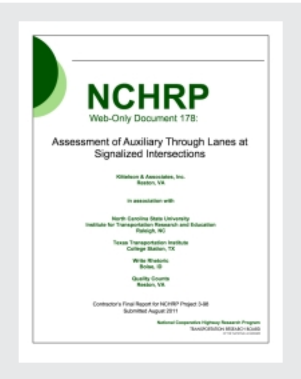
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Assessment of Auxiliary Through Lanes at Signalized Intersections

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Several additional individuals contributed to the project. Mike Alston and Brendan Lehan of North Carolina State University provided key contributions to the development of the operations and safety assessments. Séverine Maréchal and Diego Franca of Kittelson & Associates, Inc. assisted with the survey and Interim Report. Ralph Bentley and Jon Sommerville assisted with exhibits and production of the *Guidelines*. Brian Ray, Lee Rodegerdts, and Paul Ryus of Kittelson & Associates, Inc. provided review and input in the development of the *Guidelines.*

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CONTENTS

ABSTRACT

This project investigated the operational, safety, and design characteristics of Auxiliary Through Lanes (ATLs) at signalized intersections and produced guidelines for their evaluation and application. The project was conducted in two phases. Phase I included a literature review, survey, and development of concept methods for evaluating the operations, safety, and design of ATLs. Phase II included data collection, analysis, and the development of guidelines.

Operational, safety, and design data were collected at 22 ATL approaches across the United States. All of the study approaches had upstream and downstream ATLs located to the right of the continuous through lane(s) (CTLs). The study sites included a mix of continuous through lanes (either one or two) and right-turn treatments (either shared or exclusive lanes).

Statistical models were developed using the field data to predict the amount of traffic expected to use the ATL. Results from the modeling effort showed that the volume of through traffic on the approach and the through-movement demand-to-capacity ratio had the greatest influence on the amount of traffic that uses the ATL. Contradictory to previous literature reviewed as part of the project, the length of the downstream ATL was not found to have a significant influence on the amount of through traffic that uses the ATL. Separate ATL lane-use prediction models were developed for 1- and 2-CTL sites. These models were found to explain approximately 80 percent of the variability in data collected across the study sites.

A safety study was conducted by examining sixteen ATL approaches from eight intersections across the United States using a calibrated VISSIM model with FHWA's Surrogate Safety Assessment Model (SSAM). The study then attempted to validate the SSAM output by comparing the results with nine years of rear end and sideswipe collision data from each site. Ultimately, the SSAM results could not be thoroughly validated with collision data because of the low frequency of rear end and sideswipe collisions. Through simulation and field observation, the study identified several ATL design elements potentially related to conflict frequency, including congestion, ATL downstream length, and the presence of an exclusive right turn lane. In summary, the study determined that the studied ATL approaches were not unsafe as built.

ATL design characteristics were reviewed to understand practices for determining ATL design elements such as the length of the upstream and downstream ATL, pavement markings, and signing. Prior to this project there was no unified guidance identified in the literature regarding the appropriate sizing of ATLs. This project developed a theoretical approach for estimating the desired length of the downstream ATL based on two conditions. The first condition determines the downstream storage needed to enable vehicles to reach a desired merge speed when starting from a stopped (queued) position at the stop bar. The second condition provides sufficient downstream length to enable a driver in the ATL to find an acceptable gap in the CTL when approaching the signal on green with no queue present (i.e., uninterrupted flow conditions). Guidance was also developed regarding the use of signing and pavement markings on ATL approaches.

The research team produced NCHRP Report 707: *Guidelines on the Use of Auxiliary Through Lanes at Signalized Intersections* ("*Guidelines*") to assist practitioners in the evaluation and design of ATLs.

The *Guidelines* describe how practitioners can apply the ATL lane-use prediction models to predict performance measures using the Highway Capacity Manual 2010 methodology for signalized intersections. The Guidelines document safety characteristics of ATLs and provide recommendations for determining ATL design elements. operations, signing, and safety. The Guidelines and intended to supplement existing resource documents such as the AASHTO Green Book, the Highway Capacity Manual 2010, Manual of Uniform Traffic Control Devices, the Highway Safety Manual, and the FHWA Signalized Intersection Guide.

