.00	0	0.00
80	5	8.77	62	108.77	0	0.00	0	0.00
81	10	17.24	160	275.86	1	1.72	13	22.41
82	5	8.77	65	114.04	1	1.75	17	29.82
83	12	16.67	147	204.17	0	0.00	0	0.00
84	12	17.14	174	248.57	3	4.29	39	55.71
85	4	7.02	48	84.21	0	0.00	0	0.00
86	6	10.53	79	138.60	1	1.75	11	19.30
87	1	2.17	11	23.91	0	0.00	0	0.00
88	3	6.59	33	72.53	0	0.00	0	0.00
90	8	17.02	90	191.49	2	4.26	22	46.81
91	10	11.30	170	192.09	0	0.00	0	0.00
92	9	12.59	101	141.26	0	0.00	0	0.00
93	10	14.71	114	167.65	0	0.00	0	0.00
94	11	15.38	153	213.99	1	1.40	11	15.38
95	6	10.26	83	141.88	0	0.00	0	0.00
96	7	12.50	77	137.50	0	0.00	0	0.00
97	7	12.28	65	114.04	0	0.00	0	0.00
98	6	8.33	74	102.78	0	0.00	0	0.00
99	1	1.75	11	19.30	0	0.00	0	0.00
100	6	10.53	94	164.91	0	0.00	0	0.00
101	8	14.04	52	91.23	5	8.77	55	96.49
102	7	13.21	72	135.85	0	0.00	0	0.00
103	1	1.49	11	16.42	0	0.00	0	0.00
106	8	16.84	133	280.00	1	2.11	24	50.53
107	3	6.38	35	74.47	0	0.00	0	0.00
108	2	4.26	22	46.81	0	0.00	0	0.00

Table F.1 (cont.)

	Commercial				Residential			
Study ID	Accesses	Access Density	Conflict Points	Conflict Point Density	Accesses	Access Density	Conflict Points	Conflict Point Density
112	0	0.00	0	0.00	2	2.45	24	29.45
113	11	19.30	154	270.18	0	0.00	0	0.00
114	7	12.28	85	149.12	2	3.51	38	66.67
115	7	15.22	63	136.96	0	0.00	0	0.00
116	9	17.82	99	196.04	0	0.00	0	0.00
117	5	10.64	55	117.02	0	0.00	0	0.00
118	3	6.38	50	106.38	0	0.00	0	0.00
119	5	10.64	83	176.60	0	0.00	0	0.00
120	9	18.95	83	174.74	0	0.00	0	0.00
121	6	11.76	66	129.41	0	0.00	0	0.00
122	4	6.96	74	128.70	0	0.00	0	0.00
123	2	2.25	48	53.93	0	0.00	0	0.00
126	2	2.55	13	16.56	0	0.00	0	0.00
130	2	1.92	26	25.00	0	0.00	0	0.00
131	10	11.17	112	125.14	0	0.00	0	0.00
132	10	17.86	128	228.57	2	3.57	22	39.29
133	8	17.02	62	131.91	0	0.00	0	0.00
134	8	13.56	103	174.58	0	0.00	0	0.00
135	7	15.38	92	202.20	0	0.00	0	0.00
136	4	8.89	44	97.78	0	0.00	0	0.00
137	0	0.00	0	0.00	0	0.00	0	0.00
138	10	17.54	125	219.30	0	0.00	0	0.00
139	12	14.29	138	164.29	2	2.38	22	26.19
140	8	9.88	105	129.63	0	0.00	0	0.00
141	10	14.71	121	177.94	1	1.47	11	16.18
142	8	11.03	96	132.41	0	0.00	0	0.00
143	10	12.82	130	166.67	0	0.00	0	0.00
144	7	9.59	83	113.70	0	0.00	0	0.00
145	12	15.00	141	176.25	0	0.00	0	0.00
146	4	4.00	8	8.00	0	0.00	0	0.00
147	5	4.57	53	48.40	0	0.00	0	0.00
148	13	19.12	203	298.53	0	0.00	0	0.00
149	12	15.00	57	71.25	0	0.00	0	0.00
150	3	3.75	17	21.25	1	1.25	13	16.25
151	10	15.75	136	214.17	2	3.15	26	40.94
152	7	11.76	92	154.62	0	0.00	0	0.00
153	8	12.40	33	51.16	0	0.00	0	0.00
154	9	15.00	78	130.00	0	0.00	0	0.00
155	11	18.18	140	231.40	2	3.31	22	36.36
157	10	14.29	110	157.14	0	0.00	0	0.00
158	7	12.17	94	163.48	0	0.00	0	0.00
159	10	16.95	114	193.22	1	1.69	11	18.64
160	11	18.49	93	156.30	0	0.00	0	0.00
161	9	9.05	90	90.45	0	0.00	0	0.00
162	11	11.28	99	101.54	0	0.00	0	0.00
163	9	8.82	18	17.65	0	0.00	0	0.00
164	12	14.81	38	46.91	0	0.00	0	0.00
165	3	4.41	25	36.76	1	1.47	6	8.82

Table F.1 (cont.)

	Commercial				Residential			
Study ID	Accesses	Access Density	Conflict Points	Conflict Point Density	Accesses	Access Density	Conflict Points	Conflict Point Density
166	2	2.94	22	32.35	0	0.00	0	0.00
167	11	16.18	162	238.24	0	0.00	0	0.00
168	13	15.48	52	61.90	0	0.00	0	0.00
170	5	6.33	50	63.29	0	0.00	0	0.00
171	2	2.38	4	4.76	2	2.38	4	4.76
172	2	2.80	13	18.18	2	2.80	13	18.18
173	7	7.61	22	23.91	3	3.26	33	35.87
174	6	6.78	12	13.56	1	1.13	2	2.26
175	7	7.82	54	60.34	0	0.00	0	0.00
176	14	15.05	28	30.11	0	0.00	0	0.00
177	20	18.43	35	32.26	0	0.00	0	0.00
178	6	6.86	12	13.71	0	0.00	0	0.00
179	7	9.59	25	34.25	0	0.00	0	0.00
180	13	16.35	26	32.70	0	0.00	0	0.00
181	11	9.87	25	22.42	0	0.00	0	0.00

Appendix G. Study Site Crashes and Crash Severities

Table G.1 Study Site Crashes and Crash Severities

	Crash Severity						
Study ID	**No Injury**	**Possible Injury**	**Bruises & Abrasions**	**Broken Bones or Bleeding Wounds**	**Fatal**	**Total Crashes**	**Severity Costs**
2	89	41	7	7	0	144	8,169
3	45	17	8	8	0	78	7,832
4	62	27	7	1	0	97	2,752
5	56	26	13	2	0	97	3,948
6	54	37	5	1	0	97	2,977
7	30	11	5	0	0	46	994
9	33	12	8	6	0	59	5,999
10	55	18	12	6	0	91	6,668
11	27	12	1	1	0	41	1,488
12	16	5	3	2	1	27	2,875
13	32	15	9	1	0	57	2,276
14	32	9	8	4	0	53	4,299
15	17	9	3	0	0	29	693
16	24	6	3	1	0	34	1,383
17	11	5	1	3	0	20	2,693
18	16	9	4	3	1	33	3,908
19	19	10	5	6	0	40	5,614
20	27	20	8	1	0	56	2,384
21	34	15	4	4	0	57	4,240
24	83	22	11	7	0	123	7,664
32	43	14	1	3	0	61	3,212
33	9	4	1	3	0	17	2,643
34	15	6	5	1	0	27	1,503
35	59	22	9	4	0	94	5,044
36	22	14	3	2	0	41	2,495
37	13	6	4	0	0	23	629
38	31	23	4	0	0	58	1,422
39	9	1	3	0	0	13	322
40	31	9	7	2	0	49	2,644
41	26	13	5	3	0	47	3,415
42	29	12	3	0	0	44	872
43	45	24	6	2	0	77	3,256
44	75	47	10	2	0	134	4,674

Table G.1 (cont.)

