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Report No. UT-24.19

PREDICTING LANE UTILIZATION AT SIGNALIZED INTERSECTIONS IN ADVANCE OF ARTERIAL LANE DROPS

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16. Abstract

Reducing the number of through travel lanes requires drivers to decide where and when they will merge from the lane that is being dropped (the short lane) to the adjacent through lane. When a large proportion of drivers choose to merge significantly upstream, this can result in poor lane utilization at an upstream signal which, in turn, leads to underused roadway capacity and reduced signal timing effectiveness. This research builds upon research conducted previously for the Utah Department of Transportation (UDOT) in 2020-2021 to determine the relationship between lane utilization and the length of the short lane on arterials. This research also assesses right-turn queue lengths on freeway on-ramps where a single right-turn lane merges with two left-turn lanes. The results of this research indicate that increasing the length of the short lane on arterials correlates to a moderate increase in utilization. The research also suggests that auxiliary through lanes do not provide the capacity benefit they are often anticipated to have, since the presence of an auxiliary through lane was correlated to a significant reduction in utilization. For lane drops at freeway on-ramps, the research indicated that increasing right-turn queues correlated linearly to increasing volume, particularly in the right-turn lane. Also, increasing the length of the right-turn lane either before or after the merge point correlated non-linearly to reduced right-turn queues. From this relationship, the apparent benefit of increasing right-turn length decreases as length increases until it becomes essentially negligible at about 1,500 feet.

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LIST OF ACRONYMS

ATL	Auxiliary Through Lane
ATSPM	Automated Traffic Signal Performance Measures
NCHRP	National Cooperative Highway Research Program
UDOT	Utah Department of Transportation

EXECUTIVE SUMMARY

The current standard practice in Utah is to fund and construct widening projects in which an analysis determines the widening is required to improve traffic operations on a corridor or at an intersection. Often these projects do not account for tying into existing conditions downstream of the intersection. It is common for roadway-widening projects statewide to make this transition by implementing a lane drop just beyond a signalized intersection.

Reducing the number of through travel lanes requires drivers to decide where and when they will merge from the lane that is being dropped to the adjacent through lane. Based on observational studies at project sites across Utah, the decision about where and when to merge varies greatly by drivers. While some drivers merge closer to the location of the lane drop, most drivers choose to merge upstream of the intersection. This results in poor lane utilization at the signal which, in turn, leads to underused roadway capacity and reduces the effectiveness of the widened roadway.

The research in this report is built upon research conducted previously for the Utah Department of Transportation (UDOT) in 2020-2021 to determine the relationship between lane utilization and the length of the lane between the intersection and the lane drop downstream on arterials. This research also assesses right-turn queue lengths on freeway on-ramps where a single right-turn lane merges with two left-turn lanes.

The results of this research indicate that increasing the length of the lane between the intersection and the lane drop on arterials correlates to a moderate increase in utilization. The research also suggests that auxiliary through lanes do not provide the capacity benefit they are often anticipated to have, since the presence of an auxiliary through lane was correlated to a significant reduction in utilization. For lane drops at freeway on-ramps, the research indicated that increasing right-turn queues correlated linearly to increasing volume, particularly in the right-turn lane. Also, increasing the length of the right-turn lane either before or after the ramp correlated non-linearly to reduced right-turn queues. From this relationship, the apparent benefit of increasing right-turn length decreases as length increases until it becomes essentially negligible at about 1,500 feet.

1.0 INTRODUCTION

1.1 Problem Statement

The current standard practice in Utah is to fund and construct widening projects in which an analysis determines the widening is required to improve traffic operations on a corridor or at an intersection. Often these projects do not account for tying into existing conditions downstream of the intersection. It is common for roadway-widening projects statewide to make this transition by implementing a lane drop just beyond a signalized intersection.

Reducing the number of through travel lanes requires drivers to decide where and when they will merge from the lane that is being dropped to the adjacent through lane. Based on observational studies at project sites across Utah, the decision about where and when to merge varies greatly by drivers. While some drivers merge closer to the location of the lane drop, most drivers choose to merge upstream of the intersection. This results in poor lane utilization at the signal which, in turn, leads to underused roadway capacity and reduces the effectiveness of the widened roadway.

Based on findings from the first study in 2020-2021, additional data points are needed to create a broader understanding of driver behavior. The first study found that lane utilization is directly affected by the level of congestion and by the length of the lane to be dropped just past the intersection. The results of the first study also indicated the need for additional data focusing on longer lane-drop locations; the previous locations studied, along with most similar lane drops in Utah, were found not to be long enough to encourage more efficient lane utilization on both sides of the intersection.

In addition to needing more data for arterial streets, the study team determined that lane-drops on freeway on-ramps should also be studied based on observed operational differences between on-ramps with lane drops of various merge lengths.

It is important to UDOT that projects designed to increase capacity successfully maximize the benefits for drivers. To maximize the benefits, it is critical to understand the effects that roadway design decisions have on lane utilization, as well as the associated unused capacity. For example, UDOT project managers often ask traffic engineers what the distance should be from the intersection to the lane drop; however, traffic engineers lack meaningful data upon which to base a recommendation. Thus, the study team determined that this length and other design decisions, such as the presence of an auxiliary through lane at the intersection, should be studied.

1.2 Objectives

The objective of this research project is to enable UDOT planners, traffic engineers, and design engineers to make informed decisions about implementing lane drops on arterial streets and freeway on-ramps and better understand the effect that lane drops have on roadway capacity. Data on the number of vehicles using each lane at signalized intersections upstream of lane-drop locations will be collected, analyzed, and compiled into graphs and tables so UDOT can make informed, data-driven decisions about implementing lane drops after intersections and better balance the tradeoffs between project costs and roadway capacity.

This is the second phase of a 2020-2021 research project to evaluate the effects that the distance between a signalized intersection and an arterial lane-drop location can have on lane utilization upstream of the intersection. At the end of the previous phase of this project, it was determined that the 25 sample lane-drop locations were not long enough when measured as the length between the intersection and the lane drop to determine an effective length of a lane drop. This phase would expand the samples to 41, to include longer lane drops in 16 additional locations. Further, this phase would include up to 25 freeway on-ramp locations for a separate analysis not performed in the first phase of the research.

The results will be used to update the existing equation and charts created during phase 1 that illustrate the expected lane utilization based on the upstream distance to the lane-drop location. Additionally, new equations and charts will be created specifically for lane drops on freeway on-ramps to illustrate the expected right-turn queues based on the merge length and any other related variables identified in the research efforts. The information in both charts and tables would enable UDOT project managers, roadway designers, and traffic engineers to understand the capacity effects of lane drops on arterial streets and freeway on-ramps, and the tradeoffs related to reducing lane-drop lengths to reduce costs. Having this understanding would enable UDOT to make more effective and informed decisions when considering implementation of lane drops on arterial streets and freeway on-ramps.

1.3 Scope

The goal of this research was to analyze driver behavior in lane drop scenarios and assess the potential benefits of longer road lengths before the lane drop on this behavior. The tasks for this project included conducting a literature review, data collection, data analysis, and documentation. The data were limited to locations in Utah which fit the following criteria: Arterial locations could not have large driveways along the short lane, and on-ramp locations needed to have two left-turn lanes and one channelized right-turn lane that met and merged on the ramp. An additional 19 arterial locations were selected along with the 25 locations from Phase 1 of this research. Up to 25 locations were selected for the on-ramp analysis.

1.4 Outline of Report

The body of this report is organized in the following manner:

- Literature Review: The literature review was conducted for existing published research, government reports, and other appropriate documentation related to lane drops.
- Data Collection: The process of collecting and cleaning data is described in this section. This included the process of counting vehicles from video data both manually and with video detection software as well as the calculation of useful metrics from the data.
- Data Evaluation: This section describes investigations into the data including histograms and scatter plots as well as the statistical analysis used to model the data.
- Conclusions: The results of this research are summarized in the conclusions section. The limitations and challenges of the research are described.
- Recommendations: Recommendations for implementation of this research and future research are described in this section.

2.0 LITERATURE REVIEW

2.1 Overview

Auxiliary through lanes (ATLs) are lanes that are added upstream of an intersection and then removed downstream of an intersection. Such lanes are typical solutions for increasing intersection capacity, but they can negatively impact upstream lane utilization (Nevers, et al., 2011). UDOT previously conducted a study on lane drops which included a review of existing literature on the subject. The purpose of this paper is to re-examine existing literature on lane utilizations at intersections preceding lane drops, providing a review of past findings and focusing on additional existing literature on the subject. Findings of this review can be utilized to better understand characteristics of lane drops with their benefits and potential detriments. The search for relevant research was conducted primarily using the Transport Research International Documentation database of the Transportation Research Board. Journal articles and reports at both the state and national level were included in the review.

2.2 Previous Findings

As noted above, UDOT previously identified literature existing on the topic of lane drops. The following section details these findings. This prior research can be used in conjunction with newer studies to provide a better understanding of details surrounding lane drops.

Tainter et al. (2018) used driving simulations to predict lane utilization at ATLs on an individual level. They suggest that signage can play a key role in an individual's decision of lane usage that may mitigate suboptimal lane utilizations. Bugg et al. (2012) studied intersection-related variables that impact lane utilization at intersections with an ATL and a continuous through lane. They collected the data at 8 intersections for 12 hours total, including: queue lengths in each lane, time to clear the intersection, arrivals on green, the average green time to cycle length ratio, vehicle type, and lane utilization. The authors indicate that, in addition to through volume and the ratio of green time to cycle length, utility of an ATL is a function of each driver's arrival time and the queue lengths at the time of arrival.

The National Cooperative Highway Research Program (NCHRP) report 707 (Nevers, et al., 2011) was commissioned to provide guidelines for and to assess the impacts of ATLs. The project included an analysis of 22 intersection approaches with ATLs including intersections with either one or two continuous through lanes and intersections with either dedicated right-turn lanes or shared ATL-right-turn lanes. The authors created a tool to predict utilization for ATLs and shared ATL-right-turn lanes based on effective green time, cycle length, volume (both through and right), saturation flow rate (both through and right), and intersection width. McCoy and Tobin (1982) found that auxiliary lane use is positively correlated with auxiliary lane length and negatively correlated with green time, and that right-turn volume does not impact lane choice for through vehicles if it is less than 25% of the through volume.

Ring and Sadek (2012) looked at both intersections with ATLs and intersections with lane drops. They collected data during the AM peak, PM peak, and lunchtime peak for 7 intersections (including 3 ATLs and 4 lane drops), including volumes for each lane and movement, heavy vehicle volume, and the distance that merging vehicles took to merge. They estimate a regression model predicting utilization based on through volume, right-turning volume, median two-way left-turn lane presence, upstream right-side trip generation density, and downstream right-side trip generation.

Lee et al. (2005a and 2005b) studied intersections immediately upstream of arterial lane drops, either from forced merges or conversion to a turn-only lane, or where multiple left-turn lanes fed into a roadway that dropped one of those lanes. They studied 94 sites in North Carolina for 3 hours each. The primary variables for prediction were the length of the short lane (the lane which will be dropped) and traffic volume. Additional data collected included: taper lengths (or distance to first pavement indication of a required turn), storage lengths for turning vehicles, distance to next intersection, merge-related signs and locations, pavement markings, driveway number and activity level, presence of two-way left-turn lanes, left-turn lane length (when applicable), speed limits, land use, lane width, grade, volume per cycle, saturation headway, queue lengths and delay, splits for each phase (green, yellow, all-red, and red) for each cycle, heavy vehicle percentage and lane utilization. They separately estimate regression models (considering transformations as well) for each of six different intersection types: two through lanes to one through lane with exclusive right-turn lane, two through lanes to one through lane with shared through and right-turn lane, two left-turn lanes to one lane at intersections, two left-turn lanes to one lane onto ramps, three through lanes to two through lanes with exclusive right-turn lane, and three through lanes to two through lanes with shared through and right-turn lanes. Variables that were significant in at least one of the models include short lane length, average lane volume, number of merge-related signs, taper length, right-turn volume (for shared lanes) and heavy vehicle percentages. This project heavily influenced design standards in North Carolina in the years since the publishing of the final report. One of the authors, now employed by the North Carolina Department of Transportation indicated that the results from the project are still regularly used today in design standards for lane drops, with the department using a general rule of thumb of keeping short lane distances above a quarter mile (Hummer, 2020).

The literature that was reviewed highlighted several different variables for collection and analysis. The most important variables were:

- Length of short lane
- Volume by lane
- Right-turning volume
- Driveway information

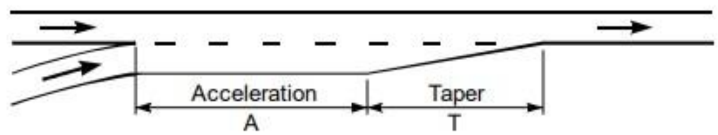
The literature also identified several variables that could be important in predicting lane utilization. These variables include:

- Saturation flow rate
- Ratio of green time to cycle length
- Taper length
- Presence of two-way left-turn lane / left-turn storage length
- Heavy vehicle percentages
- Merge-related signage
- Speed limits
- Land use
- Distance to next intersection
- Intersection and lane widths
- Queue lengths

2.3 Additional Findings

Additional reports and other literature on the subject of lane drops have been published. These studies expand further on the topic of ATLs, lane drops, associated technologies, and benefits and/or detriments of ATLs. An initial review of existing research studies and reports reveals that there has not been a great amount of literature published on ATLs and their associated lane drops. A majority of literature published since the year 2020 (the date of the previous UDOT study) on ATLs and lane drops refers to lane drops on freeways and major arterials, which was not the scope of this study by UDOT. Based on this situation, it appears that a gap in research and literature on ATLs/lane drops preceding and otherwise in proximity to intersections exists. Despite a relative lack of new literature on ATLs and lane drops, several previous studies and guidelines on these roadway elements have been added to this literature review and are discussed below.

ATLs and lane drops may be included within state roadway design and application guides which can be used to make decisions on best designs for lane drops, such as weighing the possibility of creating lane drops directly after the intersection or a distance past the intersection. The Texas Department of Transportation (TxDOT) has previously included discussions of lane drops which occur after passing through an intersection in the TxDOT Urban Intersection Design Guide Manual (Fitzpatrick, et al., 2005). This resource discusses the issues of designing an effective lane drop where ATLs are dropped past an intersection. TxDOT recommends that the Roadway Design Manual regulations be used to develop appropriate taper lengths at a lane drop to effectively filter oncoming traffic. These regulations are highlighted in Figure 2-1. These requirements match existing regulations found in the Policy on Geometric Design of Highways and Streets (more commonly known as the ‘Green Book’) established by the American Association of State Highway and Transportation Officials (AASHTO).



Highway Design Speed (mph)	Minimum Length of Taper T (ft)	Acceleration Length, A (ft) for Entrance Curve Design Speed (mph).								
		Stop Condition	15	20	25	30	35	40	45	50
		AND INITIAL SPEED (mph)								
		0	14	18	22	26	30	36	40	44
30	150	180	140	–	–	–	–	–	–	–
35	165	280	220	160	–	–	–	–	–	–
40	180	360	300	270	210	120	–	–	–	–
45	200	560	490	440	380	280	160	–	–	–
50	**230**	**720**	**660**	**610**	**550**	**450**	**350**	**130**	**–**	**–**
55	250	960	900	810	780	670	550	320	150	–
60	265	1200	1140	1100	1020	910	800	550	420	180
65	285	1410	1350	1310	1220	1120	1000	770	600	370
70	300	1620	1560	1520	1420	1350	1230	1000	820	580
75	330	1790	1730	1630	1580	1510	1420	1160	1040	780

Note:
Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1300 ft.
Lengths of Right-Turn Acceleration Lanes (US Customary).

Figure 2-1 Length of right-turn acceleration lanes in TxDOT Roadway Design Manual.

TxDOT further discussed the development of lane drops in roadway construction settings. The agency concludes that if an overall corridor is being provided with an increased number of ATLs, but the construction is in segments, it may be more efficient to construct the end intersections using the “final” section and drop the additional lanes directly after the intersection, allowing future projects to avoid performing work directly in the intersection (Fitzpatrick, et al., 2005). In addition, TxDOT includes a lane drop diagram within the Urban Intersection Design Guide, including diagrams for lane drops at the intersection, and after the intersection (Fitzpatrick, et al., 2005b).

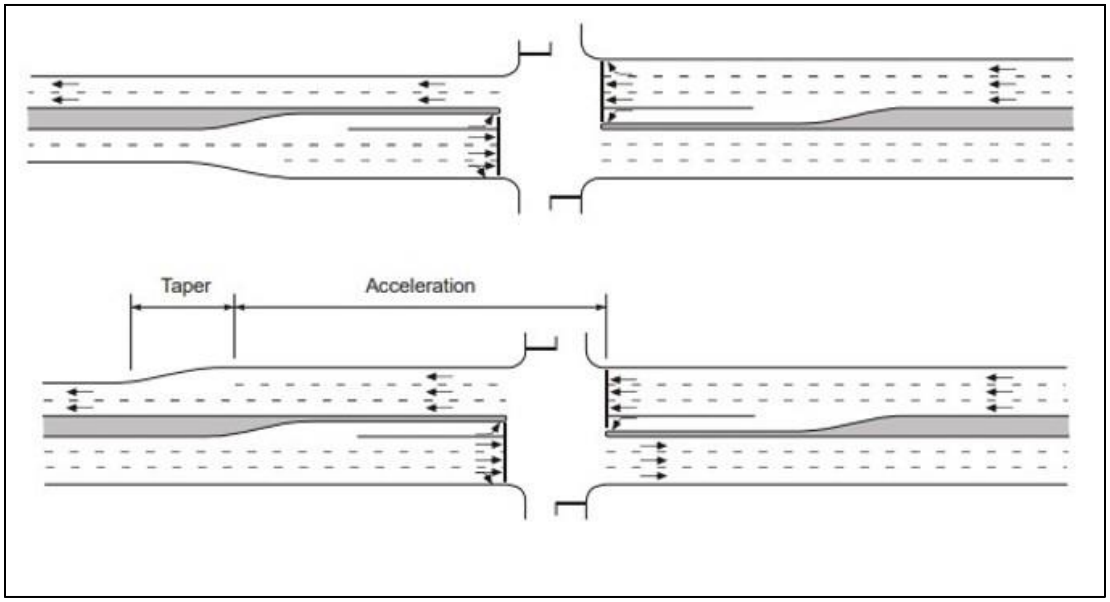


Figure 2-2 TxDOT lane drop diagrams (top: at intersection, bottom: after intersection) (Fitzpatrick, et al., 2005b).