CHAPTER 1: BACKGROUND

An auxiliary through lane (ATL) is a limited length through lane added upstream and downstream of an intersection, as shown in Figure 1.

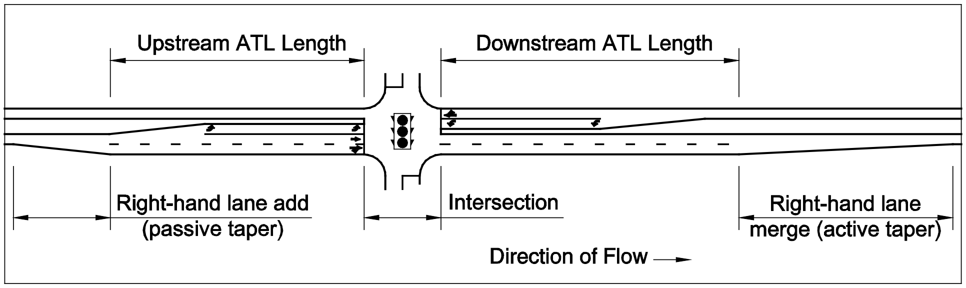


Figure 1. Typical Auxiliary Through Lane (ATL) configuration.

ATLs are typically applied as an intermediate-cost treatment to reduce recurring bottlenecks at signalized intersections. They can be applied to either the major-street or minor-street approach. When an ATL is present, through traffic is allowed to disperse across an additional through lane at the signalized intersection which increases the stop-bar capacity of the approach. This reduces delay and queuing for through vehicles. An ATL also reduces the green time required to serve the through demand on the approach, meaning the extra time can then be allocated to other movements at the intersection.

ATLs are typically applied at locations where additional through capacity is desired but construction of a continuous through lane (CTL) is not feasible. ATLs can also be applied as an interim improvement until a CTL improvement is made or can be justified. In summary, an ATL achieves a portion of the capacity benefits of a CTL for a portion of its cost and right-of-way/environmental impact.

Prior studies suggest that operational and geometric characteristics are significant factors affecting upstream lane usage, and therefore, intersection capacity. However, the conditions for their effective use and their impact on operations, safety, and the site location have yet to be documented. Thus, this research effort was needed to provide a technical assessment of their use, document their impact on operations and safety, and develop guidelines on their design and placement.

RESEARCH OBJECTIVE

The objective of this research is to provide guidelines and procedures to analyze, justify, and design auxiliary through lanes at signalized intersections. The results will assist transportation professionals in the effective and safe implementation of intersection auxiliary through lanes.

The Guidelines produced from this research effort are intended to compliment, not replace, local agency practice on intersection design and national resource documents such as:

- A Policy on Geometric Design of Highways and Streets (AASHTO Green Book) (1);
- Highway Capacity Manual 2010 (HCM) (2);
- Manual on Uniform Traffic Control Devices (MUTCD) (3);

- Highway Safety Manual (HSM) (4); and
- FHWA Signalized Intersection Guide (5).

SCOPE OF STUDY

The scope of study for the NCHRP 3-98 research effort including a description of tasks and deliverables is summarized in Table 1.

Table 1. NCHRP 3-98 scope of work.

Task	Title	Description
0	Amplified Research Plan	Revise research plan to address panel comments
1	Literature Review	Conduct a comprehensive literature review to identify previous findings and known issues related to the evaluation and design of ATLs
2	Survey	Conduct a web-based survey of transportation practitioners to identify ATL locations and current practices
3	Concept Methods and Approach	Develop a description of proposed analytical methods and plans to refine the research approach for Tasks 5 and 6
4	Interim Report	Document the results of Tasks 1-3 and present findings to Panel
5	Data Collection	Obtain data from sites identified in Tasks 2 and 3. Analyze data to quantify the impact of ATL design on operational and safety performance
6	Guidelines	Develop guidelines and procedures to analyze, justify, and design ATLs
7	Final Report	Prepare final report and document findings of the research

CHAPTER 2: RESEARCH APPROACH

INTRODUCTION

This chapter describes the research team's technical approach in terms of developing procedures for evaluating the operations, safety, and design of ATLs. It also describes the outreach activities and panel meetings that took place as part of the project.