	Crash Severity						
Study ID	No Injury	Possible Injury	Bruises & Abrasions	Broken Bones or Bleeding Wounds	Fatal	Total Crashes	Severity Costs
45	37	8	2	5	0	52	4,584
46	29	23	3	3	0	58	3,689
47	19	12	4	1	0	36	1,693
49	13	11	1	1	0	26	1,384
50	21	10	0	1	0	32	1,297
51	14	9	4	2	0	29	2,330
52	11	4	1	1	0	17	1,081
53	17	11	1	4	0	33	3,757
54	38	9	7	0	1	55	1,890
57	6	4	2	1	0	13	1,139
62	38	15	4	1	0	58	1,902
65	37	26	8	5	0	76	5,820
66	23	10	5	1	0	39	1,706
68	16	8	4	1	0	29	1,511
69	18	16	3	3	0	40	3,346
72	16	10	2	4	0	32	3,790
73	17	11	8	1	0	37	1,962
74	24	9	5	0	0	38	884
75	24	16	1	1	1	43	2,428
76	22	8	2	6	0	38	5,303
77	31	18	4	1	1	55	2,782
79	17	14	3	0	0	34	903
80	15	3	0	1	0	19	977
81	14	11	2	5	1	33	5,394
82	11	6	0	1	0	18	1,085
83	65	32	5	5	0	107	5,955
84	24	17	4	5	0	50	5,065
85	6	0	1	0	0	7	106
86	5	5	3	0	0	13	472
87	5	2	1	0	0	8	186
88	3	0	1	0	0	4	93
90	10	2	1	0	0	13	208
91	37	18	6	1	1	63	2,969
92	9	7	2	2	0	20	2,064
93	57	34	8	3	0	102	4,674
94	57	21	10	6	0	94	6,643
95	11	2	0	0	0	13	132
96	22	7	2	0	0	31	551
97	22	1	2	1	0	26	1,084
98	15	7	2	1	0	25	1,305
99	16	9	1	3	0	29	2,883
100	32	6	0	1	0	39	1,178
101	26	5	2	1	0	34	1,269
102	13	6	2	3	1	25	3,609
103	31	15	3	4	0	53	4,146
106	18	7	0	1	0	26	1,158
107	29	8	5	1	0	43	1,649
108	39	8	8	3	0	58	3,503

Table G.1 (cont.)

	Crash Severity						
Study ID	No Injury	Possible Injury	Bruises & Abrasions	Broken Bones or Bleeding Wounds	Fatal	Total Crashes	Severity Costs
112	8	5	2	1	0	16	1,190
113	13	3	1	1	0	18	1,048
114	4	1	0	0	0	5	60
115	12	3	2	0	0	17	339
116	12	4	3	1	0	20	1,246
117	19	9	2	1	0	31	1,407
118	21	7	4	1	0	33	1,491
119	18	4	2	0	0	24	407
120	32	7	6	0	1	46	1,700
121	41	18	2	1	0	62	1,881
122	74	29	3	0	0	106	1,784
123	52	18	3	6	0	79	5,935
126	35	11	9	1	0	56	2,121
130	8	6	5	1	0	20	1,472
131	16	17	3	0	0	36	1,024
132	15	9	6	0	0	30	924
133	8	7	1	1	0	17	1,194
134	19	5	1	3	0	28	2,729
135	27	5	3	2	0	37	2,139
136	10	4	2	3	0	19	2,727
137	26	1	0	1	0	28	941
138	11	2	1	1	0	15	997
139	88	56	26	13	0	183	15,024
140	37	29	8	5	0	79	5,946
141	23	19	5	2	0	49	2,869
142	38	29	13	6	0	86	7,135
143	109	50	25	3	0	187	6,935
144	96	80	17	10	0	203	12,992
145	51	38	20	3	0	112	5,775
146	50	42	15	5	0	112	7,109
147	30	26	5	1	0	62	2,409
148	36	14	5	2	0	57	2,716
149	55	26	10	3	0	94	4,489
150	35	19	4	2	0	60	2,842
151	38	19	8	2	0	67	3,175
152	51	29	6	8	0	94	8,202
153	74	25	14	4	1	118	6,421
154	39	18	8	2	0	67	3,138
155	32	13	9	0	0	54	1,407
157	72	33	11	4	0	120	5,723
158	47	9	2	1	0	59	1,530
159	15	7	1	1	0	24	1,225
160	58	14	9	0	0	81	1,563
161	64	33	6	3	0	106	4,503
162	66	27	7	1	1	102	3,554
163	43	13	3	1	0	60	1,760
164	149	55	24	14	0	242	15,876
165	41	16	2	2	0	61	2,582

Table G.1 (cont.)

	Crash Severity						
Study ID	No Injury	Possible Injury	Bruises & Abrasions	Broken Bones or Bleeding Wounds	Fatal	Total Crashes	Severity Costs
166	27	16	3	0	0	46	1,031
167	26	17	6	4	0	53	4,448
168	109	53	26	18	0	206	18,916
170	49	26	3	1	0	79	2,333
171	27	25	2	3	0	57	3,684
172	36	25	3	3	0	67	3,803
173	29	28	1	0	0	58	1,384
174	55	40	6	4	0	105	5,542
175	71	21	11	9	0	112	9,139
176	32	16	7	5	0	60	5,298
177	88	30	15	6	0	139	7,557
178	77	40	4	2	0	123	3,909
179	23	6	4	0	0	33	673
180	30	12	5	5	0	52	4,961
181	87	32	8	9	1	137	10,217

Appendix H. Study Site Crash Types and Crash Rates

Table H.1 Study Type Crash Types and Crash Rates

	Crash Type						
Study ID	**Right Angle**	**Rear End**	**Side Swipe**	**Single Vehicle**	**Head On**	**Other**	**Crash Rate**
2	30	83	14	10	0	7	3.43
3	45	24	1	4	0	4	1.86
4	36	52	3	1	0	5	2.31
5	37	47	4	4	1	4	2.31
6	19	59	8	9	0	2	2.10
7	8	30	3	0	1	4	1.00
9	12	39	1	5	0	2	1.42
10	57	29	1	4	0	0	2.95
11	19	17	2	1	0	2	1.32
12	13	8	3	0	0	3	0.87
13	30	20	1	2	0	4	1.83
14	25	23	1	1	0	3	1.35
15	14	14	1	0	0	0	0.83
16	14	14	0	1	0	5	0.81
17	5	11	0	1	0	3	0.41
18	11	14	1	6	0	1	0.67
19	17	12	2	5	1	3	0.83
20	23	29	0	3	1	0	1.17
21	27	26	0	4	0	0	1.20
24	53	54	4	7	0	5	2.81
32	35	16	2	0	1	7	2.05
33	10	5	0	2	0	0	0.57
34	12	12	1	2	0	0	0.91
35	24	46	8	12	0	4	2.81
36	11	24	0	3	0	3	1.11
37	14	5	1	2	1	0	0.69
38	31	15	5	2	1	4	2.53
39	2	7	0	4	0	0	0.81
40	20	23	2	3	1	0	1.49
41	22	18	1	4	0	2	1.43
42	21	18	2	3	0	0	1.20
43	34	40	0	2	1	0	2.10
44	49	75	4	2	1	3	3.02

Table H.1 (cont.)

	Crash Type						
Study ID	Right Angle	Rear End	Side Swipe	Single Vehicle	Head On	Other	Crash Rate
45	5	39	0	3	0	5	1.00
46	24	24	2	2	0	6	1.08
47	5	27	1	1	0	2	0.65
49	5	15	0	2	0	4	0.66
50	12	13	1	1	1	4	0.81
51	13	9	0	4	0	3	0.85
52	6	7	1	3	0	0	0.60
53	23	9	1	0	0	0	1.20
54	35	8	2	6	1	3	1.82
57	8	4	0	1	0	0	0.42
62	18	31	4	1	1	3	2.36
65	47	18	3	5	0	3	2.81
66	21	11	4	1	1	1	1.31
68	10	14	1	3	0	1	0.89
69	11	24	0	2	0	3	1.22
72	11	21	0	0	0	0	0.94
73	10	23	2	2	0	0	1.08
74	10	24	2	1	0	1	1.11
75	14	24	2	1	0	2	1.26
76	10	18	3	3	0	4	1.11
77	20	24	4	6	0	1	1.60
79	8	19	2	2	0	3	0.98
80	8	10	0	1	0	0	0.55
81	15	9	4	2	0	3	0.96
82	7	7	0	4	0	0	0.53
83	54	35	3	8	0	7	3.15
84	21	22	2	3	0	2	1.47
85	5	2	0	0	0	0	0.40
86	7	3	0	2	0	1	0.74
87	6	1	0	1	0	0	0.47
88	3	0	0	1	0	0	0.25
90	4	7	0	1	0	1	0.81
91	31	18	4	5	1	4	1.86
92	11	8	0	1	0	0	0.81
93	54	34	5	2	0	7	3.43
94	53	30	0	6	0	5	3.49
95	8	3	0	0	0	2	0.49
96	11	14	0	5	0	1	1.17
97	8	11	2	3	0	2	0.98
98	9	8	2	2	0	4	1.42
99	14	13	0	0	1	1	1.01
100	21	10	1	3	2	2	1.14
101	17	13	0	0	0	4	0.99
102	13	7	2	1	2	0	0.76
103	31	18	2	1	0	1	1.71
106	10	14	2	0	0	0	0.72
107	15	19	1	3	0	5	1.25
108	27	16	2	3	1	9	2.08

Table H.1 (cont.)