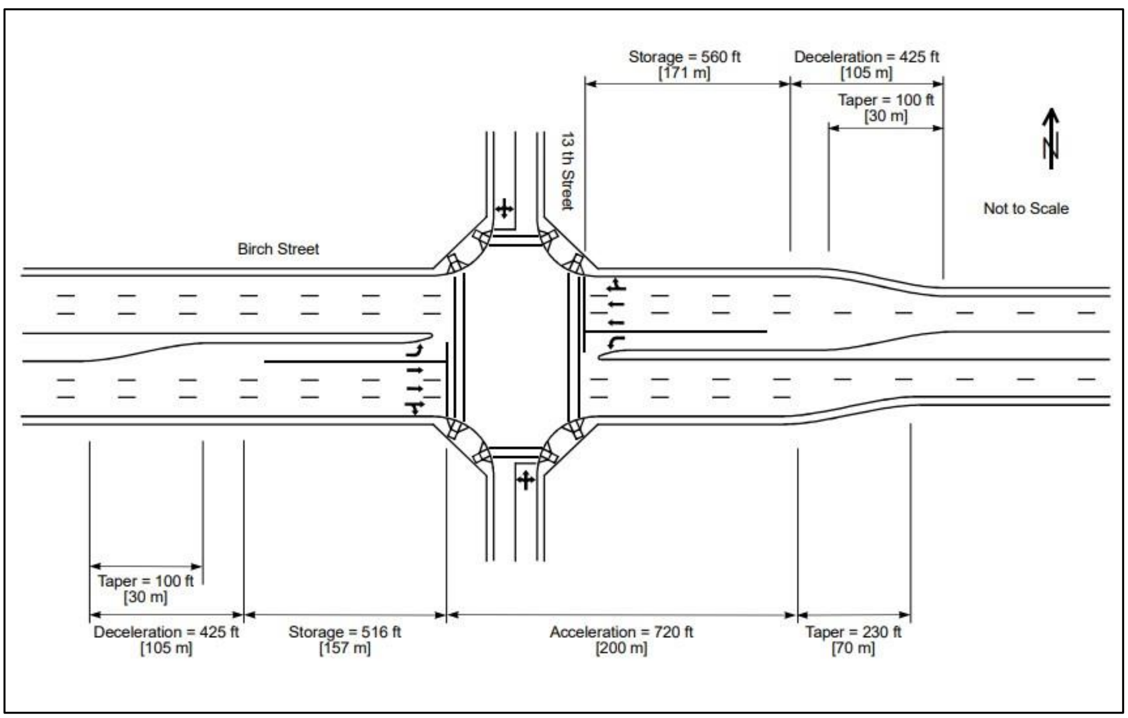
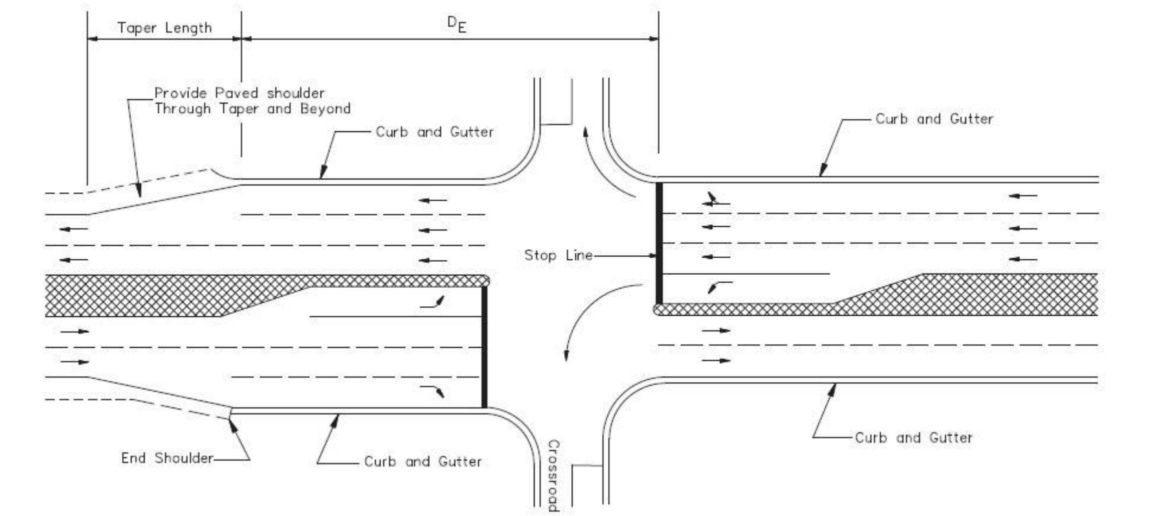


Figure 2-3 TxDOT lane drop proposed design (Fitzpatrick, et al., 2005b).

Another example of DOT ATL/Lane Drop design guidance can be found with the Illinois DOT, which provides an overall guide for the development of ATL/Lane Drops beyond or at an intersection, as seen in Figure 2-4 (IDOT, 2007). Illinois requires similar parameters to that in the TxDOT design guide for the tapering of through lanes based on design speed, which governs the required taper length (IDOT, 2007). This again highlights how DOTs will occasionally develop ATL/Lane Drop oversight for roadways, which are often based upon existing guidelines established by organizations such as the American Association of State Highway and Transportation Organizations (AASHTO).



US Customary			Metric		
Design Speed (mph))	D_E (ft)	Taper Length (ft)	Design Speed (km/h)	D_E (m)	Taper Length (m)
30	160	540	50	40	162
35	215	540	60	70	162
40	320	540	70	110	162
45	430	540	80	160	180
50	585	600	90	235	180
55	780	600	100	325	180
60	1010	600	110	455	180
65	1300	600			
70	1750	600			

Notes:

1. *D_E is that distance required by a vehicle to accelerate from a stop to 5 mph (10 km/hr) below the design speed of the highway.*
2. *The taper distance is calculated assuming a 12 ft (3.6 m) lane and a taper rate of 45:1 for design speeds of 45 mph (70 km/hr) or less or 50:1 for design speeds of 50 mph (80 km/hr) or greater.*

Figure 2-4 Illinois DOT extension of through lane beyond an intersection.

When an ATL and corresponding lane drop is present near an intersection, appropriate signing may contribute to reduced failures and other issues associated with the lane drop. The Federal Highway Administration (FHWA) has published an evaluation on late merge signing associated with ATLs and lane drops (FHWA, 2023). The study utilized a virtual laboratory study with a follow-up field study to evaluate sign types that were most effective for merging at a lane drop. Measuring right-lane utilization (RLU), they looked for an improved value that indicated that the lane drop and merge point are driving efficient operation. Results found that the signs generally increased RLU at both intersections and the lane drop merge point in the long term (but not short term), with variation based on specific sites. Additional findings show that RLU decreased when pavement arrow markings were present, and increased when driveways and businesses were present on the right side of the road (FHWA, 2023). These findings indicate that appropriate signage may improve the efficiency of lane drop use at intersections with ATLs. See Figure 2-5 below for a sample of sign designs the study utilized for field testing.

Figure 2-5 FHWA lane-drop merge signs used for field testing.

As noted previously, there is not a significant amount of new information on the subject of lane drops. Outside of the studies and resources described previously, lane drops have been studied within the context of ITS and driver assistance systems (Van Driel, 2010). Methods and strategies for developing smoother transitions in and around lane drops during temporary conditions have also been researched (Enterprise Program, 1997).

2.4 Summary

The studies previously reviewed by UDOT and the studies newly included in the review process provide a framework for data collection and expectations for this study. As noted, there has not been significant literature released in recent years regarding ATL/lane drops and intersection areas. However, the studies reviewed still provide useful information about lane drops. The differing scopes of these studies make it clear that the classification and differentiation of intersection and lane drop type is critical to adequately predict lane utilization. The differences seen between varying intersection types and the lane drops associated with them reinforce the need for collecting quality data on intersection ATL/lane drop scenarios to ensure these facilities function effectively. Other factors such as signing will also impact the efficiency of lane drop operations. Recent studies have found that appropriate signage at lane drop areas can improve performance and efficiency, lessening traffic issues stemming from the lane drops themselves (FHWA, 2023).

3.0 DATA COLLECTION

3.1 Overview

The scope of this research includes the study of 44 arterial locations and up to 25 freeway on-ramp locations in Utah. This chapter discusses the data collection process for arterial locations and ramp locations separately. First, there will be a discussion on location selection for arterials and ramps. Following is a description of the roadway data collection process. Then, there will be an explanation of the vehicle data collection.

3.2 Arterial Data Collection

Some of the arterial locations were observed in the previous phase of this research. Arterial locations fit for this research were multi-lane roadways that dropped the right-most through lane at some distance downstream of a signalized through movement. Lane drop locations with trap lanes, through/right lanes, and/or ATLs were allowed, and the presence of these configurations (if any) were noted for each location. Diagrams of these three special configurations are shown in Figure 3-1.

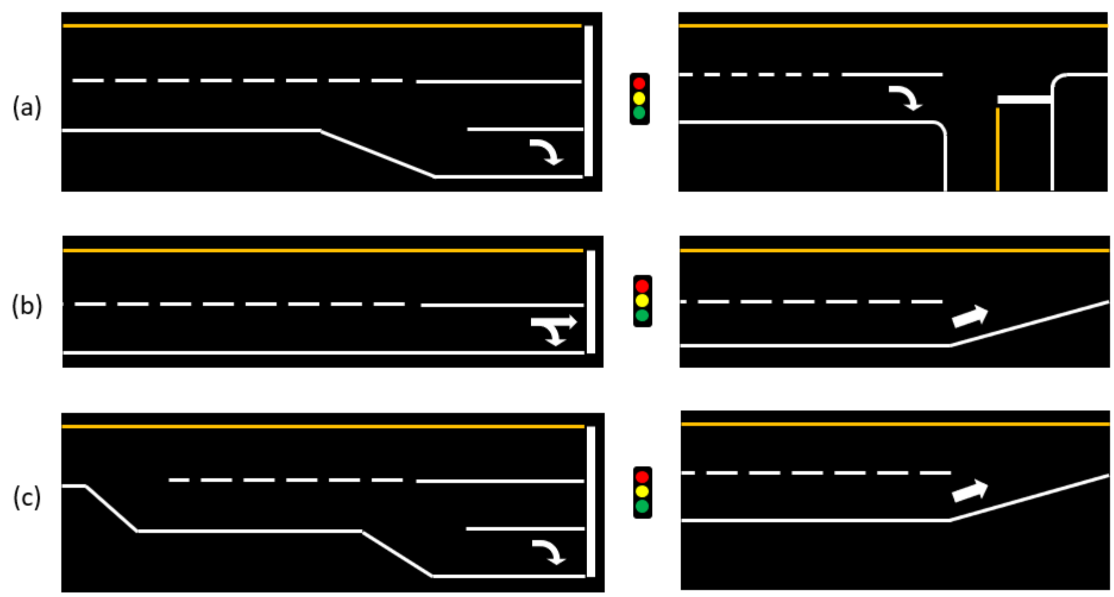


Figure 3-1 Special lane drop configurations: (a) trap lane, (b) through/right lane, (c) ATL.

The following sections explain data collected at the locations used in this research.

3.2.1 Arterial Location Selection

Locations with large driveways between the signalized intersection and the lane drop location were not selected to avoid having cases where the presence of a driveway strongly influences the upstream lane utilization. Similarly, trap lane locations were not selected if the turn lane was expected to have significant volumes during the data collection period. The distance from the upstream signal to the lane drop location was also considered, as it was desired to have a variety of lengths present in the dataset.

A total of 44 arterial locations in Utah were selected; 25 of these locations were used in a previous, related research project while the other 19 of them are introduced in this research project. The previous and new locations are listed in Table 3-1 and Table 3-2 respectively alongside their lane count upstream of the signalized intersection, speed limit, and length from signalized intersection to lane drop location measured by the striped length (measured starting immediately downstream of the signalized intersection) and taper length. A discussion on how the roadway data (speed limit and lengths) were obtained for the new 19 locations is given in the following section.

Table 3-1 Arterial Locations from Previous Research

ID	*Description*	*Direction*	*UDOT Region*	*City*	*Lanes*	*Speed Limit (MPH)*	*Striped Length (ft)*	*Taper Length (ft)*
1	12th St & Wall Ave	NB	1	Ogden	3	40	170	610
2	12th St & Wall Ave	SB	1	Ogden	3	40	230	270
3	12th St & Wall Ave	EB	1	Ogden	3	40	70	430
4	12th St & Wall Ave	WB	1	Ogden	3	40	80	100
5	5600 S & 3500 W	NB	1	Roy	2	45	170	270
6	5600 S & 3500 W	SB	1	Roy	2	45	280	160
7	5600 S & 3500 W	WB	1	Roy	2	40	920	220
8	U.S. 89 & Skyline Dr	SB	1	South Ogden	3	55	90	180
9	S.R. 39 & S.R. 126	NB	1	West Haven	2	50	770	540
10	S.R. 36 & Saddleback Blvd	SB	2	Tooele County	3	60	2,100	unknown
11	1000 N & Redwood Rd	NB	2	Salt Lake City	2	45	470	290
12	1300 S & State St	EB	2	Salt Lake City	2	30	90	290
13	2300 E & Foothill Dr	EB	2	Salt Lake City	3	40	240	360
14	2100 S & 1300 E	NB	2	Salt Lake City	2	35	170	120
15	S.R. 111 & 7800 S	NB	2	West Jordan	2	50	370	450
16	8000 S & State St	SB	2	Midvale	3	40	100	280
17	9000 S & Redwood Rd	EB	2	West Jordan	3	40	260	170
18	9000 S & Redwood Rd	WB	2	West Jordan	3	40	280	160
19	9000 S & 700 W	WB	2	Sandy	3	40	180	400
20	9000 S & State St	NB	2	Sandy	3	40	200	110
21	S.R. 92 & 4800 W	EB	3	Highland	2	45	690	350
22	Main St & State St	EB	3	American Fork	3	35	140	210
23	1600 N & State St	WB	3	Orem	2	40	330	260
24	Center St & 1200 W	EB	3	Orem	3	35	100	320
25	University Pkwy & Geneva Rd	SB	3	Orem	2	45	260	230

Note: NB = Northbound, EB = Eastbound, SB = Southbound, WB = Westbound

Table 3-2 New Arterial Locations

ID	Description	Direction	UDOT Region	City	Lanes	Speed Limit (MPH)	Striped Length (ft)	Taper Length (ft)
26	U.S. 189 & 1300 South	EB	3	Heber	2	30	250	160
27	U.S. 189 & U.S. 40	SB	3	Heber	2	40	340	410
28	S.R. 224 & S.R. 248	EB (SB)	2	Park City	2	35	1,380	0
29	S.R. 248 & Monitor Dr	NB	2	Park City	2	35	25	340
30	S.R. 248 & Richardson Flat Rd	SB	2	Park City	2	50	230	1,340
31	S.R. 248 & U.S. 189 NB Ramps	EB	2	Park City	2	45	380	610
32	U.S. 40 & S.R. 45	SB	3	Naples (Vernal)	2	45	800	430
33	S.R. 108 & Hinckley Dr	NB	1	West Haven	2	50	320	740
34	S.R. 252 & 1400 North	NB	1	Logan	2	50	360	1,050
35	S.R. 171 & S.R. 111	NB	2	Magna	2	35	120	250
36	S.R. 201 & S.R. 202	EB	2	Magna	3	60	2,200	350
37	West Temple St & 900 South	SB	2	Salt Lake City	3	30	490	500
38	4100 South & 4000 West	WB	2	West Valley City	3	40	330	980
39	S.R. 71 & S.R. 266	SB	2	Millcreek	4	45	830	380
40	S.R. 209 & 2300 East	SEB	2	Sandy	2	40	525	870
41	S.R. 71 & Lone Peak Pkwy	WB	2	Draper	3	40	215	345
42	900 East & Temple View Dr	NB	3	Provo	2	35	450	400
43	U.S. 89 & 1600 North	SB	3	Mapleton	2	50	380	320
44	U.S. 6 & 200 West	WB	4	Delta	2	30	1,230	115

Note: NB = Northbound, EB = Eastbound, SB = Southbound, WB = Westbound

3.2.2 Roadway Data Collection for Arterials

Two of the lane drop characteristics needed for this research were approach speed limit and distance from the upstream intersection. These two characteristics were helpful in the data analysis process as will be described in further detail in Chapter 4.

The speed limit for each study location was determined from posted speed limit signs as found in the street view feature of Google Maps (Google, 2021a). As shown in Figure 3-2, the most common approach speed limit is 40 MPH, with most of the locations having a speed limit between 30 MPH and 50 MPH. The speed limits of the 25 locations taken from the previous research are shown in blue, while those of the new 19 locations are shown in orange.

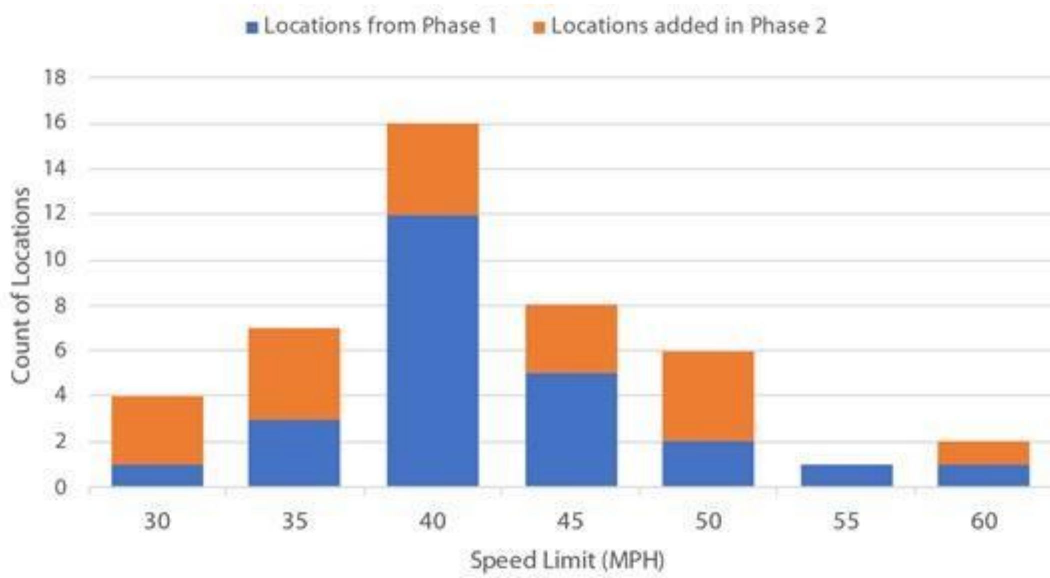


Figure 3-2 Distribution of speed limits.