OPERATIONS APPROACH

The research team explored two conceptual approaches for estimating the volume of traffic that is expected to use an ATL (includes both "captive" and "non-captive" vehicles): a simulation-based approach and an analytical approach, with the objective of ultimately determining the "best" model for predicting ATL use for a given application and incorporate that model into the Guidebook. Figure 2 illustrates the overall conceptual approach.

Both the simulation and analytical-based approaches consider: 1) the ATL volume that minimizes ATL delay and 2) the ATL volume that minimizes system delay. As shown in the results from preliminary testing, these volumes are not equal.

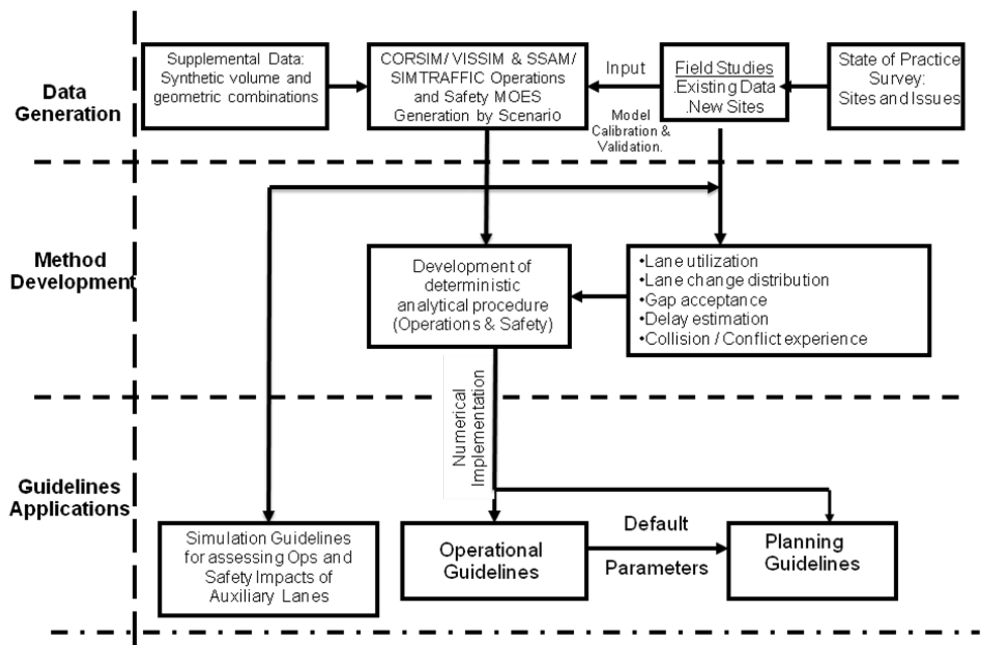


Figure 2. Conceptual framework and approach.

The basic premise of the operational conceptual models described herein is that drivers' use of the ATL (or lane utilization factor) is governed by their perceived travel time savings gained by selecting the ATL path rather than staying in the continuous through lane(s), or CTL. Of course, this model applies to choice drivers only. For those drivers who must use the ATL to reach their destination, their volume should be measured or initially estimated and treated as if they were just like right-turners in a shared through lane. The fact that there are captive users will/should affect the degree of "attractiveness" of the ATL to the choice users.

The team conducted extensive simulations in VISSIM to track the travel time and delay savings under various operational scenarios. The key variables studied included the ATL length (levels used were 200, 500, and 800 feet), congestion level (approach volume-to-capacity [v/c] = 1.0, 1.25 and 1.50 based on the capacity of the continuous lane(s) only), and ATL utilization (tested at nominal levels of 10 percent, 25 percent, 35 percent and 50 percent). The simulation analysis was analyzed to determine the operational thresholds and characteristics that impacted the use of the ATL in simulation. These findings were used to inform Task 5 Data Collection Activities and the development of the analytical method included in the Guidelines. The findings from the simulation analysis were also instrumental in developing the methodology for evaluating ATLs using microsimulation which is described in the Guidelines.