	Crash Type						
Study ID	Right Angle	Rear End	Side Swipe	Single Vehicle	Head On	Other	Crash Rate
112	13	2	0	1	0	0	2.81
113	7	5	2	0	0	4	1.94
114	3	0	1	0	0	1	0.54
115	7	9	0	0	0	1	1.43
116	9	8	1	2	0	0	1.76
117	7	19	2	2	0	1	1.37
118	12	17	0	3	0	1	1.14
119	6	15	2	0	0	1	0.68
120	13	26	1	5	0	1	1.30
121	11	48	3	0	0	0	1.66
122	12	78	4	2	0	10	2.74
123	47	22	5	1	1	3	2.78
126	33	18	1	2	0	2	1.26
130	5	8	0	3	0	4	1.00
131	16	14	1	0	0	5	1.81
132	7	18	2	3	0	0	1.51
133	2	10	0	3	0	2	0.82
134	12	10	2	2	0	2	1.13
135	17	8	1	6	0	5	1.50
136	5	8	1	2	0	3	0.77
137	10	8	1	2	0	7	1.03
138	10	1	0	1	0	3	0.51
139	130	41	2	3	2	5	7.51
140	50	17	3	5	0	4	2.63
141	32	11	0	3	1	2	1.55
142	43	27	4	6	2	4	2.54
143	88	77	7	4	2	9	4.69
144	88	93	7	8	2	5	4.70
145	45	43	5	14	0	5	2.74
146	31	65	8	5	0	3	2.45
147	18	32	7	3	0	2	1.12
148	23	21	8	2	0	3	1.03
149	18	57	6	6	1	6	2.04
150	24	25	5	0	0	6	1.76
151	23	32	4	7	0	1	1.77
152	51	32	2	5	2	2	2.48
153	43	48	9	11	0	7	3.18
154	20	31	4	10	1	1	1.89
155	20	19	3	11	0	1	1.52
157	41	54	9	9	0	7	4.15
158	19	26	6	2	0	6	2.28
159	10	10	2	0	1	1	0.93
160	44	17	8	6	0	6	3.35
161	36	44	8	6	2	10	4.39
162	40	37	11	8	0	6	3.30
163	23	25	4	4	1	3	1.75
164	103	94	23	4	1	17	4.04
165	20	33	2	1	0	5	1.19

Table H.1 (cont.)

	Crash Type						
Study ID	Right Angle	Rear End	Side Swipe	Single Vehicle	Head On	Other	Crash Rate
166	16	24	1	5	0	0	0.90
167	24	21	4	4	0	0	1.04
168	107	56	8	22	0	13	4.12
170	23	42	7	4	0	3	2.58
171	21	30	1	2	0	3	1.70
172	32	29	4	2	0	0	1.99
173	7	39	4	1	0	7	1.99
174	33	51	7	4	1	9	4.17
175	54	35	11	6	0	6	3.72
176	29	16	2	6	1	6	1.93
177	56	60	8	5	0	10	4.49
178	28	81	9	2	1	2	4.04
179	4	18	3	3	0	5	1.10
180	18	22	4	3	0	5	1.65
181	39	66	15	6	1	10	4.33

Appendix I. Crash Totals Model Variable Relationships

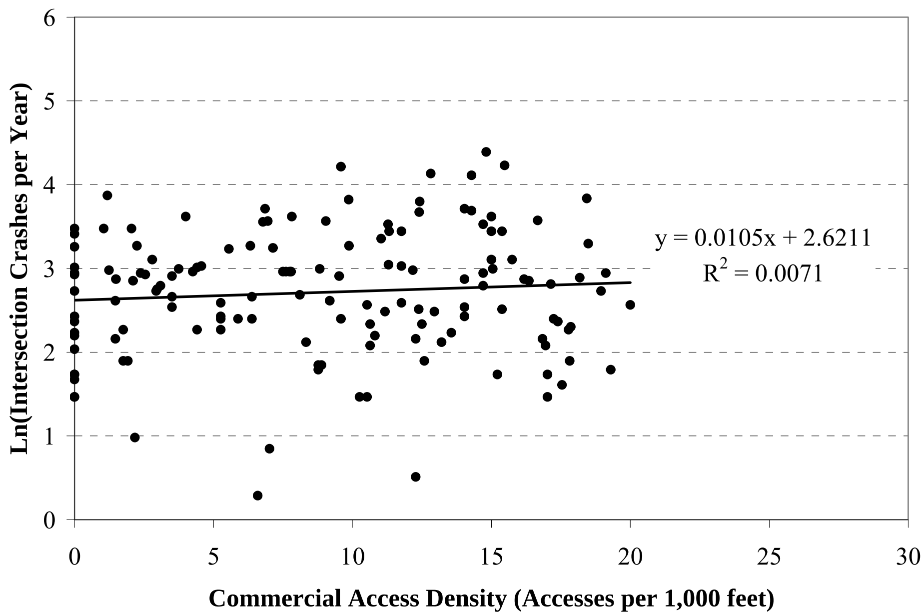

Figure I.1 Functional area crashes versus commercial access density.

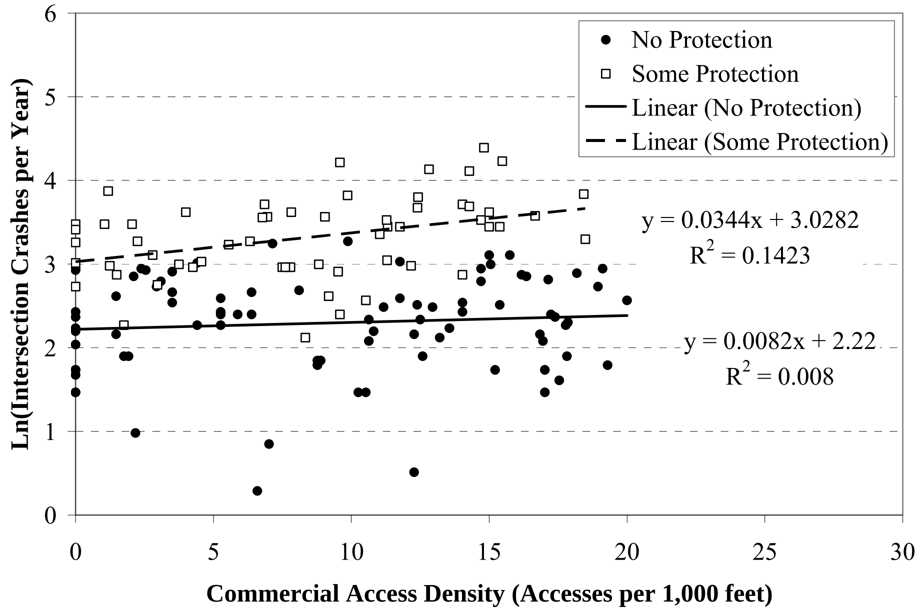

Figure I.2 Functional area crashes versus commercial access density by minor-street left-turn protection.

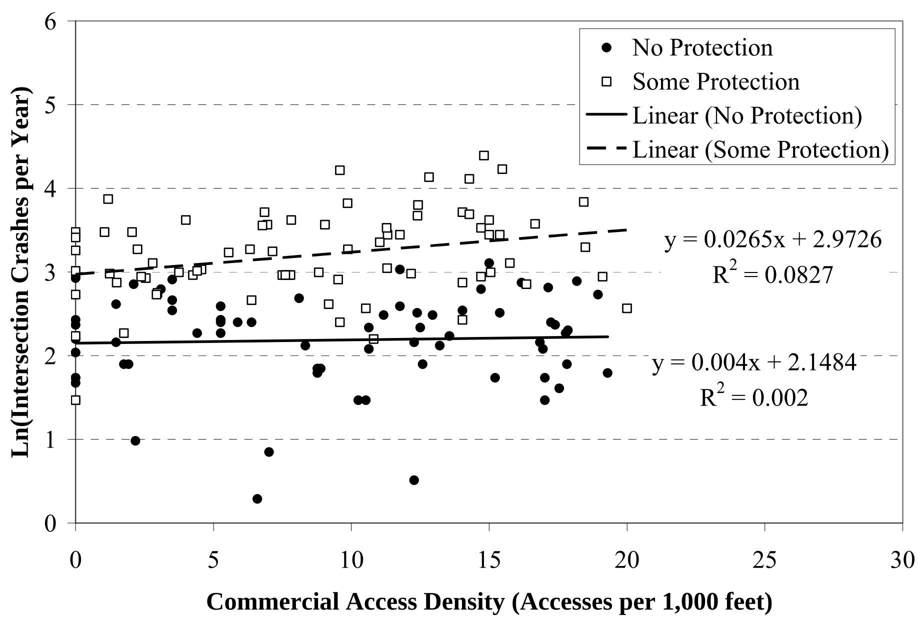

Figure I.3 Functional area crashes versus commercial access density by major-street left-turn protection.