As shown in Figure 3-3, the striped length was measured as the distance between the downstream edge of the intersection and the end of lane striping, and the taper length was measured as the distance between the end of lane striping and the point where the lane width returns to that of a single lane. In the case of a trap lane scenario, the striped length ended when the lane striping changed from dashed to dotted, and the taper was measured as the length of the dashed lane striping (ending where the lane markings turned to a solid line). The lengths were measured using Google Maps (Google, 2021b) and rounded to the nearest 5 feet. As shown in Figure 3-4, only five of the 19 new locations (shown in orange) have striping 300 feet or less in length, and three of the 19 new locations have striping longer than 1,000 feet. The overall distribution is still stacked heavily toward shorter lengths due to the high number of the 25 locations taken from the previous research (shown in blue) that measured 300 feet or shorter.

Figure 3-3 Example of length measurements.

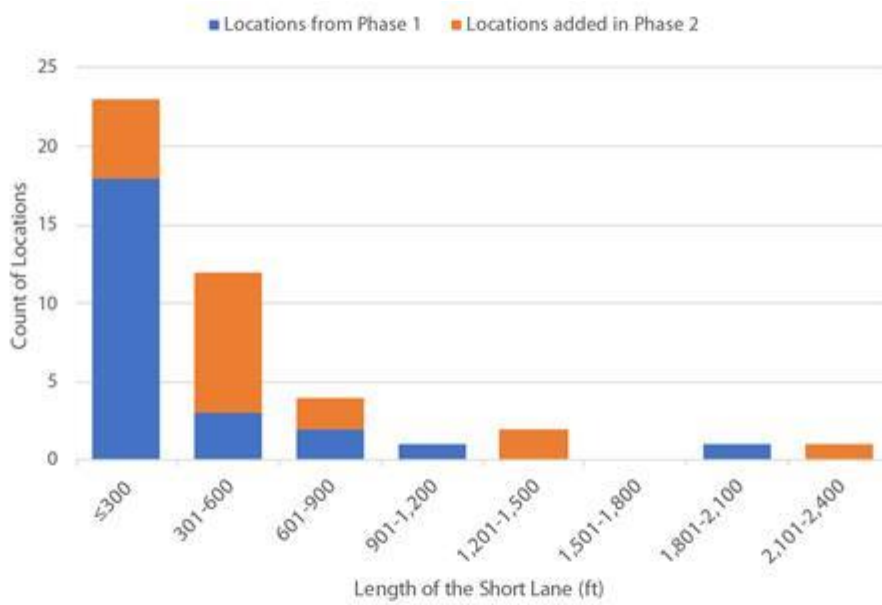


Figure 3-4 Distribution of short lane lengths (striped lengths).

3.2.3 Vehicle Data Collection for Arterials

Data for the 25 locations taken from the previous research had already been collected in a format ready to use in the analysis. The following discussion of data collection is specifically for the 19 new locations. The new method follows that which was done for the 25 locations from the previous research, but readers wanting to see the nuances between the two data collection efforts can refer to the previous research: UDOT research report UT-22.01.

The vehicles approaching the intersection upstream of the lane drop locations were video recorded. All approaches were recorded from 4 PM to 6 PM on a mid-October weekday without inclement weather. The PM peak period (approximated as 4 PM to 6 PM for the purposes of this research) was used because it offered the highest traffic volumes of the day. Where available, video recordings of the approaches were obtained from the UDOT Traffic Operations Center. In all other cases, video footage was recorded independently.

The vehicle lane use data was collected from the video recordings using one of two methods. The first method was done via manual counting. The data collection personnel would watch the video and use a custom Excel tool to tally the number of cars in each lane for each signal cycle. If a shared through/right lane was present, the right-turning vehicles were also tallied, but they were counted as a separate group from the through vehicles in their same lane. At the end of the tallying, each cycle was assigned a 15-minute bin within a two-hour window based on when the cycle began. In a few cases, the starting time of the cycles were not recorded, so they were instead estimated based on the average cycle length, calculated by the ratio between exact number of minutes in the video and number of cycles observed.

The second method of collecting the vehicle lane use data from the video recordings was via a deep learning model trained by the researchers. The angle of the vehicles relative to the camera view was a limiting factor in the accuracy of the model, as was the video quality and the presence of any large obstructions near the stop bar in the camera view. For poor quality videos, videos with visual obstructions, and videos with a relative vehicle angle for which the model was not well trained, the first method (manual counting) was used. In the end, only four videos were counted with the deep learning model.

For this second method, the researchers first converted videos from whatever format in which they were received to MP4. Then the researcher input “tag” and “count” polygons into the model – one of each for every through lane to be analyzed, plus one more of each if the outer through lane was shared with the right-turn movement. The zones were numbered, and the code of which zones corresponded to which lanes was recorded. More details can be found on this deep learning model in the appendix. The output of this deep learning model method was a spreadsheet that included the following for each detected and counted vehicle: video timestamp (in seconds since the start of the video), tag zone, and count zone. The output of the deep learning model was reviewed by the researchers and the following weaknesses were observed:

- Pickup-trucks pulling loads were often double counted
- Taller vehicles such as single unit trucks or some SUVs were sometimes counted in a count zone adjacent to the lane in which they were actually driving
- The model did not recognize, tag, or count any semi-trucks

The research group developed a clean-up method to delete duplicate counts and correct the count zone to match the tag zone for though vehicles using a macro in Excel. The count of semi-trucks, however, could not be rectified.

After the deep-learning model output was obtained and ran through the clean-up method, the gap between each vehicle was calculated by subtracting the timestamp of the leading vehicle from the timestamp of the following vehicle. Additionally, each counted vehicle was assigned a 15-minute bin within a two-hour window based on their timestamp.

3.3 Freeway On-Ramp Data Collection

Freeway on-ramp locations were considered for data collection if they included two left-turn lanes and one right-turn lane entering the ramp where the right-turn lane merges with the adjacent left-turn lanes downstream of the ramp entrance. The following sections explain how data were collected for these locations.

3.3.1 Ramp Location Selection

Freeway on-ramp location selection was simpler than arterial location selection. Study locations were selected to include a relatively even distribution of drop lane lengths and a random distribution of traffic volumes. Each location consisted of two left-turn lanes and one right-turn lane entering a freeway on-ramp where the right-turn lane is dropped and merges with the nearest left-turn lane.

A total of 24 ramp locations in Utah were selected. While the scope called for up to 25 ramp locations, one of the original locations, the southbound on-ramp at 1200 S (Marriot-Slaterville) and S.R. 39, was the only location with a non-channelized right-turn lane, so it was removed to maintain data consistency. Care was taken while selecting locations that the distribution of merge lengths was relatively uniform so the data would be representative of the various merge lengths found in Utah. This was done by selecting a random sample of locations and removing locations from the list until the distribution was more uniform. Section 3.3.2 explains how these merge lengths were measured and includes a distribution showing the uniformity of the data. Another criterion for these locations was that they had enough traffic volume to collect vehicle data from. Traffic volumes were estimated as described in Section 3.3.3, and all the locations were found to have sufficient volumes.

Ramp locations are listed in Table 3-3 alongside an indication of whether the lane dropped at the freeway meter and length from where the right- and left-turn lanes started to run adjacently to the lane drop location measured by the striped length and taper length. A discussion on how the roadway data were obtained is given in the following section.

Table 3-3 Selected Ramp Locations

ID	Description	Direction	UDOT Region	City	Drop at Meter?	Striped Length (ft)	Taper Length (ft)	Merge Length (ft)
1	I-15 & 400 South	SB	3	Springville	Yes	370	280	650
2	I-15 & Center St	SB	3	Orem	No	0	300	300
3	I-15 & 1600 North	SB	3	Orem	Yes	970	270	1240
4	I-15 & Main St	NB	3	Lehi	No	0	70	70
5	I-15 & Main St	SB	3	Lehi	No	20	160	180
6	I-15 & Bangerter Hwy	SB	2	Draper	Yes	740	260	1000
7	I-15 & 11400 South	NB	2	Sandy	Yes	790	230	1020
8	I-15 & 11400 South	SB	2	Sandy	Yes	840	240	1080
9	I-15 & 9000 South	SB	2	Sandy	Yes	840	310	1150
10	I-15 & 7200 South	NB	2	Midvale	No	0	70	70
11	I-15 & 5300 South	SB	2	Murray	No	50	590	640
12	I-15 & 3300 South	NB	2	South Salt Lake	No	170	275	445
13	I-15 & 3300 South	SB	2	South Salt Lake	Yes	990	220	1210
14	I-15 & 600 North	SB	2	Salt Lake City	No	0	0	0
15	I-15 & Layton Pkwy	SB	1	Layton	Yes	230	270	500
16	I-15 & Hill Field Rd	NB	1	Layton	Yes	390	170	560
*17**	I-15 & 1200 South	SB	1	Marriot-Slaterville	N/A	440	710	1150
18	Bangerter & 600 West	EB	2	Draper	N/A	410	370	780
19	Bangerter & 11400 South	NB	2	South Jordan	N/A	90	300	390
20	Bangerter & 7800 South	NB	2	West Jordan	N/A	0	50	50
21	Bangerter & 7000 South	NB	2	West Jordan	N/A	230	300	530
22	Bangerter & 6200 South	NB	2	West Jordan	N/A	450	320	770
23	Bangerter & 6200 South	SB	2	West Jordan	N/A	560	320	880
24	I-80 & 700 East	EB	2	Salt Lake City	N/A	270	400	670
25	I-215 & Big Cottonwood Rd	WB	2	Cottonwood Heights	N/A	20	380	400

Note: NB = Northbound, EB = Eastbound, SB = Southbound, WB = Westbound.

* Removed because it was the only location without a channelized right-turn lane.

3.3.2 Roadway Data Collection for Ramps

The lane drop characteristics needed for ramp locations were the distance for right-turning vehicles to merge into the adjacent lane and the distance these right-turning vehicles had available to queue up while waiting to merge, if necessary. These characteristics were helpful in the data analysis process as will be described in further detail in Chapter 4.

The merge length is made up of two portions – the striped length and the taper length. As shown in Figure 3-5, the striped length was measured as the distance between the point where the right-turn lane and left-turn lanes started running adjacently and the end of lane striping, and the taper length was measured as the distance between the end of lane striping and the point where the lane width returns to that of a single lane. The lengths were measured using Google Maps (Google, 2021b) and rounded to the nearest 10 feet. The merge lengths (striped length plus taper length) were shown previously in Table 3-3. As shown in Figure 3-6, The locations are rather evenly distributed with merge lengths, and the lengths range from 0 to about 1200 feet.

The advance length is measured from the start of the right-turn bay upstream of the ramp (not including the opening taper) to the ramp (starting point of the merge length). The lengths were measured using Google Maps (Google, 2021b) and rounded to the nearest 10 feet. The advance length plus the merge length is the “total length” referred to in this report when pertaining to the analysis of lane drops on freeway on-ramps.

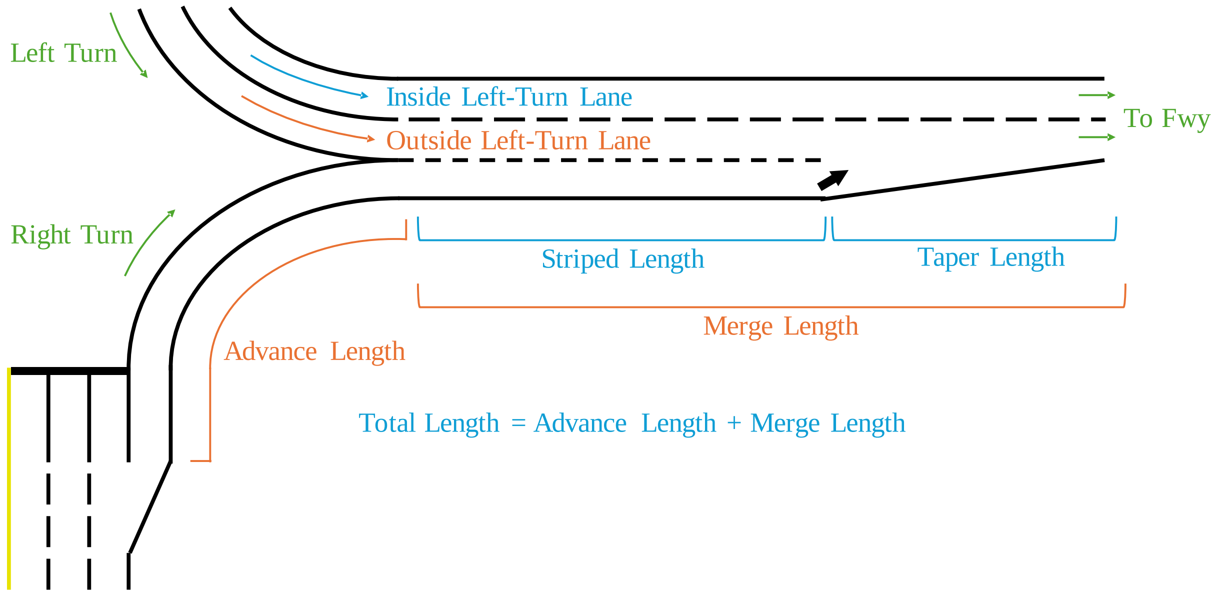


Figure 3-5 Example of length measurements for ramps.

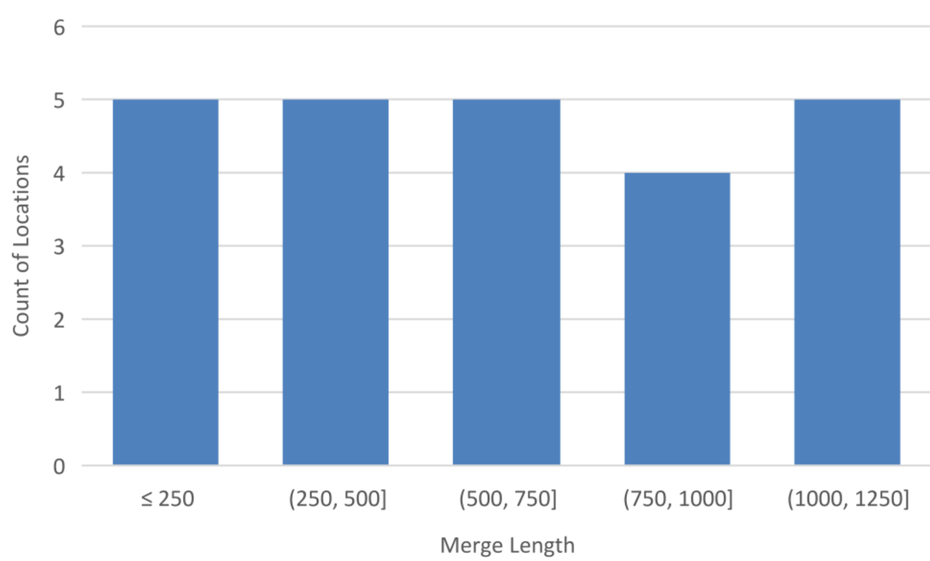


Figure 3-6 Distribution of merge lengths for ramps.

3.3.3 Vehicle Data Collection for Ramps

The vehicles approaching the ramp were video recorded. Approaches were recorded for 2 hours when there would be near-peak traffic, but not when ramp metering was active. Most were recorded on a weekday in February or March without inclement weather, but two (Lehi Main Street and I-15 southbound, and 3300 South and I-15 southbound) were recorded in May to replace locations with poor quality recordings. Ramp metering could not be active while the data were collected because this would create artificial queues in the right-turn lane. Where available, video recordings of the approaches were obtained from the UDOT Traffic Operations Center. In all other cases, video footage was recorded independently.

The vehicle lane use data was collected from the video recordings via manual counting. The data collection personnel watched the video and used a custom Excel tool to tally the number of cars in each lane during 5-minute periods. The maximum queue lengths in the right-turn lane were also recorded for each 5-minute period, and if the queue overflowed, this was also recorded.

3.4 Summary

Data collection included roadway characteristics and vehicular counts for a total of 44 arterial locations (25 of which were from a previous research project) and 24 freeway on-ramp locations. In the case of arterials, the speed limit, number of lanes, short lane and taper lengths, and per-lane volumes were collected. Most arterial locations had a speed limit of 30-50 MPH and a short lane between 0 and 600 feet long. In the case of freeway on-ramps, the merge length, advance length, and per-lane volumes were collected. The ramp locations had a near-uniform distribution of merge lengths.

4.0 DATA EVALUATION - ARTERIALS

4.1 Overview

This chapter discusses the analysis and evaluation of the collected data. First, is an explanation of how the data from the 19 new locations were manipulated to match the structure needed for further calculations. Next, is a description of the aggregations and calculations performed on that data. Following that, is a discussion on the observational analysis for the combined dataset of 44 locations, including explanations of trends observed in the data. Finally, a discussion on the statistical analysis is given which includes numerical and graphical representations of the selected statistical model.

4.2 Combining Datasets into Desired Structure

The data required the following structure prior to performing the analysis:

- Through volumes for each through lane (including through-right lanes, if present) needed to be presented per cycle; datasets needed one row for every cycle observed, with separate columns for each through lane.
- Each cycle needed to be assigned to a 15-minute period within a 2-hour span.
- Unique identifying information about the approach to which the volumes belonged needed to be included in each row.

The data collected manually met these criteria immediately, but the data collected by the deep learning model (which constituted four locations) did not since cycle number could not be determined by the deep learning model that was used. Instead, the data obtained from the deep learning model listed each vehicle detection on separate lines with no indication of which vehicles entered the intersection in the same cycle as each other.

Time gaps between vehicles detected by the deep learning model were calculated. ATSPM data was reviewed, and the average red time for the study approach was identified for the PM peak period on the same day as the approach was video recorded. For each approach, the average red time (rounded to the nearest five seconds) was set as a threshold – any time gaps between vehicles longer than the threshold were assumed to indicate the end of a signal cycle, i.e., a period when the signal indication for the study approach was red. Using these identified red periods, the average cycle length was calculated and compared to the average cycle length reported by ATSPM. The threshold was then adjusted up or down by five-second increments until the resultant average cycle length was as close to the cycle length reported by ATSPM as possible. The thresholds that produced the most accurate average cycle length are shown in Table 4-1. Once the red periods were identified using the thresholds in Table 4-1, the vehicles detected by the deep learning model were assigned cycle numbers.