The analytical approach for this step is summarized computationally in Figure 4. It describes an iterative procedure that is intended to estimate the likely level of ATL utilization based on criteria of delay optimality to ATL and all users, respectively, as described in the simulation approach. It begins with an initial assumption of ATL utilization (the most logical would be an equal volume allocation to all through and auxiliary lanes), which means that for a through-lane group with N lanes, the initial ATL utilization would be 100/N %. Subsequently, all three delay components are calculated as described above and both ATL users and overall through delays are estimated, and stored. A second run is then performed by reducing ATL % by a fixed amount (TBD), and recalculating. Finally, the delay gradient is computed and if it indicates that delays are moving in the wrong direction, the procedure ends, and the current solution is considered to be optimal. If, on the other hand performance improves with a reduction in ATL use, another iteration is carried out, until no further delay improvement can be made, at which point the procedure terminates.

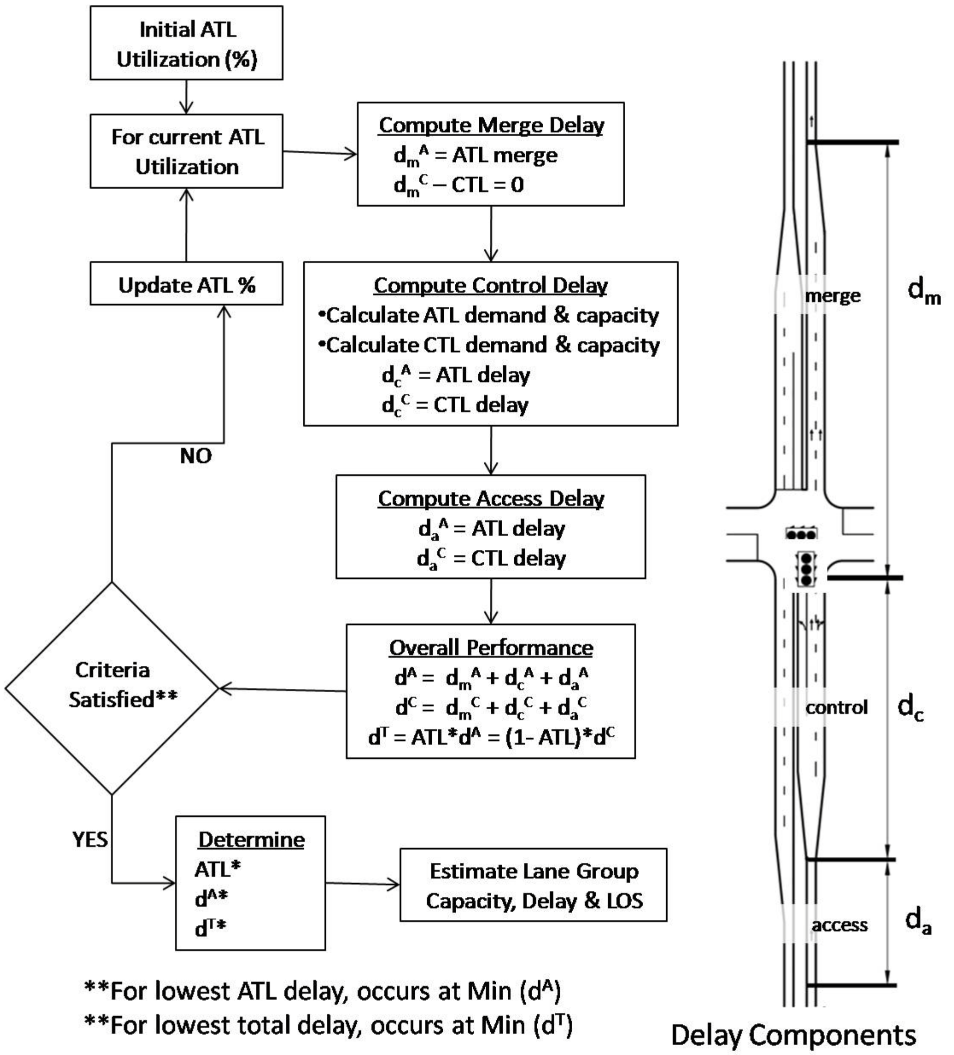


Figure 3. Analytical model development procedure.