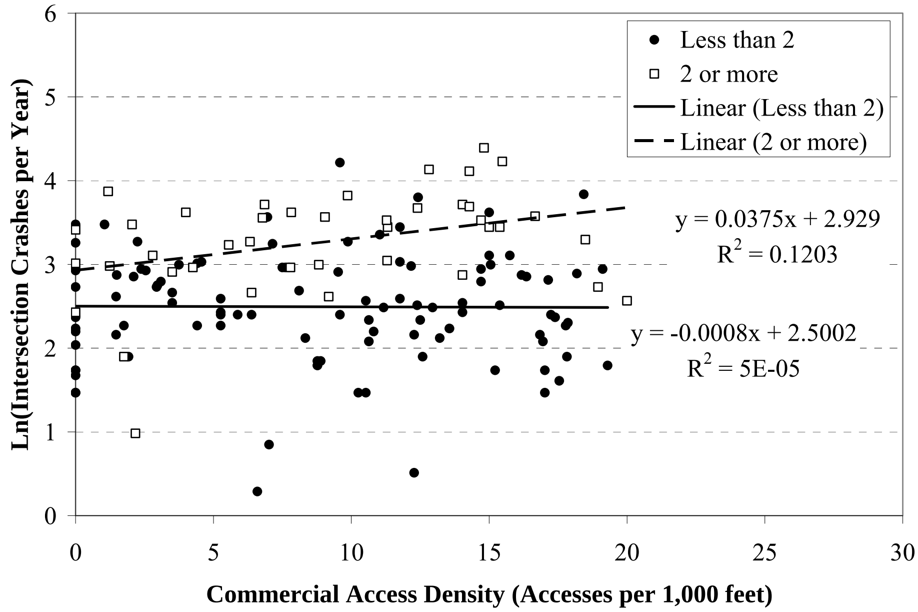

Figure I.4 Functional area crashes versus commercial access density by minor-street though lanes.

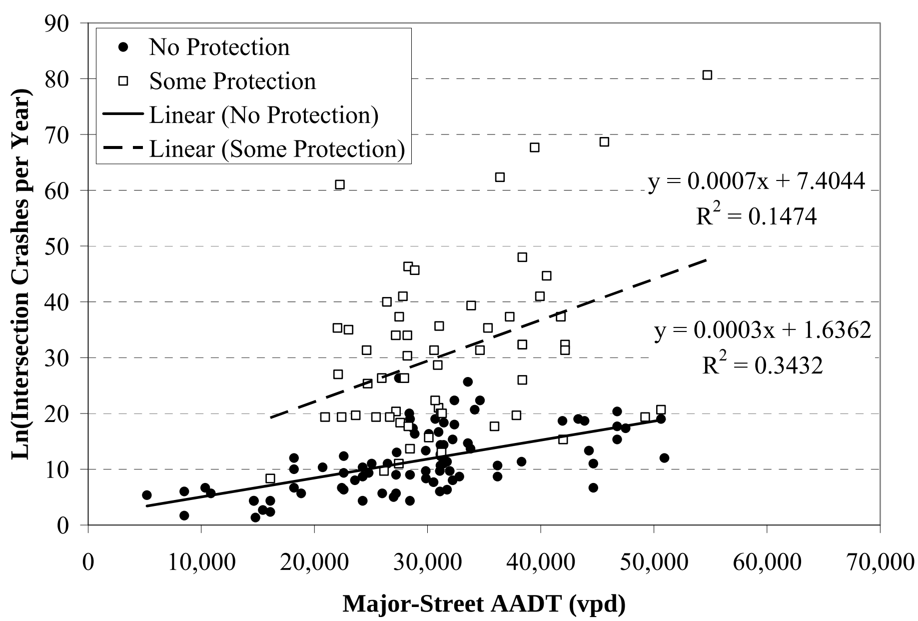

Figure I.5 Functional area crashes versus major-street AADT by minor-street left-turn protection.

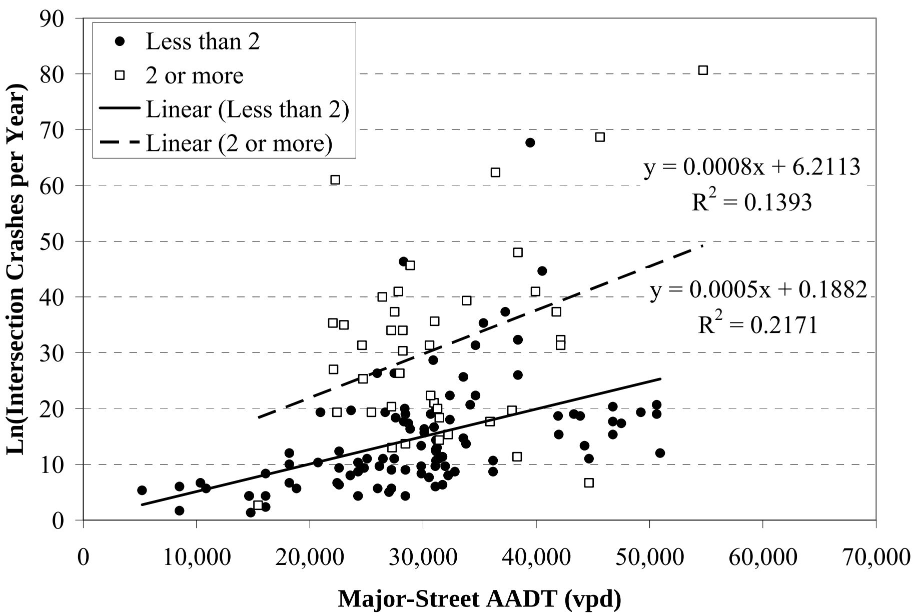

Figure I.6 Functional area crashes versus major-street AADT by minor-street through lanes.

Appendix J. Crash Rate Model Variable Relationships

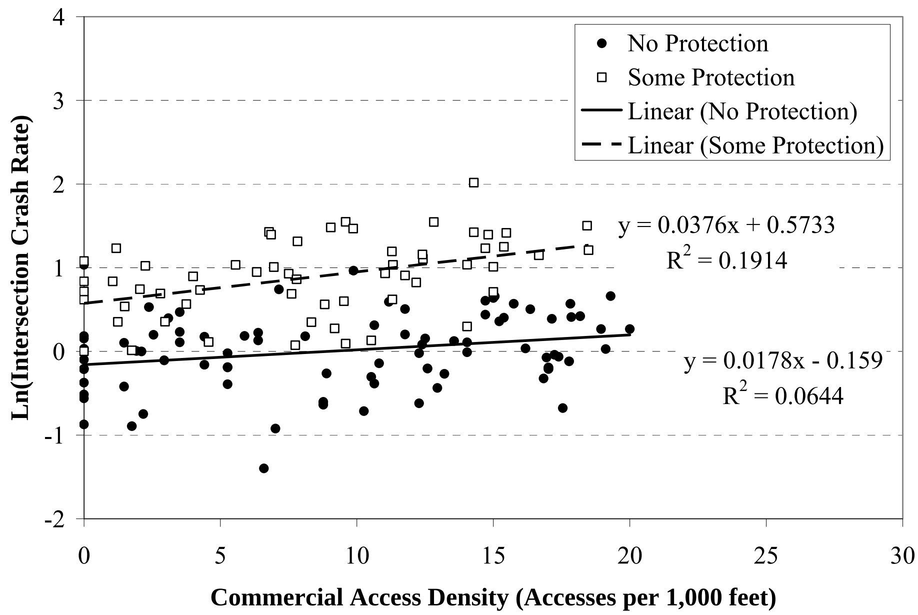

Figure J.1 Functional area crash rate versus commercial access density by minor-street left-turn protection.

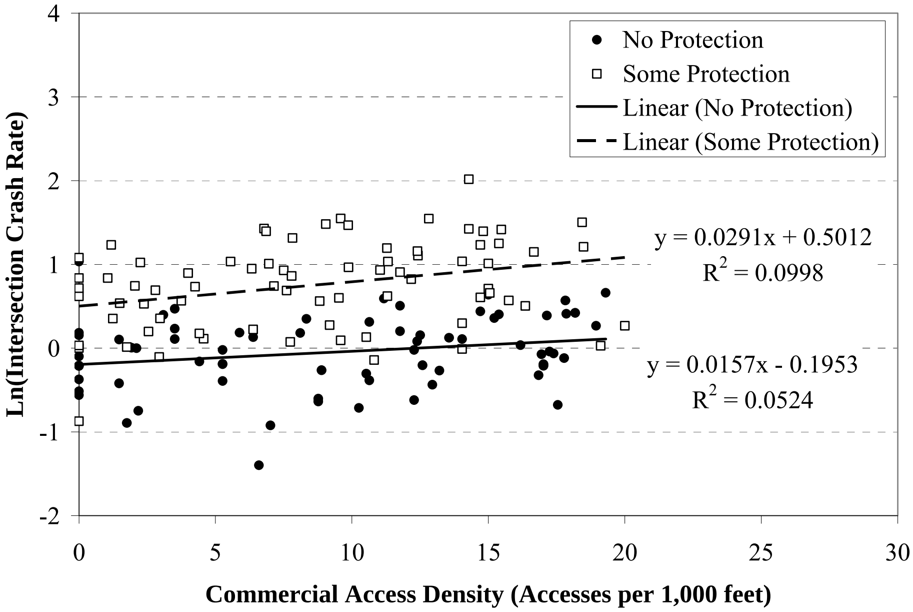

Figure J.2 Functional area crash rate versus commercial access density by major-street left-turn protection.