Table 4-1 Minimum Gap Lengths Indicating a Cycle Break

ID	*Description*	*Direction*	*Minimum Gap Length Indicating a Cycle Break (sec)*
28	S.R. 224 & S.R. 248	EB (SB)	35
30	S.R. 248 & Richardson Flat Rd	SB	20
33	S.R. 108 & Hinckley Dr	NB	30
39	S.R. 71 & S.R. 266	SB	40

After all the vehicle detections were assigned to a cycle number, the number of through vehicles using each through lane were summed up by cycle and the timestamp of the first vehicle detection in each cycle was used to assign each cycle to a 15-minute period within a 2-hour span. These actions aligned the structure of the deep learning model results with the desired structure. The data from the locations counted manually and by the deep learning model were then combined into one dataset for further aggregation and calculation.

4.3 Aggregations and Calculations

The dependent variable of this research is the utilization rate of the short lane (shortened in this report to just “utilization rate”). The utilization rate is calculated by dividing the short lane volume by the average volume per lane as shown in Equation 4-1, where the average volume per lane is calculated by dividing the total approach volume (the sum of all the through vehicles regardless of lane) by the number of approach lanes.

$$Utilization\ Rate = \frac{Short\ Lane\ Volume}{Average\ Volume\ per\ Lane} \tag{4-1}$$

The utilization rate is equal to 1 when the short lane is used by one half of the approaching vehicles for two-lane approaches and by one third of the approaching vehicles for three-lane approaches. A utilization rate greater than 1 indicates that the short lane is favored over the continuous through lanes, and a utilization rate less than 1 indicates that the continuous through lanes are favored over the short lane.

As was mentioned previously, the collected data had been organized into cycles and each cycle was assigned a 15-minute period. The utilization rate was calculated twice – once for each cycle of each location and once for each 15-minute period of each location (the latter including multiple cycles per 15-minute period). To calculate the utilization rate for each 15-minute period of each location, the volumes for the cycles assigned to the same 15-minute period were summed up and used to calculate the short lane volume and average lane per volume as needed for Equation 4-1.

4.4 Observational Analysis

Trends in the data were analyzed to better understand which variables might be important to test in a statistical model. This section looks at both the cycle and 15-minute aggregations.

4.4.1 Cycle-Level Observations

For the cycle-level observations, the data was refined two steps further. First, the first and last cycle of the 2-hour data collection period for each approach were removed since they did not necessarily represent an entire green-to-red period. Second, any cycles that lasted 180 seconds or longer were removed because they likely represented two cycles as one, thus incorrectly showing an unusually high volume for the observed utilization rate; this step removed a total of 17.1 minutes from location #28 (S.R. 224 & S.R. 248).

Figure 4-1 shows the short lane utilization plotted against the approaching volume per cycle per lane. The per-lane volume is used because the locations studied included sites with differing numbers of through lanes (2 or 3, including the short lane). The distinct grouped curves in the chart come from the volume of the short lane: All points in the bottom curve had one vehicle in the short lane; all points in the second-to-bottom curve had two vehicles in the short lane, and so on. This figure shows that the utilization rate has a wide range (from 0 to 2) at low volumes but appears to converge to somewhere between 0.4 and 0.6 at higher volumes. One explanation for the relatively low utilization at higher volumes is that signals might be programmed to provide more green time to approaches with higher per-lane volume. As headways increase with long green intervals, there is less incentive to use the short lane.

There are two outliers in the dataset, one visible in the figure at approximately 25 vehicles per cycle and a utilization rate of 1.25. The other outlier extends beyond the limits of the figure and is approximately 90 vehicles per cycle and a utilization rate near 0.2. These two outliers are from different locations. Additionally, the points with a utilization rate of 0 come from a variety of locations.

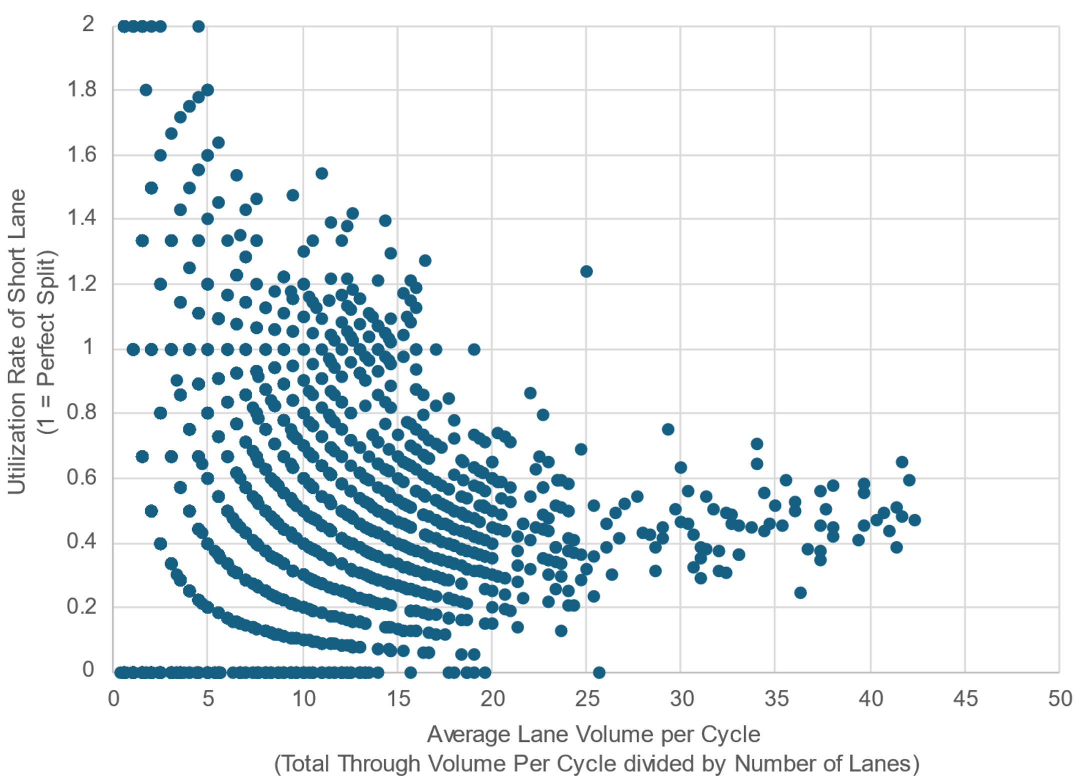
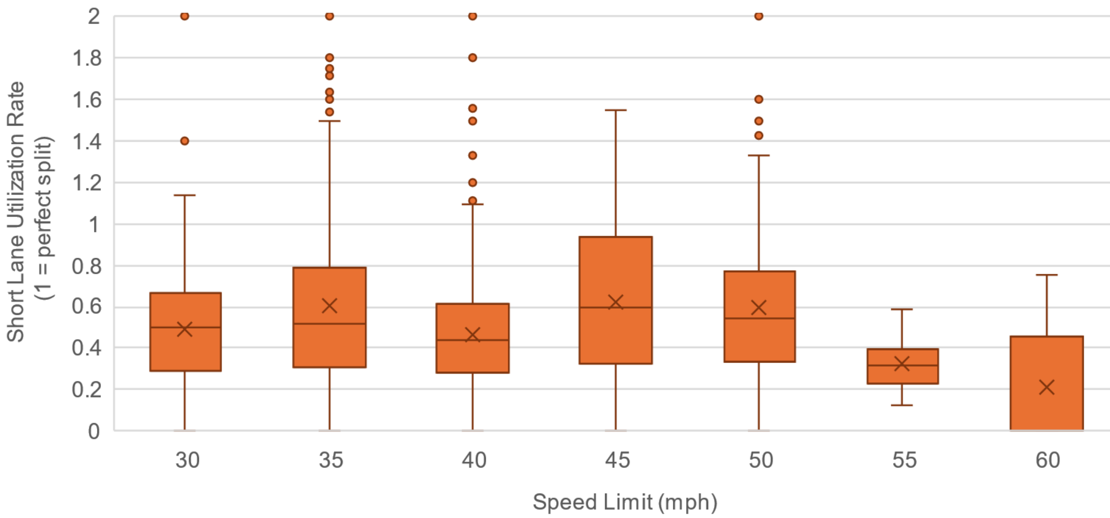


Figure 4-1 Utilization rate versus approach volume.

Figure 4-2 shows the short lane utilization rate plotted against the speed limit. No strong trend appears present except for the decrease in utilization rate shown for speed limits of 55 MPH and 60 MPH. It should be noted that only one study approach had a speed limit of 55 MPH and only two study approaches had a speed limit of 60 MPH. However, it is possible that the lower utilizations are due to longer green times that are often present on approaches with higher speeds.



Note: For this and all box-and-whisker plots in this report, the middle 50% of the data falls within the colored rectangles, the horizontal bar in the rectangle represents the median, the X represents the mean, and the round points represent any outliers.

Figure 4-2 Utilization rate versus speed limit.

Figure 4-3 shows the short lane utilization rate plotted against the short lane length (striped length). With some exceptions, there appears to be a slight upward trend of utilization rate increasing as the short lane length increases. This is a logical trend because vehicles are more likely to stay in the short lane if they have a longer opportunity during which to merge before the lane drops. Note that the location with the longest short lane has the lowest utilization rates – this is an indicator that something about that location is significantly different than the rest of the sites.

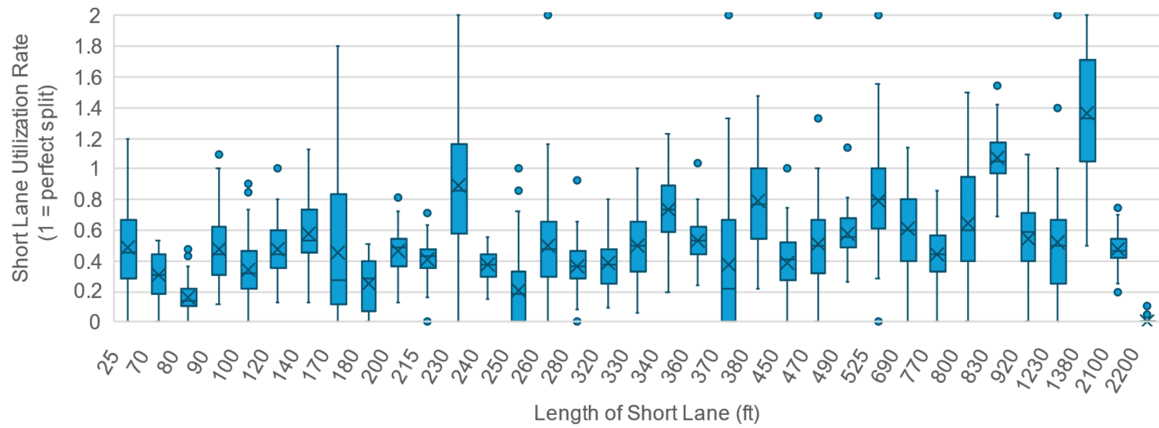


Figure 4-3 Utilization rate versus striped length.

Figure 4-4 shows the short lane utilization rate plotted for different lane drop types including trap lanes, through-right lanes, and ATLs. These charts do not suggest that the lane drop type affects the utilization rate.

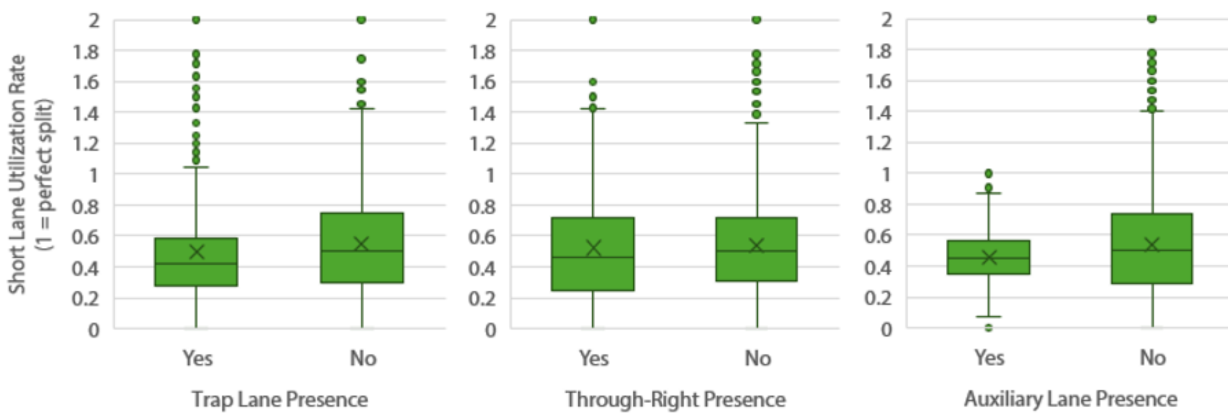


Figure 4-4 Utilization rates for different special lane drop types.

4.4.2 15-Minute Period Observations

This section discusses the same trends as the previous section but for the data after it has been grouped into 15-minute periods rather than individual cycles. In doing this, each location has the same number of points (eight) to make comparisons more normalized.

Figure 4-5 shows the short lane utilization plotted against the approaching average volume per hour per lane. The per-lane volume is used because the locations studied included sites with differing numbers of through lanes (2 or 3, including the short lane). This figure shows that the utilization rate is more random at low volumes but appears to converge to approximately 0.5 at higher volumes.

There are eight points with a utilization rate of less than 0.15 – all of these are from location #36 (S.R. 201 and S.R. 202 eastbound approach in Magna), and the eight points constitute the entirety of the location #36 data. Due to the extremely low utilization rates, the researchers investigated the location further, noting several unusual features, including signed warning of the lane drop far in advance of the intersection and a similarity of the pavement markings of this lane drop to several upstream deceleration and acceleration lanes. Drivers likely assume the lane drop is a deceleration or acceleration lane, thus eliminating the operational benefit of an extra lane at the intersection. The research team removed the location from the later statistical analysis.

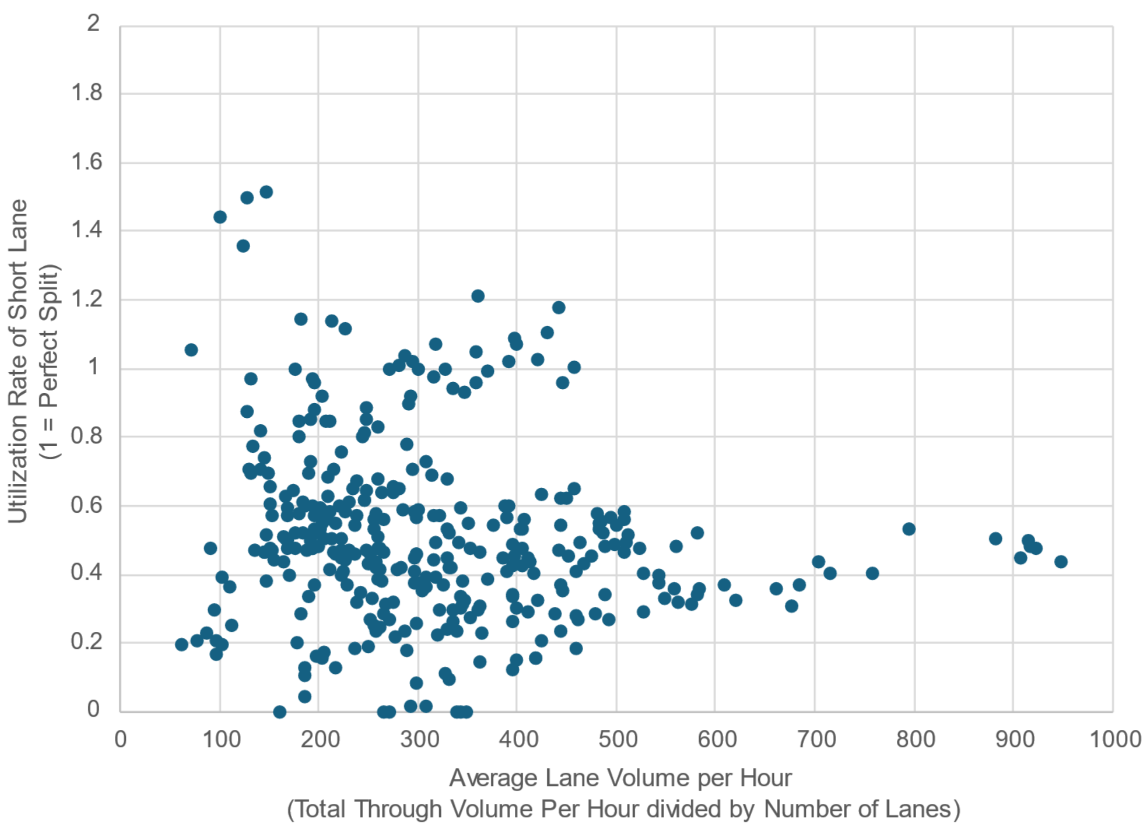


Figure 4-5 Utilization rate versus average approaching volume per hour per lane.

Figure 4-6 shows the short lane utilization rate plotted against the speed limit. The boxplots show that speed limits of 35 and 45 MPH have a larger range, likely because there are more locations with that speed limit. Note also that the box for the 60 MPH category touches zero – this is due to location #36, which has a speed limit of 60 MPH, being included in this part of the plot. As was previously discussed, location #36 will be excluded from the statistical analysis due to significant differences from other locations.

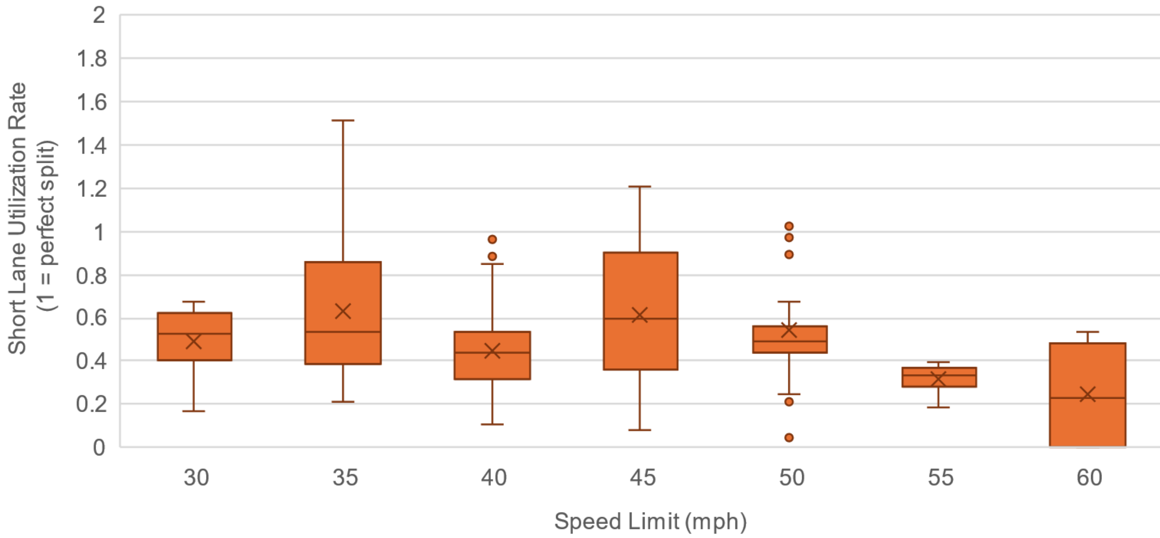


Figure 4-6 Utilization rate versus speed limit.

Figure 4-7 shows the short lane utilization rate plotted against the short lane length (striped length). There appears to be a possible upward trend of the utilization rate increasing as the short lane length increases. This would be a logical trend because vehicles are more likely to stay in the short lane if they have a longer opportunity during which to merge before the lane drops. Note that location with a short lane length of 2,200 ft (and the lowest utilization rate) is location #36.

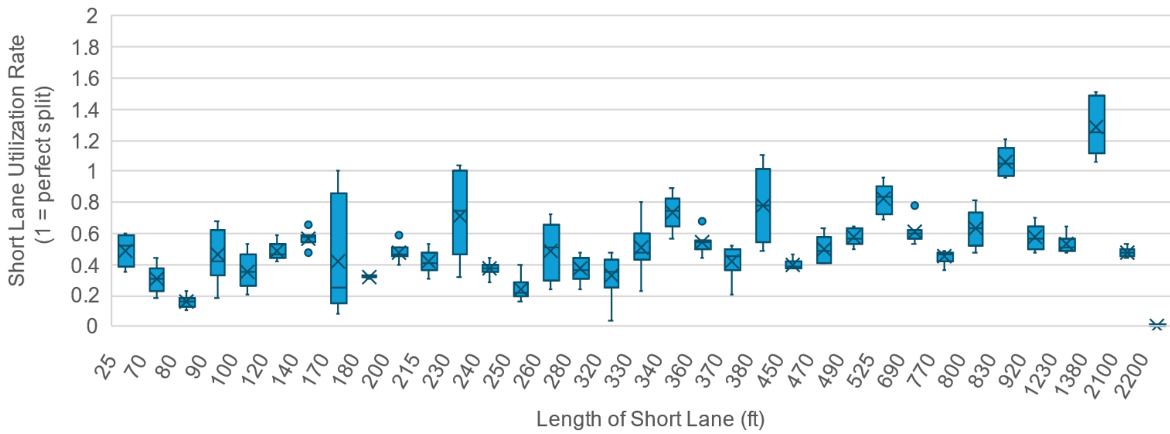


Figure 4-7 Utilization rate versus the length of the short lane (striped length).

Figure 4-8 shows the short lane utilization rate plotted for different lane drop types including trap lanes, through-right lanes, and ATLs. These charts do not suggest that the lane drop type affects utilization rate.

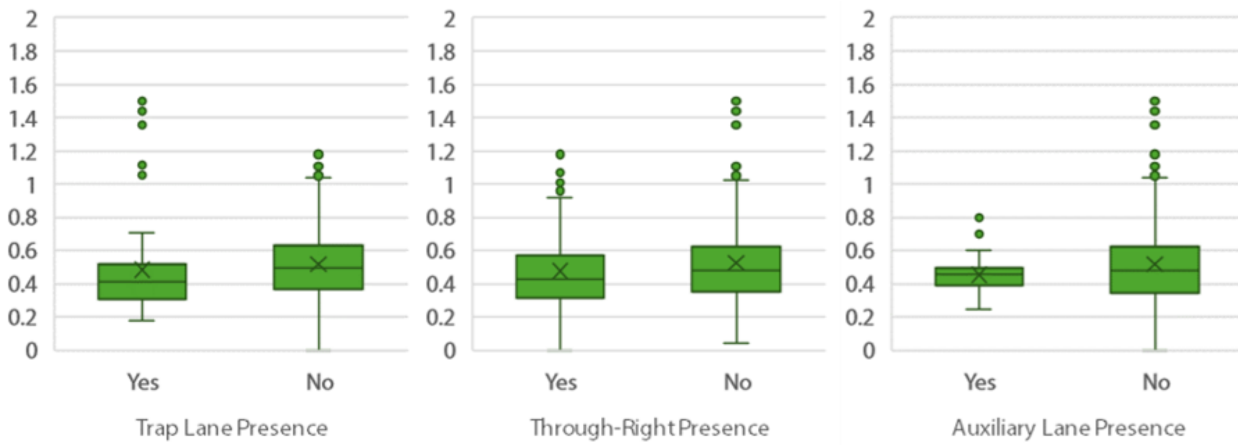


Figure 4-8 Utilization rates for different special lane drop types.

4.5 Statistical Analysis

A statistical analysis was performed on the 15-minute data (excepting location #36, which was completely removed from the dataset as previously discussed) to build upon the findings from the observational analysis and to quantify the impacts of roadway and volume characteristics on the utilization rate. The benefits of building statistical regression models are that several variables can be assessed for their simultaneous impacts on lane utilization and the regression models can be used for predictive purposes. Statistical regression models were estimated using Statistical Analysis Software (SAS).

4.5.1 Statistical Methods

The statistical analysis was performed using Ordinary Least Squares (OLS) linear regression models. Using a case-wise selection method, the variables of interest were examined to identify potential significant correlations. The model runs performed are shown in Table 4-2. After examining these preliminary relationships, shown in each model run, a final model was developed using the significant variables.

Table 4-2 Stepwise Regression Models for Short Lane Utilization

Variable	***Model 1***	***Model 2***	***Model 3***	***Model 4***
Vehicles per hour				
Vehicles per hour per lane				
Number of Lanes		***	***	***
Striped Length (ft)	***	***	***	***
Trap Lane				
Speed Limit				*
Auxiliary Through Lane (ATL)			**	*
Combined Through/Right Lane				
R^2	0.187	0.244	0.260	0.310

Note: NS = Not Significant, # = Significant at $p \leq 0.1$, * = Significant at $p \leq 0.05$,
** = Significant at $p \leq 0.01$, *** = Significant at $p \leq 0.001$

4.5.2 Selected Model

The selected model included the significant variables the case-wise selection method identified, namely number of lanes, striped length, speed limit, and presence of an ATL. Table 4-3 indicates the coefficient for the variable along with its statistical significance.

Table 4-3 Regression Coefficient Estimates for Short Lane Utilization Rate

Variable	*Estimate*	*Std. Error*	*T-value*	*P-value*
Intercept	0.902	0.102	8.877	<0.001
Number of lanes	-0.125	0.026	-4.826	<0.001
Striped length (ft)	0.00024	0.000	7.281	<0.001
Speed limit	-0.004	0.002	-2.188	0.035
Auxiliary Through Lane (ATL)	-0.104	0.045	-2.285	0.023
R^2	0.224			
Adj R^2	0.215			
Sample Size	341			

The resulting regression equation is given in Equation 4-2:

$$U = 0.902 - 0.125 \times N + 0.024 \times L - 0.004 \times S - 0.104 \times A \qquad (4\text{-}2)$$

Where U = short lane utilization rate,

N = number of through lanes present at the upstream signal

L = striped length (in 100s of feet)

S = speed limit in miles per hour

A = logical variable for ATL (1= ATL present, 0= no ATL present)

The influence of each variable on short lane utilization can be interpreted as follows:

- A lane drop location with one more through lane at the signal than another lane drop location is anticipated to have a lower short lane utilization rate by 0.125.
- A lane drop location with a speed limit 5 MPH higher than another lane drop location is anticipated to have a lower short lane utilization rate by 0.020 (which is 5 times 0.004).
- A lane drop location with an ATL is anticipated to have a lower short lane utilization rate by 0.104 compared to a location without an ATL.

- A lane drop location with a short lane length (striped length) 100 feet longer than another lane drop location is anticipated to have a larger short lane utilization rate by 0.024 (which is 100 times 0.00024).

Figure 4-9 summarizes the predicted short lane utilization rate based on the speed limit by various combinations of number of lanes and ATL presence, assuming the striped length is 500 ft. Observed values are also shown.

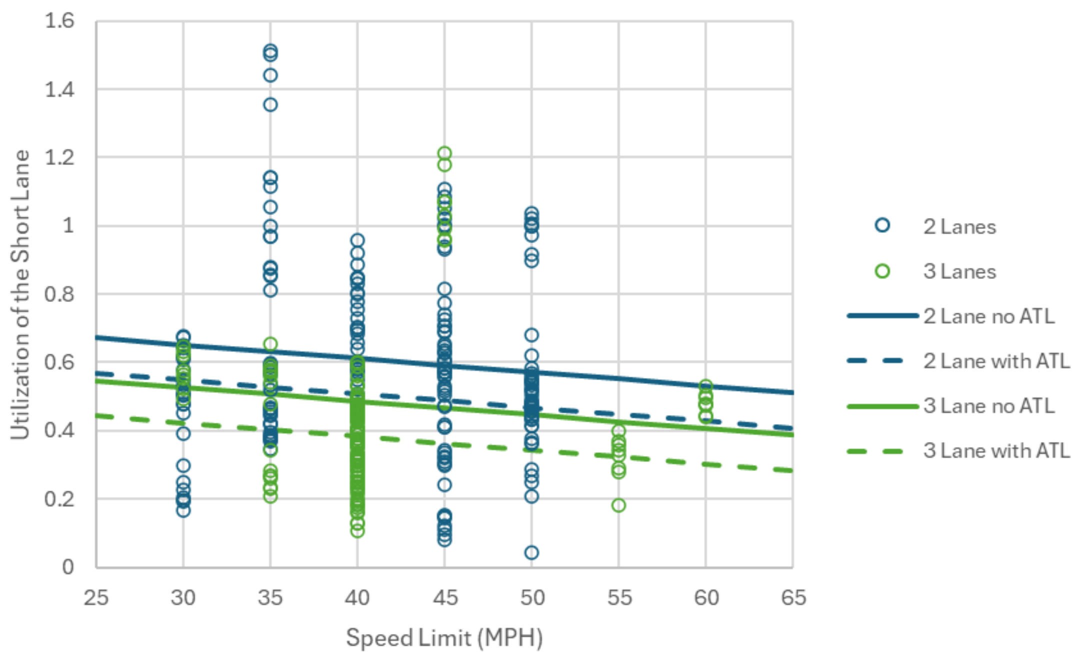


Figure 4-9 Predicted lane utilization rates by speed limit and lane configuration.

Figure 4-10 summarizes the predicted short lane utilization rate based on the speed limit by various striped lengths, assuming there are three through lanes present at the signal and there is no ATL. Observed values are also shown.

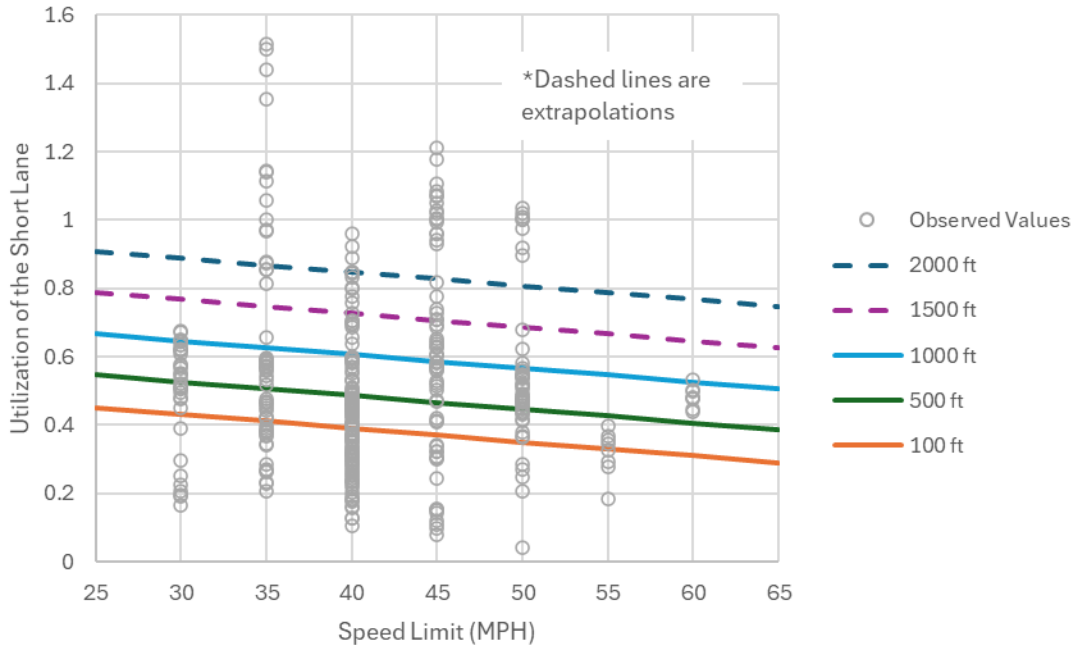


Figure 4-10 Predicted short lane utilization rate based on the speed limit by striped length.

Figure 4-11 summarizes the predicted short lane utilization rate based on the striped length by various combinations of speed limit, assuming there are three through lanes present at the signal and there is no ATL. Observed values are also shown.

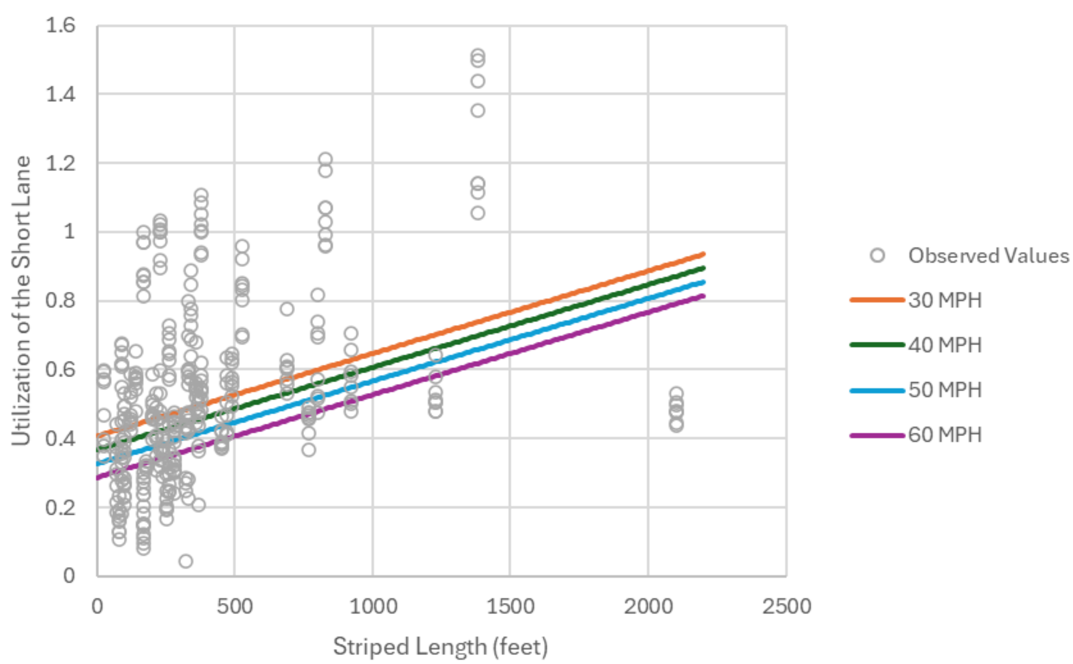


Figure 4-11 Predicted short lane utilization rate based on the striped length by speed limit.

4.6 Summary

Observational analysis of the arterial lane drop data revealed initial trends in the data that were quantified in the statistical analysis. After using a case-wise selection method to identify variables for use in a statistical model, the selected linear regression model was developed with significant variables that include number of through lanes at the signal, striped length of the short lane, speed limit, and presence of an ATL. The statistical model predicts that an increase in short lane utilization rate is correlated with a decrease in number of lanes, a decrease in speed limit, the absence of an ATL, and an increase in striped length.

5.0 DATA EVALUATION - RAMPS

5.1 Overview

This chapter discusses the analysis and evaluation of the collected data for lane drops on freeway on-ramps. First, is an explanation of how the data from the 24 locations were manipulated to match the structure needed for further calculations. Next, is a description of the aggregations and calculations performed on that data. Following that, is a discussion on the observational analysis for the dataset, including explanations of trends observed in the data. Finally, a discussion on the statistical analysis is given which includes numerical and graphical representations of the selected statistical model.

5.2 Combining Datasets into Desired Structure

The data required the following structure prior to performing the analysis:

- Each data entry (row) included maximum right-turn queues and volumes from the inside left-turn lane, outside left-turn lane, and right-turn lane (see Figure 3-5 for a diagram showing the lane configuration) from a 5-minute period for each specific location.
- Unique identifying information about the location to which the volumes belonged needed to be included in each row.

5.3 Aggregations and Calculations

Hourly volumes (flow rates) were calculated for each lane of each 5-minute period and location by multiplying raw 5-minute counts by 12 to produce hourly flow rates with units of vehicles-per-hour (vph). Flow rates from total left-turning vehicles were calculated by summing left-turn lanes, and flow rates from total vehicles entering the ramp were calculated by summing all lanes. As shown in Figure 5-1, most of the 5-minute periods had flow rates between 450 and 1200 vph.