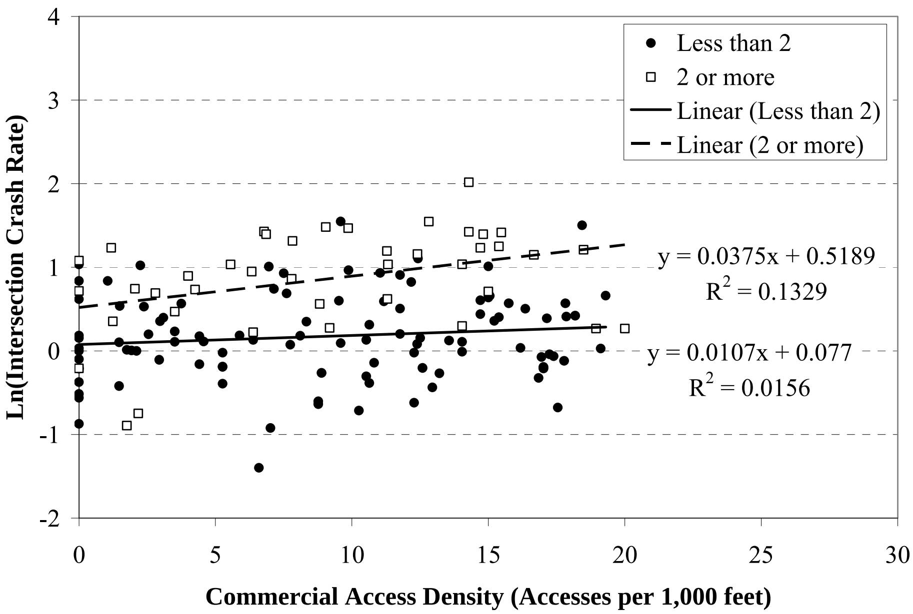

Figure J.3 Functional area crash rate versus commercial access density by minor-street through lanes.

Appendix K. Crash Severity Model Variable Relationships

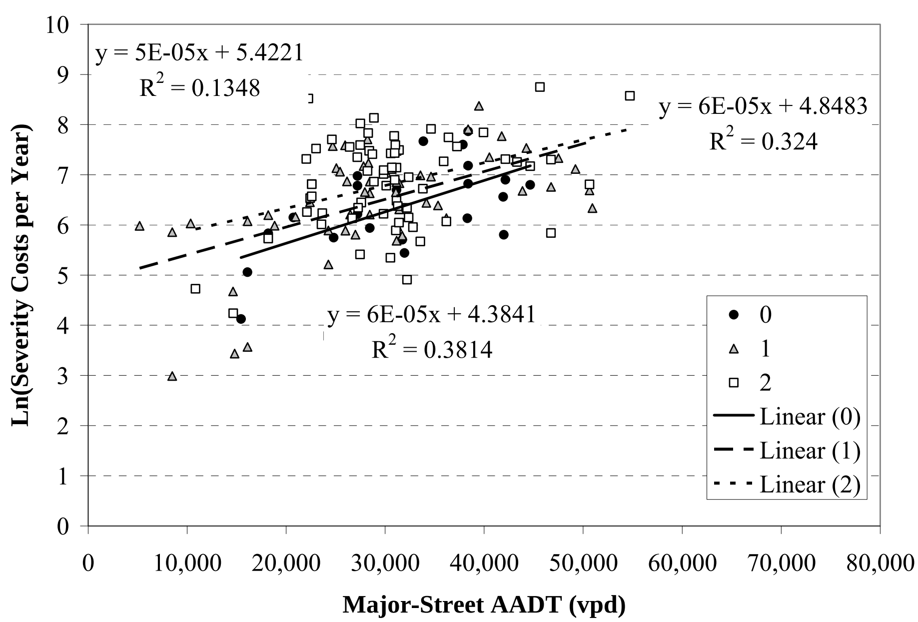

Figure K.1 Functional area crash severity costs versus major-street AADT by corner clearance score.

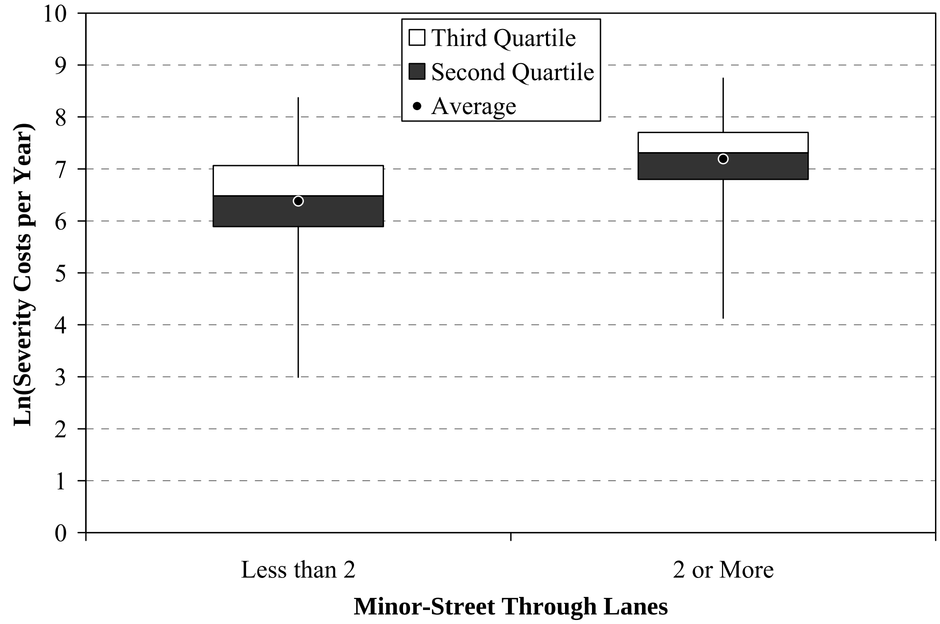

Figure K.2 Box plot of functional area crash severity costs and minor-street through lanes.

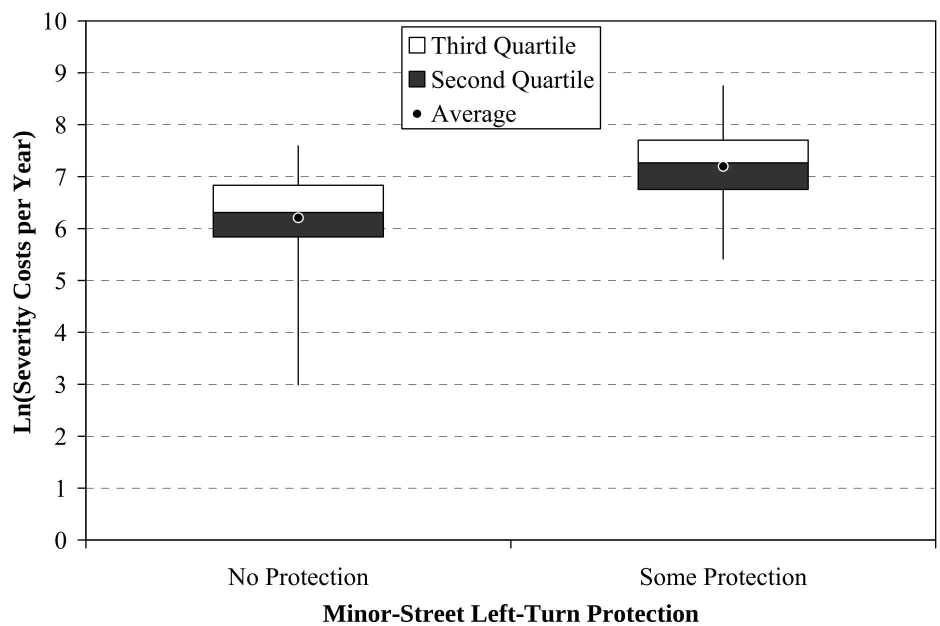

Figure K.3 Box plot of functional area crash severity costs and minor-street left-turn protection.