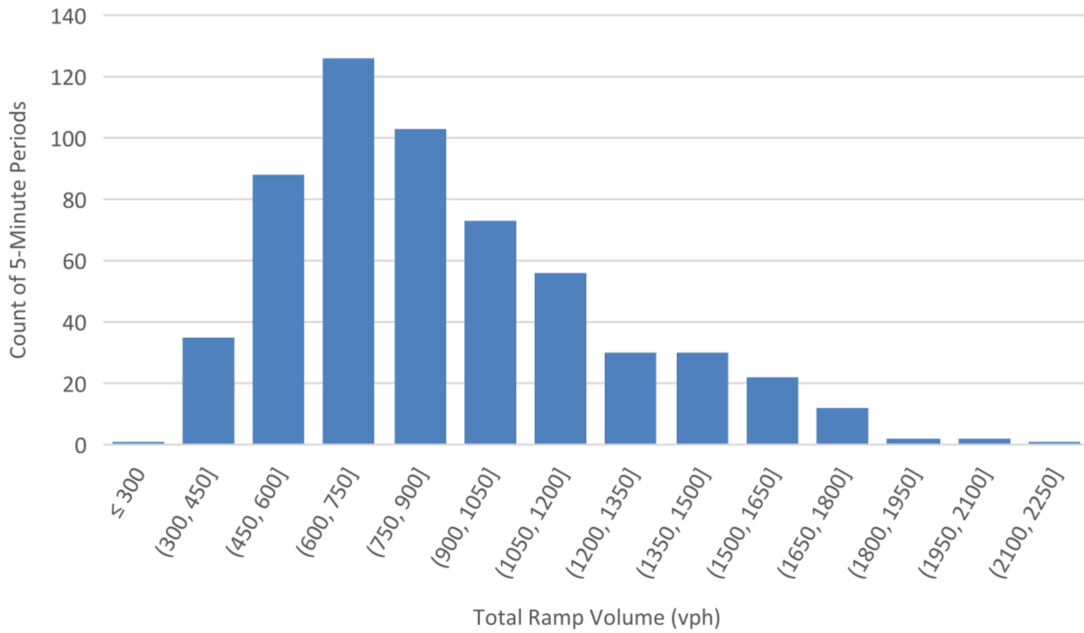


Figure 5-1 Distribution ramp flow rates.

5.4 Observational Analysis

Trends in the data were analyzed to better understand which variables might be important to test in a statistical model. Figure 5-2 shows the maximum right-turn queue plotted against total ramp volume. The data are aggregated in 5-minute periods. The data are clear enough to draw trendlines through, but there is an excess of zeros. While the trendlines are not sophisticated, they show that right-turn queues seem to increase mostly linearly as volume increases.

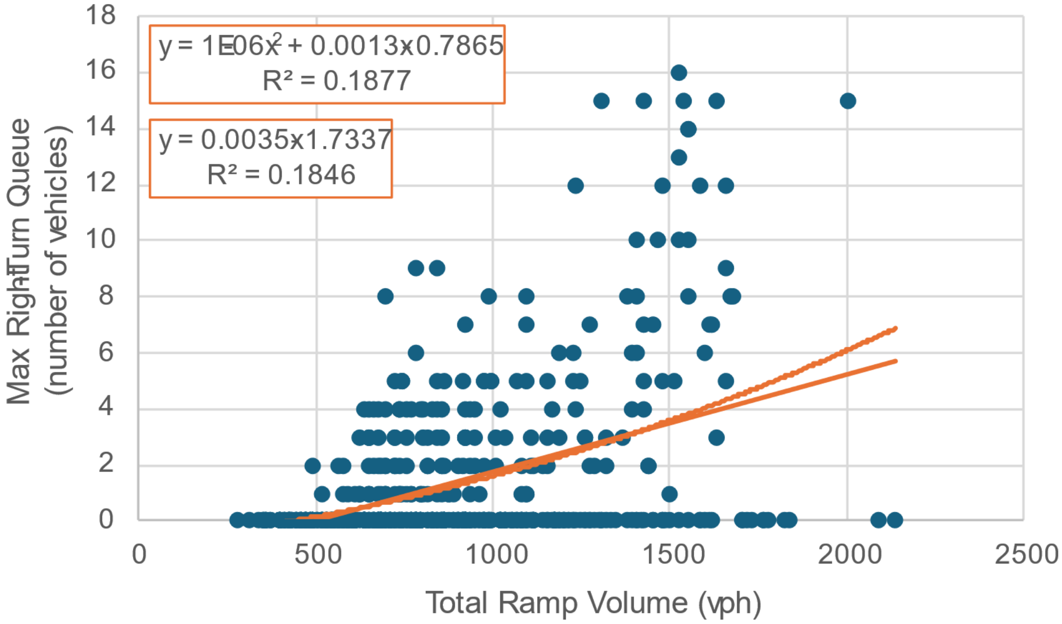


Figure 5-2 Max right-turn queue versus total ramp volume (vph).

Next, Figure 5-3 shows the max right-turn queue plotted against the merge length. The trendlines show that max right-turn queues decrease as merge length increases. The relationship between max right-turn queue and merge length appears to be non-linear and may follow a polynomial curve. This indicates that right-turn queues decrease rapidly as merge length increases, and there is less benefit at higher merge lengths. This follows observations of driver behavior which noted that most drivers took advantage of longer merge lengths, but several drivers stopped and waited to merge even if they were given what would appear to be adequate merge length.

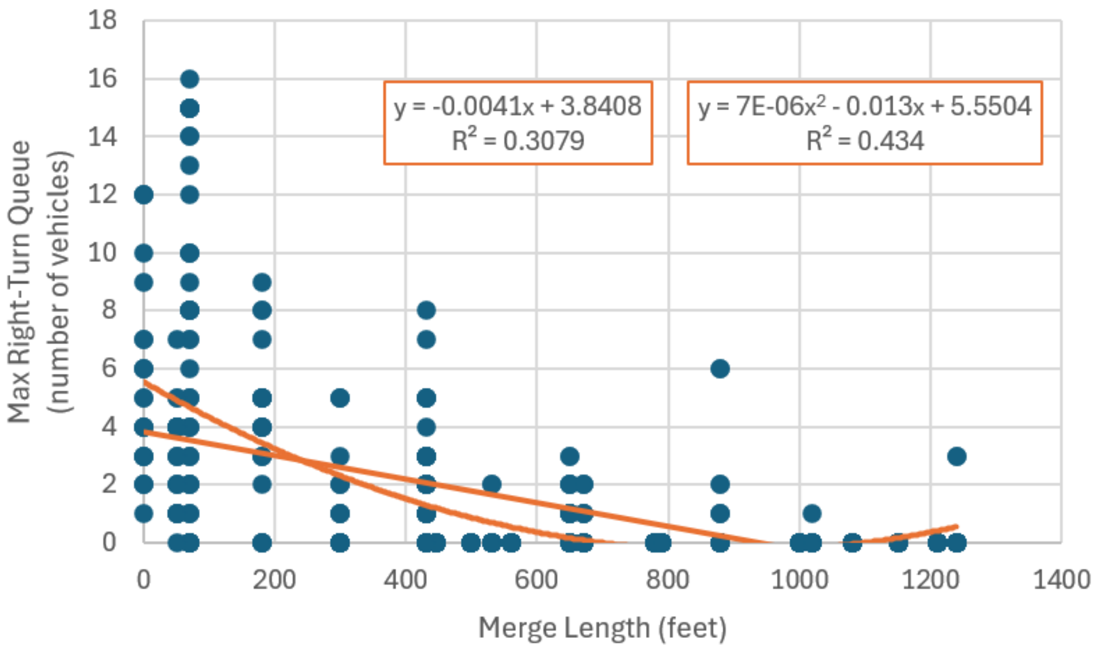


Figure 5-3 Max right-turn queue versus merge length (feet).

5.5 Statistical Analysis

A statistical analysis was performed on the data to build upon the findings from the observational analysis and to quantify the impacts of roadway and volume characteristics on right-turn queues. The benefits of building statistical regression models are that several variables can be assessed for their simultaneous impacts on right-turn queues, and the regression models can be used for predictive purposes. Statistical regression models were estimated using Statistical Analysis Software (SAS).

5.5.1 Statistical Methods

Ordinary Least Squares (OLS) linear regression models were initially considered for modeling the relationship between max right-turn queues and other variables. Using a case-wise selection method, the variables of interest were examined to identify potential significant correlations. The model runs performed are shown in Table 5-1. After examining these preliminary relationships, shown in each model run, a final model was developed using the significant variables. Figure 3-5 may be referred to illustrate variables such as advance length, striped length, merge length, and outside left-turn lane volume.

Table 5-1 Stepwise Regression Models for Queue Length

Variable	*Model Runs*								
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
Lane drops at the ramp meter (yes/no)									
Advance length (ft)							***	***	***
Striped length (ft)									
Taper length (ft)									
Merge length (ft)					***				
Square-root of merge length	***	***	***	***	*	**	***	***	***
Total length (Advance length + Merge length)						***	***	***	***
Square-root of total length				***	***	***	***	***	***
Right-turn hourly volume			***	***	NS				
Outside left-turn lane hourly volume								*	***
Total left-turn hourly volume		***	***	***	***	***	***	NS	
Total ramp hourly volume									
Outside left-turn lane utilization									
Right-turn volume to left-turn volume ratio									
R^2	0.402	0.543	0.578	0.616	0.682	0.682	0.705	0.710	0.710

Note: NS = Not Significant, # = Significant at p ≤ 0.1, * = Significant at p ≤ 0.05, ** = Significant at p ≤ 0.01, *** = Significant at p ≤ 0.001

Because the distribution of the data for the max right-turn queue is not normally distributed as shown in Figure 5-4, a secondary model was tested to better fit the data. This model was a Zero-Inflated Poisson (ZIP) model, used to account for the strong overrepresentation of zero count observations in the data.

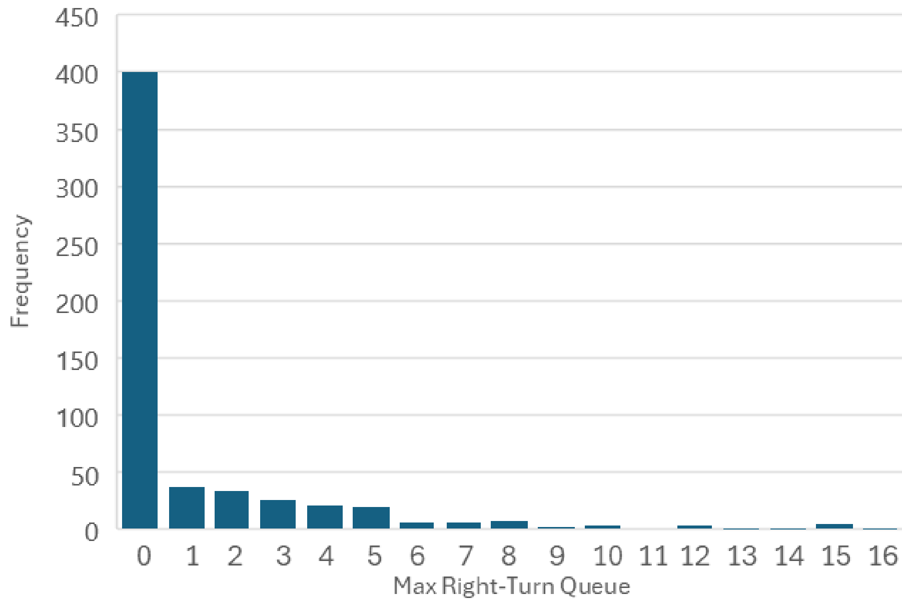


Figure 5-4 Histogram of max right-turn queues.

Zero-Inflated Poisson regression is used to model count data that has an excess of zero counts. The excess zeros are generated by a separate process from the remainder of the count values, and zeros can be modeled independently. Thus, the ZIP model has two parts, a Poisson count model and a logit model for predicting excess zeros (UCLA, 2024). The Poisson count model uses a standard negative binomial regression structure, and the logit model uses a Poisson distribution model to predict whether or not the outcome (in this case, the number of vehicles in the right-turn queue) is zero. The output of the logit model for this research is given in units of "log odds," which is the logarithm of the odds (where the odds are equal to the probability of having no queue divided by the probability of having a queue). For this research, the same variables used in the linear regression model tests were used in the ZIP model test.

5.5.2 Selected Models

Since there was an excess of zero counts in the right-turn queues, two models were required to understand relationships in the data. First the zero-inflated model was used to represent the log odds that a queue does not exist. This was done since the excess of zero counts made it unreasonable to model zeros with a normal regression. Table 5-2 shows the coefficients for the zero-inflated model. The second model, the count model, was used to represent queue lengths when a queue exists. Two models were considered for the count model, a Negative Binomial model based on only the non-zero data, and a linear regression model based on all the data. The coefficients for the Negative Binomial and linear models are shown in Table 5-3 and Table 5-4 respectively. The linear model was chosen as the count model because it fit the observed trends better.

Table 5-2 Zero-Inflated Poisson Model of Max Right-Turn Queue

Variable	***Estimate***	***Std. Error***	***T-value***	***P-value***
Count Model				
Intercept	3.997	10.405	.384	0.701
Advance Length (ft)	0.032	0.011	2.840	0.005
Merge Length (ft)	-0.016	0.019	-.855	0.393
Square-Root of Merge Length	3.125	1.559	2.005	0.045
Total Length (Advance Length + Merge Length)	*Not included in final model due to collinearity*			
Square-Root of Total Length	-2.549	0.859	-2.968	0.003
Right-Turn Hourly Volume	0.006	0.004	1.611	0.107
Outside Left-Turn Lane Hourly Volume	-0.017	0.011	-1.563	0.118
Left-Turn Total Hourly Volume	0.001	0.003	0.191	0.849

The zero-inflated model coefficients can be used to understand the log odds that a queue does not exist given a change in the associated variable. In terms of predicting a zero-length queue in the right-turn lane, the baseline odds for having zero vehicles in the right-turn queue is nearly 4 (3.997). Increasing the Advance Length (0.032) has a tendency to increase the odds of no queue, as does increasing the Square Root of Merge Length (3.125). However, these variables are both related to the Total Length (not included in the model). An increase in Square Root of Total Length offsets these increased odds by decreasing the likelihood of a zero-length queue

(-2.549). Because this is an interaction model, there are no clear measured coefficients relating to decreasing or increasing the number of vehicles queueing. However, the relationships are significantly tied to the likelihood of there being zero vehicles in the queue.

For example, in a scenario with 100 feet of advance length and 100 feet of merge length the log odds ratio of no queues increases from 2.4 to 7.2 if the merge length is increased to 200 feet. This was determined by calculating the values for each scenario using only the intercept and significant coefficients (i.e., Advance Length, Square-Root of Merge Length, and Square-Root of Total Length). Figure 5-5 shows the log odds of no queue based on merge length by various advance lengths.

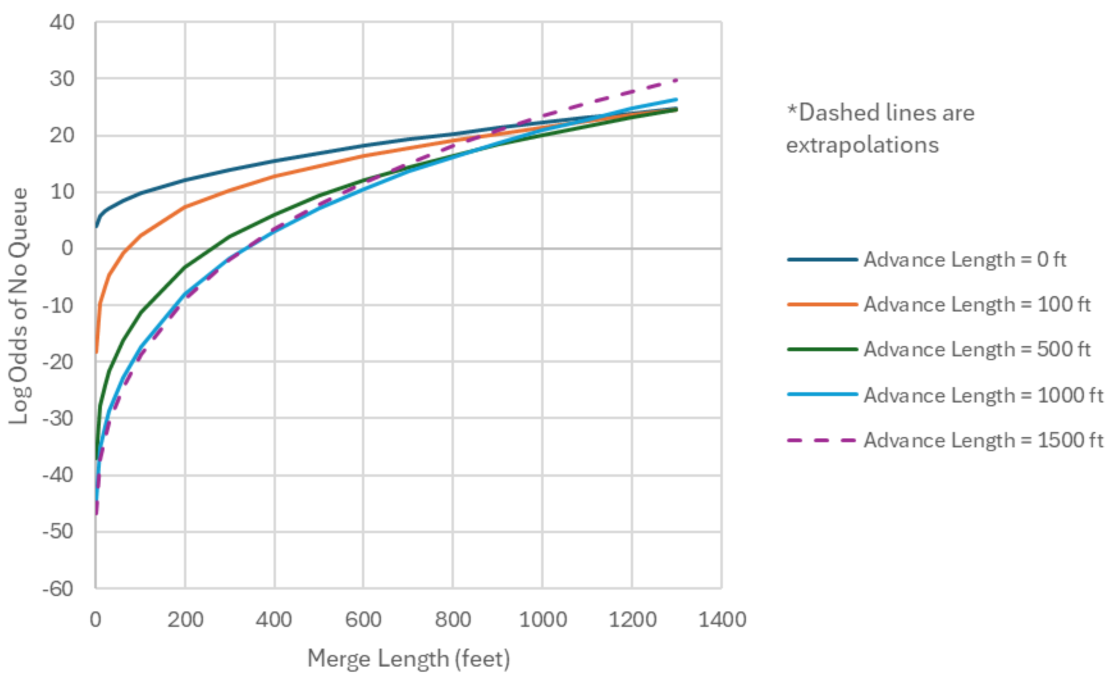


Figure 5-5 Predicted relative log odds of no queue based on the merge length by advance length.

While the zero-inflated model is useful for predicting whether a queue will exist or not, a count model was required to predict the queue length if a queue exists. Initially, a Negative Binomial regression model based only on non-zero-length queue data was investigated for the count model since it separated the non-zero data from the zero data. However, the Negative Binomial regression count model outputs shown in Table 5-3 did not follow observations pertaining to max right-turn queue length and merge length. For example, the positive coefficient for the square-root of total length in the Negative Binomial regression count model would show that increasing merge length correlates to increased max right-turn queues, which contradicts observations about merge length shown previously in Figure 5-3.

Table 5-3 Negative Binomial Count Model Coefficients

Variable	***Estimate***	***Std. Error***	***T-value***	***P-value***
Count Model				
Intercept	-2.148	0.484	-4.433	<.001
Advance Length (ft)	-0.005	0.001	-5.806	<.001
Merge Length (ft)	-0.004	NaN	NaN	NaN
Square-Root of Merge Length	-0.025	0.022	-1.116	0.264
Total Length (Advance Length + Merge Length)	*Not included in final model due to collinearity*			
Square-Root of Total Length	0.206	0.040	5.217	<.001
Right-Turn Hourly Volume	0.002	0.000	8.014	<.001
Outside Left-Turn Lane Hourly Volume	0.002	0.002	1.380	0.168
Left-Turn Total Hourly Volume	0.000	0.001	0.284	0.776

Note: NaN = "Not a Number"

A linear regression count model including non-zero and zero data was chosen over the Negative Binomial regression count model since its output aligns better with observations. The coefficients for the selected model are shown in Table 5-4. Even though the linear model includes zeros, these coefficients are only useful to represent queue lengths given the condition that a queue exists. If there is no queue, the zero-inflated model should be used instead. However, the zero-inflated model does not predict queue lengths, which are important for understanding the performance of freeway on-ramp facilities. Therefore, both models are necessary to understand the performance of freeway on-ramp facilities.