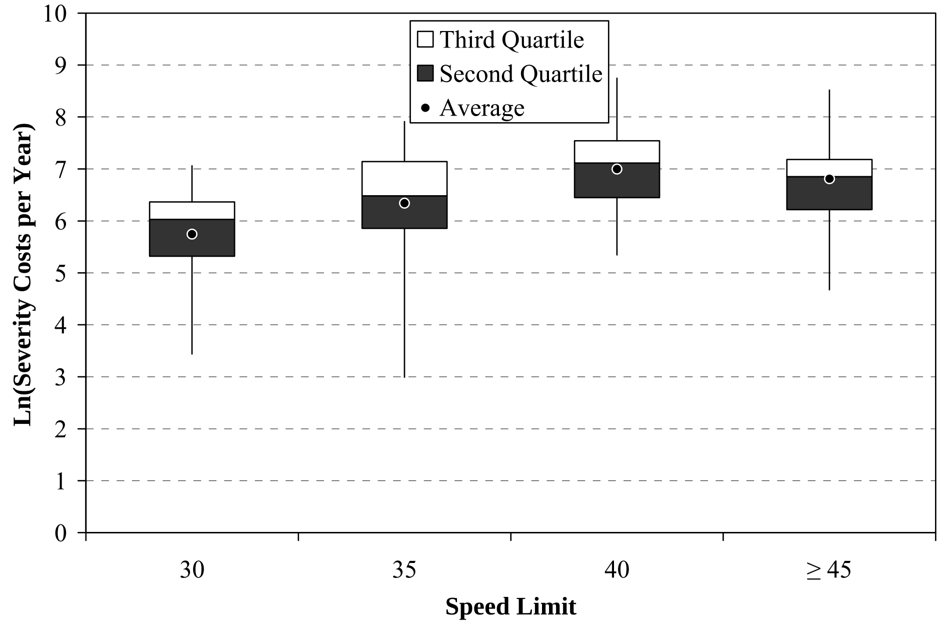

Figure K.4 Box plot of functional area crash severity costs and speed limit.

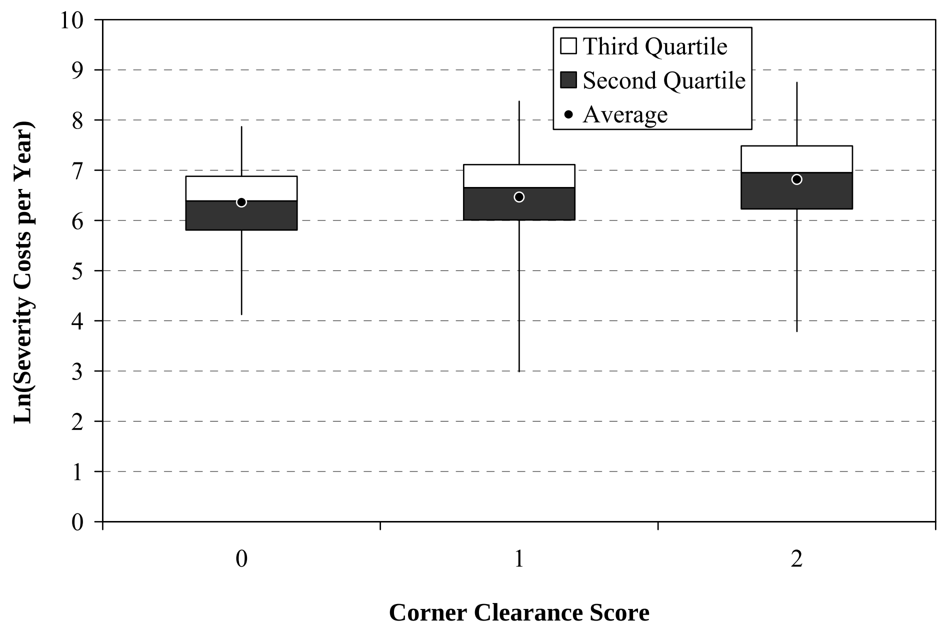

Figure K.5 Box plot of functional area crash severity costs and corner clearance score.

Appendix L. Right Angle Model Variable Relationships

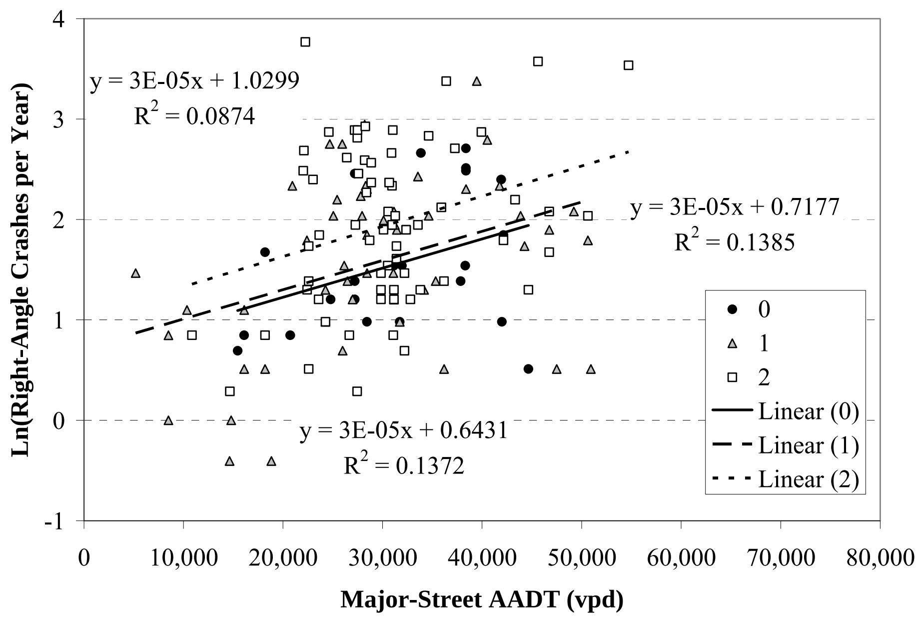

Figure L.1 Functional right-angle crashes versus major-street AADT by corner clearance score.

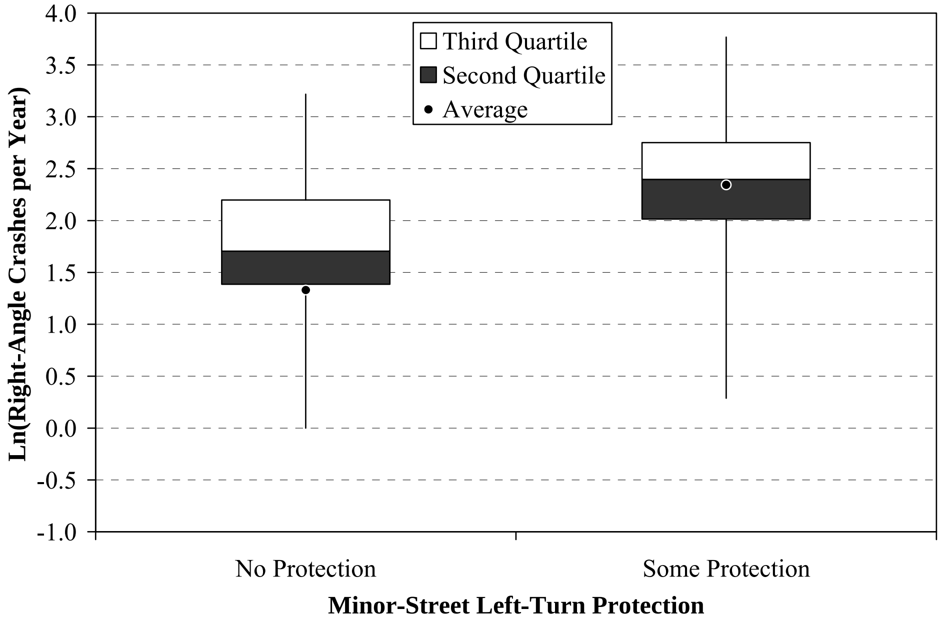

Figure L.2 Box plot of functional area right-angle crashes and minor-street left-turn protection.