Table 5-4 Linear Regression Coefficient Estimates for Max Right-Turn Queue

Variable	*Estimate*	*Std. Error*	*T-value*	*P-value*
		Count Model		
Intercept	11.002	1.499	7.341	<.001
Advance Length (ft)	0.006	0.001	4.369	<.001
Merge Length (ft)	0.008	0.001	5.470	<.001
Square-Root of Merge Length	-0.144	0.035	-4.120	<.001
Total Length (Advance Length + Merge Length)	*Not included in final model due to collinearity*			
Square-Root of Total Length	-0.526	0.091	-5.755	<.001
Right-Turn Hourly Volume	0.002	0.000	4.273	<.001
Outside Left-Turn Lane Hourly Volume	0.00041	0.002	-0.185	0.853
Left-Turn Total Hourly Volume	0.005	0.001	4.354	<.001
R^2	0.590			
Adj R^2	0.585			
N	581			

The intercept and other significant coefficients, (i.e., Advance Length, Merge Length, Square-Root of Merge Length, Square-Root of Total Length, Right-Turn Hourly Volume, and Left-Turn Hourly Volume) shown above were used to build an equation for predicting the max right-turn queues given a queue exists. The resulting regression equation is given in Equation 5-2:

$$RTQ = 11.002 + 0.6 \times AdvL + 0.8 \times M - 0.144 \times srM - 0.526 \times srT + 0.2 \times RV + 0.05 \times LV \quad (5\text{-}2)$$

Where: $RTQ =$ the max right-turn queue,

$AdvL =$ the length of the advance lane in 100s of feet,

$M =$ the length of the merge in 100s of feet,

$srM =$ the square root of the length of the merge in feet,

$srT =$ the square root of the combined merge and advance length in feet,

$RV =$ the right-turn hourly volume in 100s of vehicles per hour,

$LV =$ the total left-turn volume in 100s of vehicles per hour.

Figure 5-6 summarizes the predicted max right-turn queues based on the advance length by various merge length. Observed values are also shown. The following worst-case assumptions were made about volumes based on some of the highest volumes seen in the data:

- Right-turn hourly volume = 700 vph,
- Left-turn hourly volume = 900 vph.

This chart shows that predicted right-turn queues decrease as merge length and advance length increase. Interestingly, even though increased advance length correlated to a higher log odds that queues existed as shown previously in Figure 5-5, Figure 5-6 shows that it also correlates to shorter queues when queues exist. This is especially true for shorter merge lengths, suggesting that advance length may be able to supplement deficient merge lengths. However, there is no correlated benefit from merge lengths or advance lengths longer than 1500 feet.

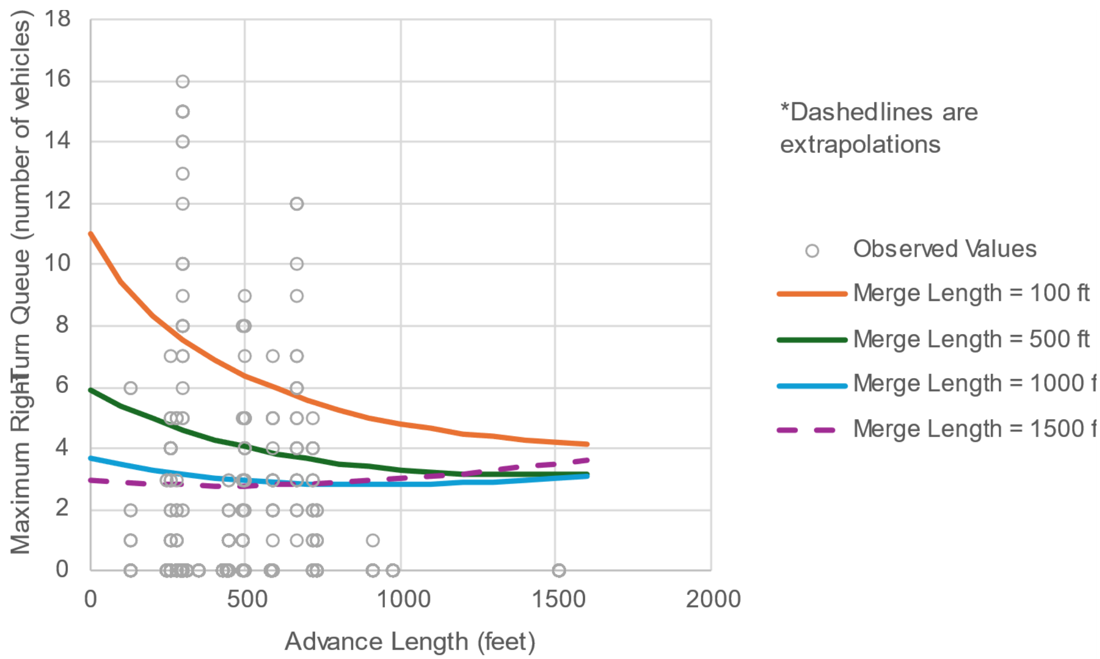


Figure 5-6 Predicted max right-turn queue based on the advance length by merge length.

Figure 5-7 summarizes the predicted max right-turn queue based on advance length by various total ramp volumes. The following assumptions were made for merge length and the right-turn to left-turn volume ratio based on averages in the data:

- Merge length = 500 feet,
- Right-turn versus left-turn volume ratio = 45/55.

This chart shows that there is a strong correlation between increasing ramp volume and increasing right-turn queues. The chart also illustrates that increasing advance length correlates to shorter queues, especially at lengths below 1000 feet.

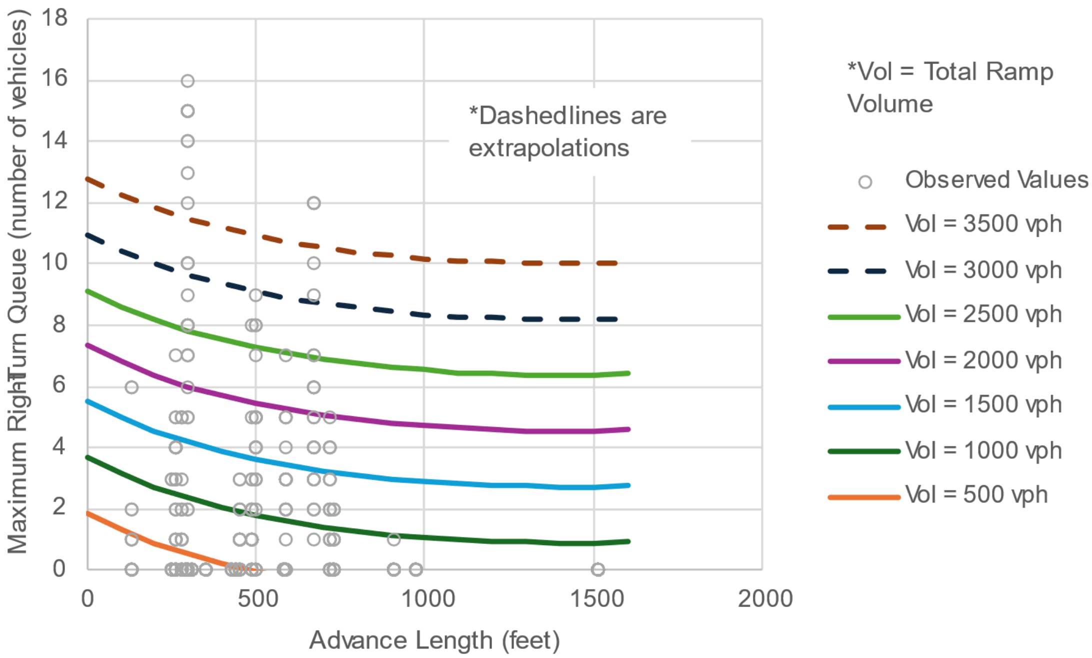


Figure 5-7 Predicted max right-turn queue based on the advance length by total ramp volume.

Figure 5-8 summarizes the predicted max right-turn queue based on merge length by various total ramp volumes. The following assumptions were made for advance length and the right-turn to left-turn volume ratio based on averages in the data:

- Advance length = 400 feet,
- Right-turn versus left-turn volume ratio = 45/55.

The chart illustrates that increasing merge length correlates to shorter queues, especially at lengths below 1000 feet, similar to advance length. It is important to recognize that while the charts for advance and merge length look slightly different, changes in the advance length affect the shape of this chart and changes in merge length affect the shape of the previous chart shown in Figure 5-7.

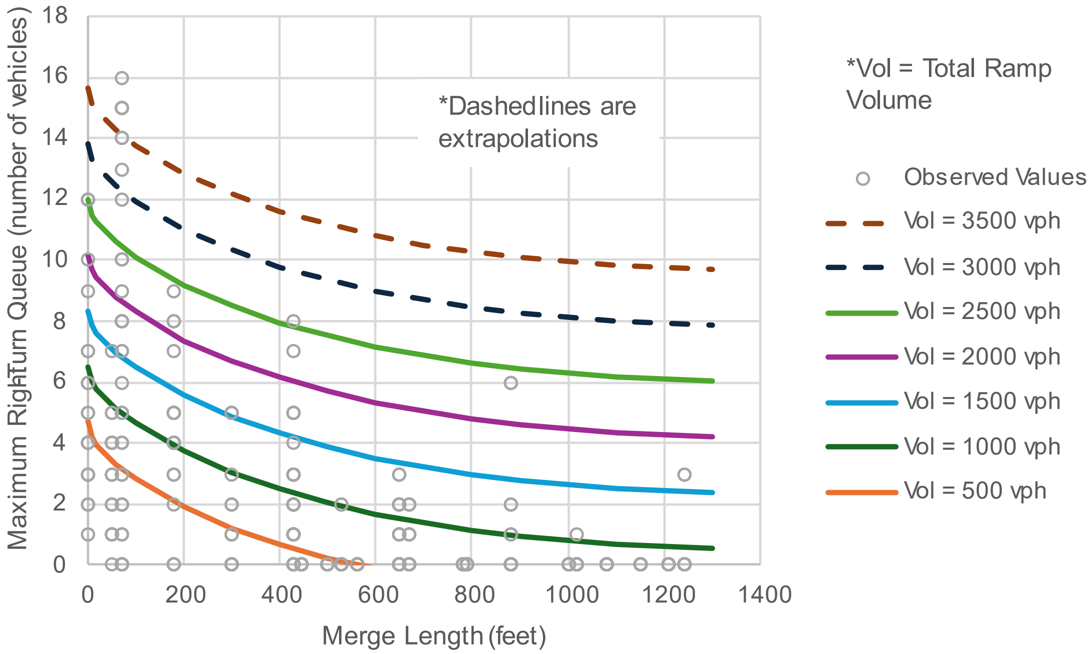


Figure 5-8 Predicted max right-turn queue based on the merge length by total ramp volume.

Figure 5-9 summarizes the max right-turn queues based on total ramp volume by various total lengths. The following assumptions were made about the ratios between advance and merge length and right-turn versus left-turn volume based on averages in the data:

- Advance length versus merge length ratio = 45/55,
- Right-turn versus left-turn volume ratio = 45/55.

This chart shows that increased volume correlates to a linear increase in right-turn queues. Also, increasing total length correlates to a much stronger queue reduction at shorter lengths. There is no correlated marginal benefit at total lengths above 1,500 feet.

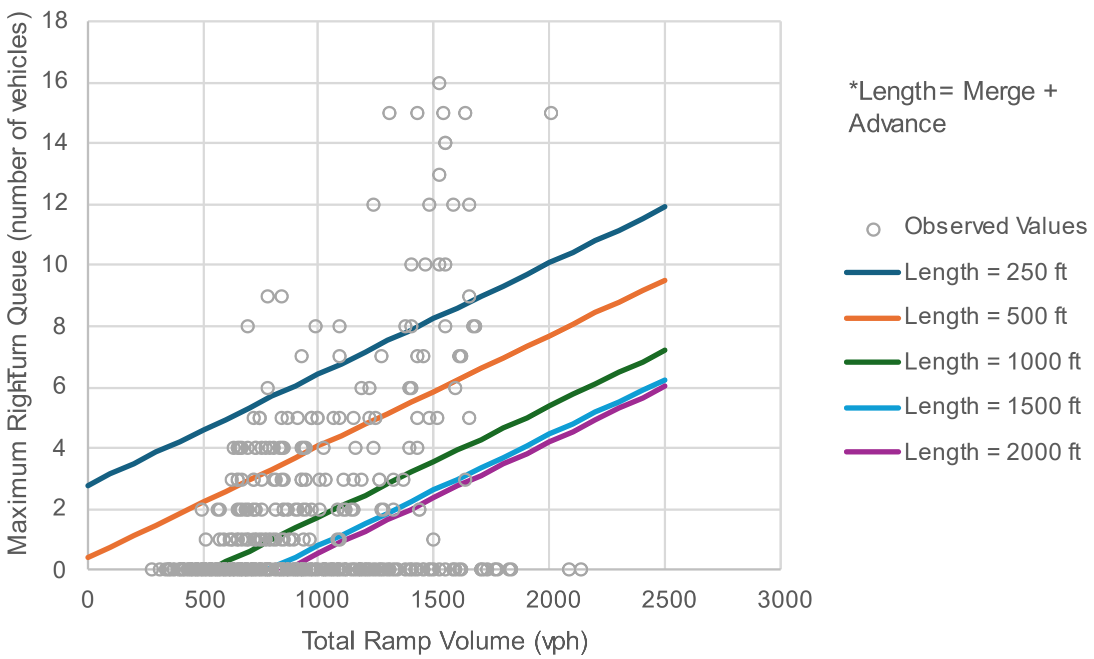


Figure 5-9 Predicted max right-turn queue based on total volume by total length (i.e., combined advance and merge length).

5.6 Summary

Observational analysis of the ramp lane drop data revealed initial trends that were quantified in the statistical analysis. Both ZIP and count models were investigated for predicting the presence of right-turn queues and queue lengths respectively. The variables in the ZIP model are advance length, square-root of merge length, and square-root of total length. The ZIP model predicts that increasing advance length is correlated to lower odds of no queue and increasing merge length is correlated to higher odds of no queue. A linear regression count model was selected over a Negative Binomial count model for predicting queue lengths. The variables in the selected count model are the advance length, the merge length, the square root of merge length, the square root of total length (merge length plus advance length), the right-turn hourly volume, and the left-turn hourly volume. The count model predicts that a decrease in max right-turn queue is correlated with an increase in advance length, an increase in merge length, a decrease in right-turn hourly volume, and a decrease in left-turn hourly volume.

6.0 CONCLUSIONS

6.1 Summary

In dropped lane scenarios, it is common for lane utilization of the short lane to decrease and vehicle queues to increase due to some drivers' tendency to merge before it is necessary. This research identifies the factors with the strongest potential influence on lane utilization and vehicle queues in two scenarios to help UDOT improve traffic conditions in these scenarios. The first scenario is when a lane is dropped directly downstream from a signalized intersection on an arterial roadway. The second scenario is when a lane is dropped on a freeway on-ramp after the entrance of a right-turn lane, forcing right-turning traffic to merge at or near the start of the on-ramp. For the arterial scenario, lane utilization was the primary concern; for the ramp scenario, queueing in the right-turn lane was the primary concern.

Data for this research included peak hour or near-peak-hour lane volumes. For arterial locations, these volumes were grouped by 15-minute bins and signal cycles of the adjacent intersection. For ramp locations, the volumes were grouped by 5-minute bins and the maximum right-turn queue was recorded for each 5-minute bin. Data were collected by obtaining video at each location either using UDOT Traffic Operations Center cameras or cameras set up by a third-party company. Videos were recorded during peak hours whenever possible or near peak hours in the case of metered freeway on-ramps (observations did not occur during metered times).

Data collection also included categorical observations about each location. For arterials, this included the number of lanes, speed limit, length of the striped lane prior to merging, and taper length. For ramps, this included the merge length (including the striped length and taper prior to merging), the advance length measured as the storage length upstream of where right-turning vehicles can start merging, and an indicator of whether the merge happened at a ramp meter. Sites selected for ramps only included locations with two left-turn lanes and a channelized right-turn lane.

Finally, metrics from the data such as lane utilization and hourly volume rates were calculated, and the data were compiled according to location characteristics and counting periods. Initial investigations identified patterns in the data using scatter plots and boxplots which helped with choosing variables to include in statistical models. Appropriate statistical models were created for arterial locations and ramp locations separately using multivariate linear regression.

6.2 Findings

Equations for arterial and ramp locations were derived using the statistical models previously described. For arterial lane drops, Equation 4-2 shows that a decrease in lane utilization correlates to an increase in the number of lanes, an increase in speed limit, the presence of an auxiliary through lane, and a decrease in the striped length. For ramp lane drops, Equation 5-2 shows that an increase in right-turn queues is correlated with an increase in volumes and a decrease in merge length and advance length.

The coefficients from these equations can be used to predict utilization and queues. For example, the arterials equation exhibits linear relationships between utilization and other variables. Predicted short lane utilization decreases by 0.125 for each additional lane, 0.004 for each speed limit increase in miles per hour, and 0.104 if an auxiliary through lane is present. Additionally, predicted short lane utilization increases by 0.024 for each additional 100 feet of striped length. It should be noted that there were few sampled locations with striped lengths above 600 feet, so this assumption is not strong in situations with longer striped lengths.

While the arterials equation is linear, the results for ramps are less straightforward because the ramps equation exhibits non-linear relationships. Graphing the relationships shows that a decrease in right-turn queue lengths is correlated to an increase in advance and merge length, but this relationship becomes less significant at about 1,500 feet in combined advance and merge length. Additionally, the ramps equation shows that predicted queue lengths increase by 0.2 vehicles for each additional 100 vph of right-turning volume and 0.02 vehicles for each additional 100 vph of left-turning volume. Due to the excess of zero length queues in the data, zero-inflation was also used to model the existence of queues. This model showed that the odds of zero length queues increased with increasing merge length and decreasing advance length.

6.3 Limitations and Challenges

As with any observational study, the results do not indicate causation and should be used with caution. Equations generated from statistical models indicate trends observed in the data which can be used to predict future trends, but should not be taken as principle. It is especially important to avoid using the statistical model to extrapolate past the limits of the available data. The statistical model can only confirm relationships observed in the sample data, so one should be careful not to input values into the model that are outside the observed range used to build the model.

A noted challenge with data collection in this study involved the video recognition software used to count vehicles at arterial locations. Errors in the data could only be fixed through a robust debugging process, which mostly occurred during Phase 1 of this research; however, there were still locations which needed to be counted manually. Future research involving similar methods for counting vehicles should be approached cautiously to ensure data accuracy.