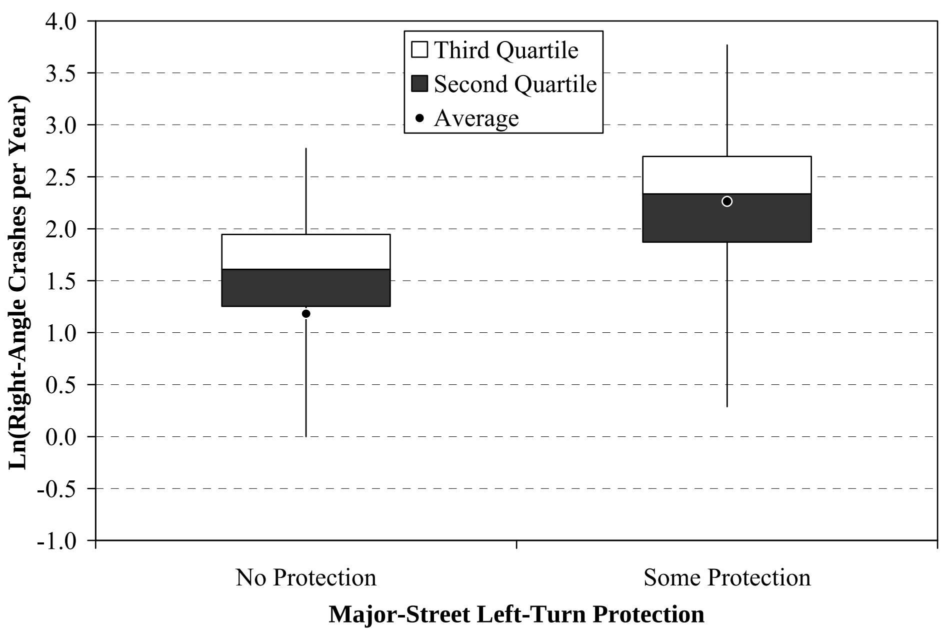

Figure L.3 Box plot of functional area right-angle crashes and major-street left-turn protection.

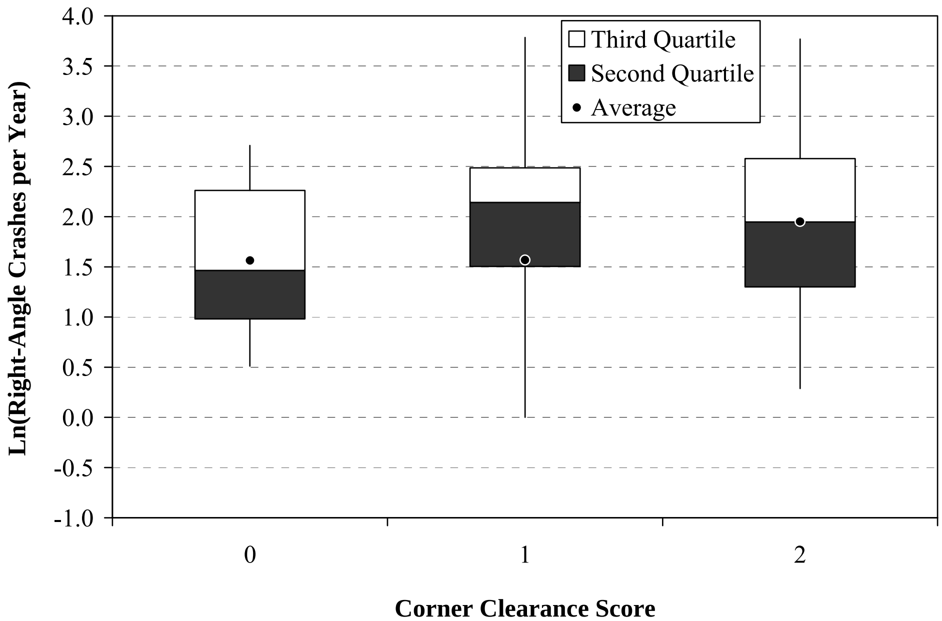

Figure L.4 Box plot of functional area right-angle crashes and corner clearance score.

Appendix M. Rear End Model Variable Relationships

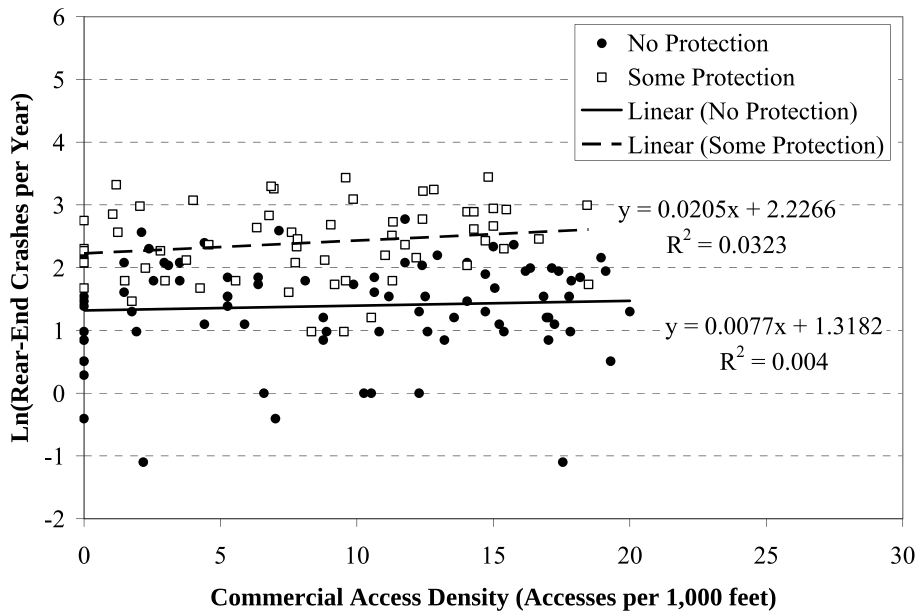

Figure M.1 Functional area rear-end crashes versus commercial access density by minor-street left-turn protection.

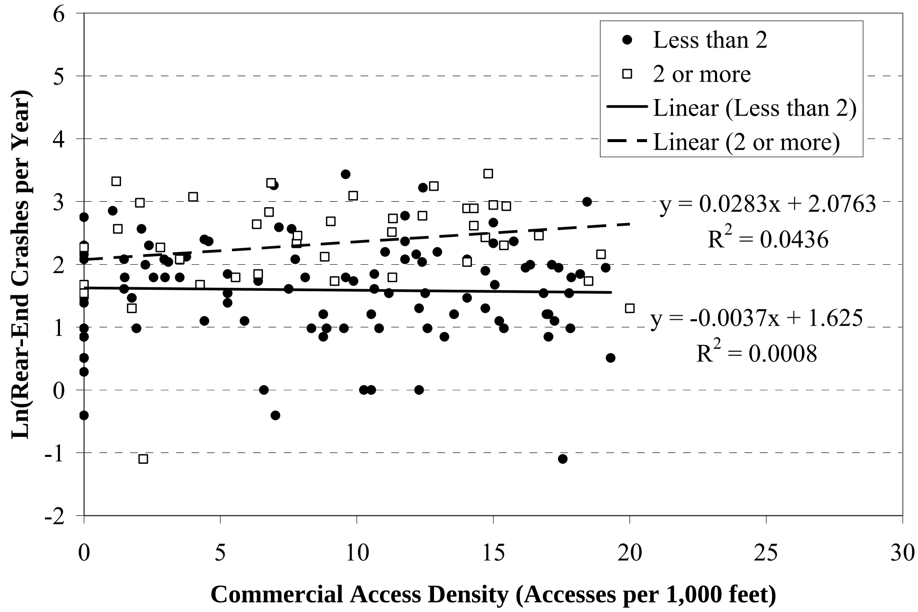

Figure M.2 Functional area rear-end crashes versus commercial access density by minor-street through lanes.

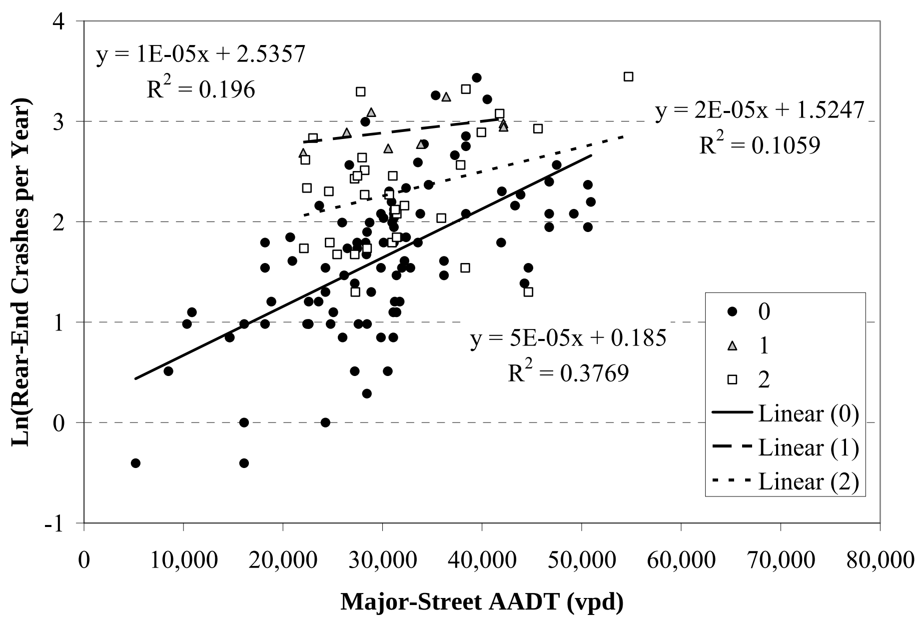

Figure M.3 Functional area rear-end crashes major-street AADT by median score.