7.0 RECOMMENDATIONS

7.1 Recommendations

Using the relationships from the data as evidence of driver behavior patterns, UDOT can implement charts drawn in this research to guide design practices and standards to optimize traffic capacity and safety. Figure 7-1, Figure 7-2, Figure 7-3, and Figure 7-4 could be used for lane drops after an intersection on arterials for four base conditions, respectively: two-lane arterials with no ATL, two-lane arterials with an ATL, three-lane arterials with no ATL, and three-lane arterials with an ATL.

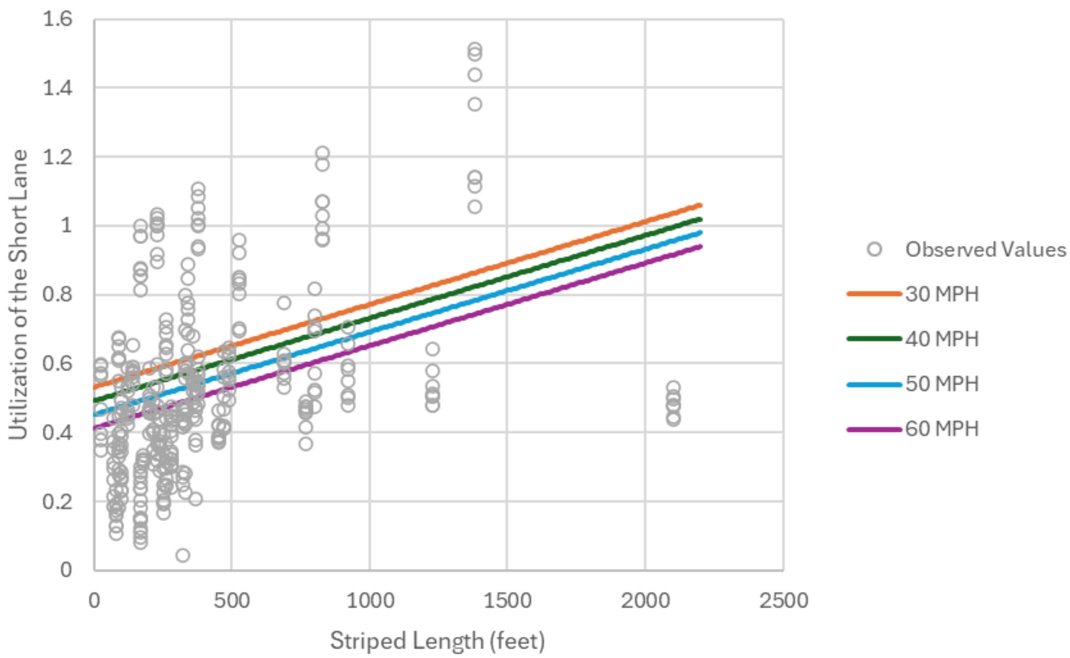


Figure 7-1 Utilization on two-lane arterials (no ATL) by striped length and speed limit.

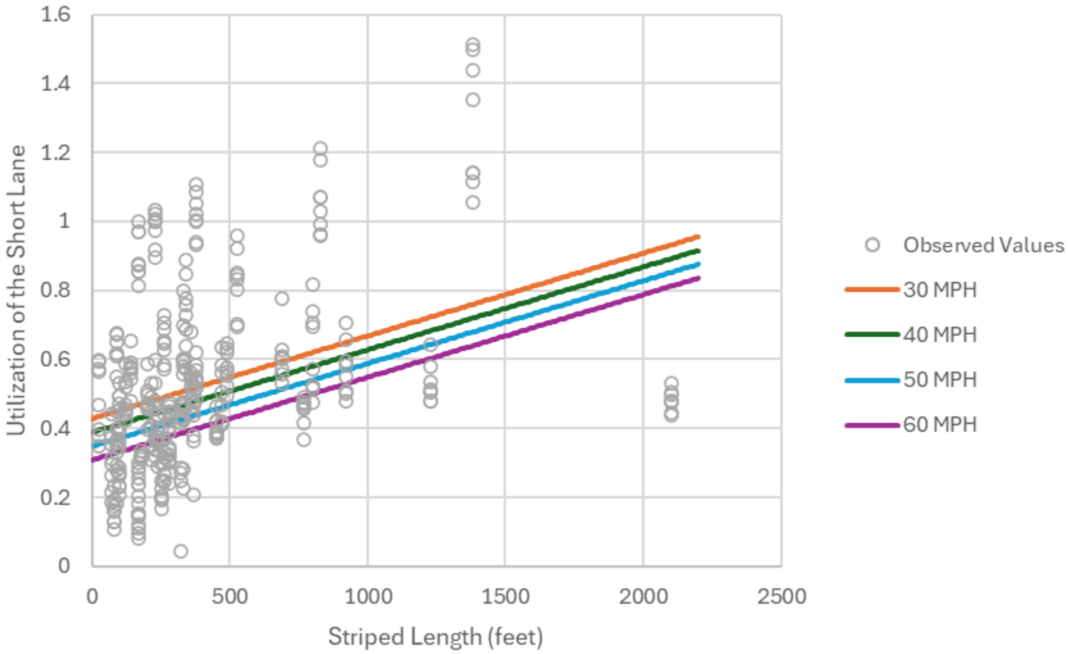


Figure 7-2 Utilization on two-lane arterials (with ATL) by striped length and speed limit.

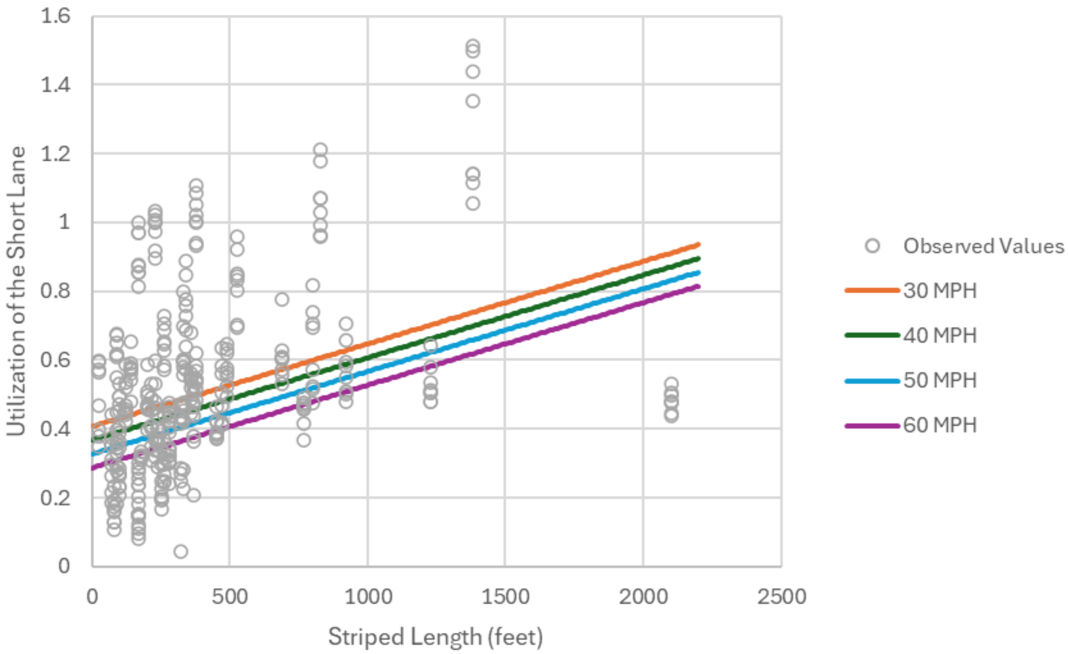


Figure 7-3 Utilization on three-lane arterials (no ATL) by striped length and speed limit.

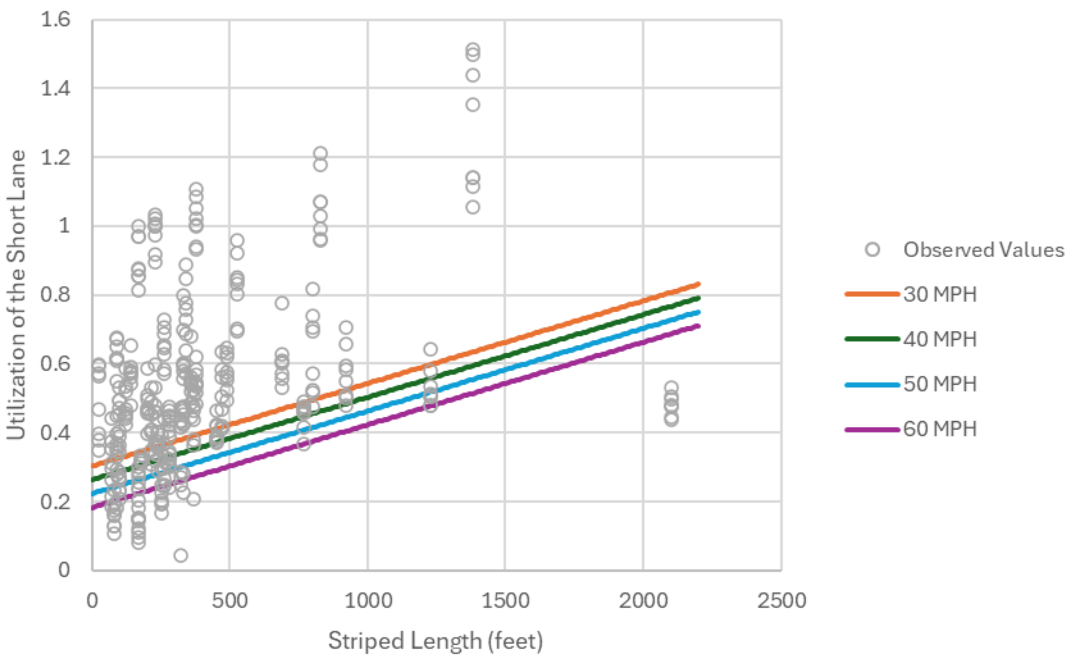


Figure 7-4 Utilization of three-lane arterials (with ATL) by striped length and speed limit.

While these figures are useful for illustrating most scenarios UDOT may encounter, Equation 4-2 may also be used to calculate predicted values for utilization of the short lane. However, the equation should only be used for estimations, and care should be taken not to extrapolate past the range of data used. Table 7-1 shows the thresholds to consider for inputs to the equation.

Table 7-1 Thresholds for Arterial Equation Inputs

Variable	Lower Bound	Upper Bound
Striped Length (ft)	25	2,100
Speed Limit (mph)	30	60
Number of Lanes	2	3
ATL Presence	0	1

For freeway on-ramps, Figure 5-6, Figure 5-7, Figure 5-8, and Figure 5-9 are useful representations for locations with conditions that match the assumptions identified in the paragraph preceding each figure. For cases that don't match the assumptions of any of those figures, Equation 5-2 may be used; however, inputs to this equation should be limited to ranges in the data to avoid extrapolation. Table 7-2 shows the thresholds to consider for inputs to the equation.

Table 7-2 Thresholds for Ramp Equation Inputs

Variable	Lower Bound	Upper Bound
Right-Turn Hourly Volume	0	1,880
Left-Turn Total Hourly Volume	50	1,360
Advance Length (ft)	130	1,520
Merge Length (ft)	0	1,240
Total Length (ft)	310	2,520

7.2 Further Research Opportunities

This research has identified valuable relationships related to lane drops. Additional value can be gained by investigating trends in other states or locations, or conducting this research at intersections with other types of control, such as roundabouts, near railroad crossings, or on freeway mainlines. Research conducted for different types of facilities could lead to robust standards for lane drops.

8.0 IMPLEMENTATION

8.1 Arterial Lane Drops Implementation

The results from the arterials lane drop analysis detail the relationship between lane utilization and the striped length. An effective application for this is in UDOT's Vissim traffic simulation guidelines. Traffic engineers could use the information from this research to input appropriate lane utilization factors into Vissim. One way to do this is with an incremental method where each additional 100 feet of striped length is used to increment the predicted lane utilization. Similar increments would also be used for the number of lanes, speed limit, and auxiliary through lane presence as shown in Table 8-1.

Table 8-1 Incremental Arterial Lane Utilization Calculation

Default Value for Utilization Rate is 0.556		
Parameter	**Base Assumption**	**How to adjust for differences from the Base Assumption**
Number of Lanes	2	subtract 0.125 for 3-lane scenarios
Striped Length	100 ft	add 0.024 for every additional 100 ft (up to 2,000 ft)
Speed Limit	35 MPH	subtract 0.02 for every 5 MPH increase (up to 60 MPH)
Auxiliary Through Lane	Not Present	subtract 0.104 if present

Another implementation method is to use a lookup table with different lane utilization factors. The values obtained from this table would be the same as the values obtained from the incremental method but would require fewer calculations. However, the values in the table are only applicable for two-lane roads without an auxiliary through lane, so the user would still need to subtract 0.125 for three-lane scenarios and 0.104 if an auxiliary through lane is present. Table 8-2 shows what this table could look like.

Table 8-2 Arterial Lane Utilization Lookup Table

		Speed Limit						
		30 mph	35 mph	40 mph	45 mph	50 mph	55 mph	60 mph
Striped Length	100 ft	0.56	0.54	0.52	0.50	0.48	0.46	0.44
	200 ft	0.58	0.56	0.54	0.52	0.50	0.48	0.46
	300 ft	0.60	0.58	0.56	0.54	0.52	0.50	0.48
	400 ft	0.63	0.61	0.59	0.57	0.55	0.53	0.51
	500 ft	0.65	0.63	0.61	0.59	0.57	0.55	0.53
	600 ft	0.68	0.66	0.64	0.62	0.60	0.58	0.56
	700 ft	0.70	0.68	0.66	0.64	0.62	0.60	0.58
	800 ft	0.72	0.70	0.68	0.66	0.64	0.62	0.60
	900 ft	0.75	0.73	0.71	0.69	0.67	0.65	0.63
	1000 ft	0.77	0.75	0.73	0.71	0.69	0.67	0.65
	1100 ft	0.80	0.78	0.76	0.74	0.72	0.70	0.68
	1200 ft	0.82	0.80	0.78	0.76	0.74	0.72	0.70
	1300 ft	0.84	0.82	0.80	0.78	0.76	0.74	0.72
	1400 ft	0.87	0.85	0.83	0.81	0.79	0.77	0.75
	1500 ft	0.89	0.87	0.85	0.83	0.81	0.79	0.77
	1600 ft	0.92	0.90	0.88	0.86	0.84	0.82	0.80
	1700 ft	0.94	0.92	0.90	0.88	0.86	0.84	0.82
	1800 ft	0.96	0.94	0.92	0.90	0.88	0.86	0.84
	1900 ft	0.99	0.97	0.95	0.93	0.91	0.89	0.87
	2000 ft	1.01	0.99	0.97	0.95	0.93	0.91	0.89

Notes:
1. Subtract 0.125 for 3-lane scenarios
2. Subtract 0.104 if auxiliary through lane is present

Alternatively, the equations and charts developed from the research could be used to calculate utilization factors. Equation 8-1 could be used for precise utilization factor calculations and Figure 8-1 could be used for rough estimations. It should be noted that the user will still need to modify the estimated utilization factor for three-lane scenarios or scenarios where an auxiliary through lane is present. Also, Table 8-3 should be considered when choosing appropriate inputs for Equation 8-1.

$$\boldsymbol{U} = 0.902 - (0.125 * \boldsymbol{N}) + (0.024 * \boldsymbol{L}) - (0.004 * \boldsymbol{S}) - (0.104 * \boldsymbol{A}) \qquad (8\text{-}1)$$

Where:

U	Utilization rate of the short lane
N	Number of lanes present at the upstream signal
L	Striped length downstream of the signal (in 100s of feet)
S	Speed limit in miles per hour
A	Presence of auxiliary through lane (1 if present, 0 if not)

Table 8-3 Thresholds for Arterial Equation Inputs

Variable	Lower Bound	Upper Bound
Striped Length (ft)	25	2,100
Speed Limit (mph)	30	60
Number of Lanes	2	3
ATL Presence	0	1

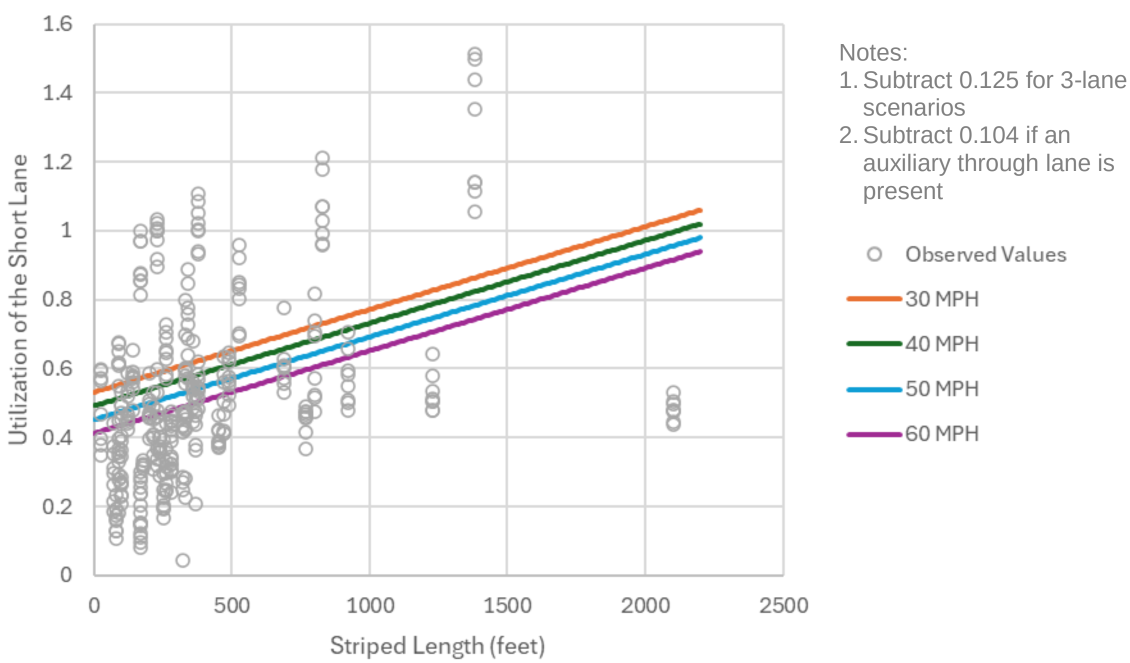


Figure 8-1 Utilization on arterials based on striped length and speed limit.

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