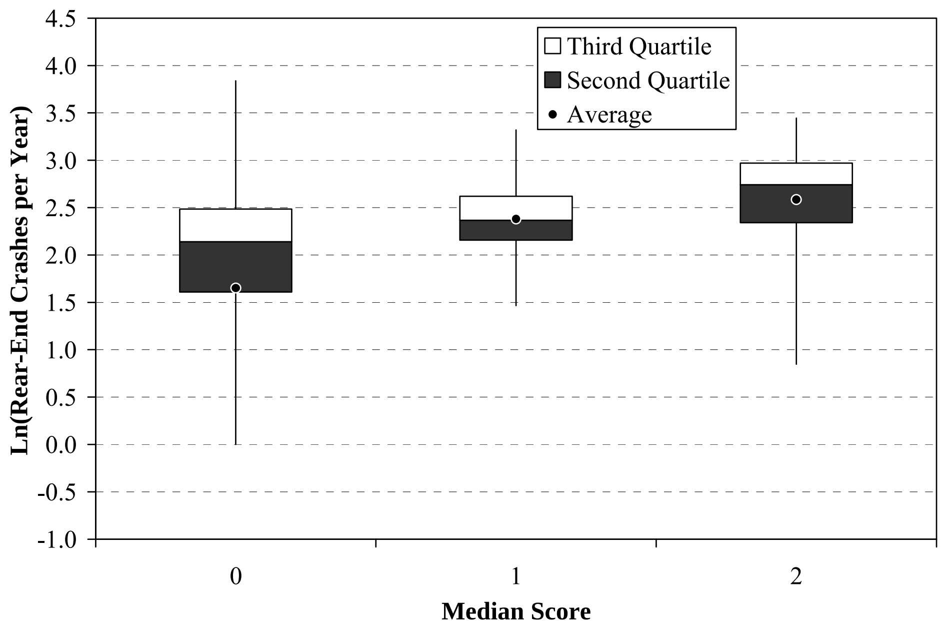

Figure M.4 Box plot of functional area rear-end crashes and median score.

Appendix N. Reference Data

Table N.1 Reference Site Locations

Study ID	Route Num	Major Street	Minor Street	Functional Class	Access Class
182	0152	Van Winkle Expswy	1300 East	Arterial	System Priority Urban
183	0152	Van Winkle Expswy	5600 South	Arterial	System Priority Urban
184	0152	Van Winkle Expswy	6100 South	Arterial	System Priority Urban
192	0154	Bangerter Hwy	9800 South	Arterial	System Priority Urban
193	0154	Bangerter Hwy	9000 South	Arterial	System Priority Urban
194	0154	Bangerter Hwy	7800 South	Arterial	System Priority Urban
195	0154	Bangerter Hwy	7000 South	Arterial	System Priority Urban
196	0154	Bangerter Hwy	6200 South	Arterial	System Priority Urban
197	0154	Bangerter Hwy	5400 South	Arterial	System Priority Urban
198	0154	Bangerter Hwy	4700 South	Arterial	System Priority Urban
199	0154	Bangerter Hwy	4100 South	Arterial	System Priority Urban
200	0154	Bangerter Hwy	3500 South	Arterial	System Priority Urban
201	0154	Bangerter Hwy	3100 South	Arterial	System Priority Urban
202	0154	Bangerter Hwy	Parkway Blvd	Arterial	System Priority Urban
203	0154	Bangerter Hwy	2400 South	Arterial	System Priority Urban

Table N.2 Reference Site Characteristics

Study ID	UDOT Region	Speed Limit	Major-Street Left Turn	Minor-Street Left Turn	Median	Analysis Years
182	2	50	Permitted	Protected-permitted	Barrier	02-04
183	2	50	Protected	Permitted	Barrier	02-04
184	2	50	Protected-permitted	Protected	Barrier	02-04
192	2	60	Protected	Protected-permitted	Barrier	03-05
193	2	60	Protected	Protected-permitted	Barrier	03-05
194	2	55	Protected	Protected	Barrier	03-05
195	2	55	Protected	Protected	Barrier	03-05
196	2	55	Protected	Protected	Barrier	03-05
197	2	50	Protected	Protected	Barrier	03-05
198	2	50	Protected	Protected	Barrier	03-05
199	2	50	Protected	Protected	Barrier	03-05
200	2	50	Protected	Protected	Barrier	03-05
201	2	50	Protected	Protected-permitted	Barrier	03-05
202	2	50	Protected	Protected-permitted	Barrier	03-05
203	2	50	Protected	Protected-permitted	Barrier	03-05

Table N.3 Reference Site Road Configurations

Study ID	Major-Street AADT	Minor-Street AADT	Average Major-Street Through Lanes	Average Minor-Street Through Lanes	Freeway Adjacent
182	20,223	17,696	2	1	0
183	22,611	9,259	2	1	0
184	25,211	17,068	3	1	0
192	32,659	n/a	3	1	0
193	37,846	14,653	3	2	0
194	42,793	10,723	3	2	0
195	48,719	n/a	3	1	0
196	51,799	12,107	3	2	0
197	51,248	18,209	3	3	0
198	51,706	14,870	3	3	0
199	50,980	15,119	3	3	0
200	50,158	18,200	3	3	0
201	50,057	8,782	3	2	0
202	44,898	n/a	3	2	0
203	39,065	n/a	3	1	0

Table N.4 Reference Site Geometry

	Increasing Milepost Approach			Decreasing Milepost Approach		
Study ID	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length	Approach-Side Corner Clearance	Right-Turn Bay Striping Length	Left-Turn Bay Striping Length
182	3,730	270	430	3,970	250	400
183	620	900	400	3,670	180	350
184	4,670	200	270	550	n/a	220
192	4,070	630	520	5,210	590	570
193	5,210	580	540	7,780	380	380
194	7,780	410	410	5,190	400	420
195	5,190	410	360	5,200	390	450
196	5,200	400	370	3,540	440	440
197	5,350	400	310	5,170	150	280
198	5,170	130	360	5,180	170	310
199	5,180	220	430	5,190	200	430
200	5,190	170	450	2,560	170	450
201	2,560	130	270	2,780	540	310
202	2,780	200	350	2,590	220	250
203	2,590	210	260	1,620	140	260

Table N.5 Reference Site Functional Areas

Study ID	Increasing Milepost Approach Functional Distance	Increasing Milepost Approach Functional Distance	Physical Area	Functional Area	Functional Area Overlap
182	640	640	150	1,430	N
183	1,080	620	128	1,828	N
184	550	525	105	1,180	Y
192	880	860	120	1,860	N
193	855	680	144	1,679	N
194	695	700	122	1,517	N
195	695	715	100	1,510	N
196	690	710	125	1,525	N
197	615	555	173	1,343	N
198	595	570	160	1,325	N
199	630	630	140	1,400	N
200	640	640	150	1,430	N
201	550	800	130	1,480	N
202	590	540	172	1,302	N
203	545	545	138	1,228	N

Table N.6 Reference Site Crashes and Crash Severities

	Crash Severity						
Study ID	No Injury	Possible Injury	Bruises & Abrasions	Broken Bones or Bleeding Wounds	Fatal	Total Crashes	Severity Costs
182	34	14	2	3	0	53	3,253
183	21	7	6	1	0	35	1,651
184	41	11	6	3	0	61	3,477
192	42	17	2	1	1	63	2,629
193	69	28	8	2	0	107	3,690
194	60	19	4	0	0	83	1,382
195	43	36	5	2	0	86	3,671
196	71	39	8	5	0	123	6,515
197	98	28	7	4	0	137	5,307
198	85	40	7	9	0	141	9,679
199	61	35	9	4	0	109	5,598
200	63	25	12	5	0	105	6,212
201	34	19	10	5	1	69	6,458
202	19	11	3	5	1	39	5,496
203	24	10	2	2	0	38	2,256

Table N.7 Reference Site Crash Types and Crash Rates

	Crash Type						
Study ID	Right Angle	Rear End	Side Swipe	Single Vehicle	Head On	Other	Crash Rate
182	11	11	2	5	0	3	2.39
183	9	7	0	8	0	3	1.41
184	25	8	4	0	0	5	2.21
192	13	35	5	6	0	4	1.76
193	8	41	7	5	3	0	2.58
194	10	29	3	4	0	4	1.77
195	7	65	7	4	1	2	1.61
196	9	67	8	4	0	2	2.17
197	11	87	15	9	0	5	2.44
198	11	50	9	9	0	4	2.49
199	14	50	7	6	1	5	1.95
200	6	36	12	8	0	5	1.91
201	8	15	5	4	0	0	1.26
202	2	22	3	3	1	1	0.79
203	7	23	5	1	0	2	0.89