Based on the optimal ATL % that minimizes delay for either ATL or all users, the actual throughput is computed. It should be noted that while the latter ATL utilization value may be optimistic, it does provide a benchmark by which to gauge the potential for operational improvement in lane utilization and approach capacity.

SAFETY APPROACH

Three basic approaches were explored for the safety aspects of this research: before & after analysis, collision modeling, and simulation with VISSIM (9) and the Surrogate Safety Assessment Model (SSAM, 10). A before-and-after analysis would examine whether changes in collision frequencies and/or severities occurred after conventional intersections were fitted with ATLs. This method is useful because it can directly relate an increase or decrease in crash rate to the presence of an ATL. However, such conversions are scarce. In addition, ATL installations are often bundled with other intersection improvements, making isolation of the effect of the ATL challenging. Also, collision, traffic volume, and other data from prior to improvements are not always available. In addition, a before-and-after analysis would be difficult because of the wide variability among existing and improved intersection designs. As the effort to develop good crash modification factors (CMFs) for the Highway Safety Manual has shown, high-quality before &

after studies are scarce and difficult even on "easier" safety countermeasures like signal and turn-lane installation; a before & after effort for ATLs would be fraught with difficulty.

The second possible method of safety analysis for this project is to construct a collision model, which is an equation that would predict crash frequency based on traffic volume, intersection geometry, and other inputs. Researchers have made great strides in safety modeling in the past few years, especially with new statistical techniques and higher quality data available. However, the effort to develop a safety model depends upon collection of a large database: most models are based on data from at least 50 sites with hundreds of collision reports. Moreover, a huge number of important variables could affect collision frequency for the signalized intersections within the scope of this project. These variables include turning volumes, lane configurations, and driveway configurations in addition to all of the ATL-related factors discussed previously. As we have demonstrated in the operational analysis, even small changes in v/c, for example, can produce large changes in ATL use and, presumably, the collisions associated with ATL use, so the chances of a good model fit are not high. Good collision-prediction models for signalized intersections are scarce in the literature for these reasons. The study team believes that the effort to develop a good collision model for signalized intersections that included ATL-related variables would be a massive effort far exceeding the resources available in this project.

The final (and most feasible) option to develop the safety relationships and guidelines desired in this project is to use SSAM to estimate the number of collisions that will occur at ATL intersections with various configurations. Developed by the Federal Highway Administration for use in conjunction with microscopic simulation software, SSAM allows analysts to predict collisions based on the number of vehicle-to-vehicle conflicts counted within the simulation network. SSAM is useful because it works in conjunction with VISSIM, which is already being applied as part of this research effort. In addition, the wide availability of VISSIM allows analysts across the world to create their own customized simulations to examine their own ATL design alternatives. If the panel will forgive the metaphor, by using VISSIM and SSAM we will teach designers to fish rather than just handing them a fish.

Figure 5 details the process explored for determining ATL safety guidelines. The first step in this process is field data collection. Collision data taken from an extended period (as long as, say, ten years) should be collected for each intersection if available. This will be helpful in boosting the sample size. The collision data will only include collision types that were related to the ATL, such as sideswipe, merging, diverging, and rear end collisions. Collision data will be segregated by time (up to the hour) and the area relative to the intersection that the collisions took place. The needed data at each test intersection includes turning-movement volume by hour for peak and off-peak hours, truck and pedestrian volumes, driveway activity levels in the ATL zones, intersection geometry, and signal timing.

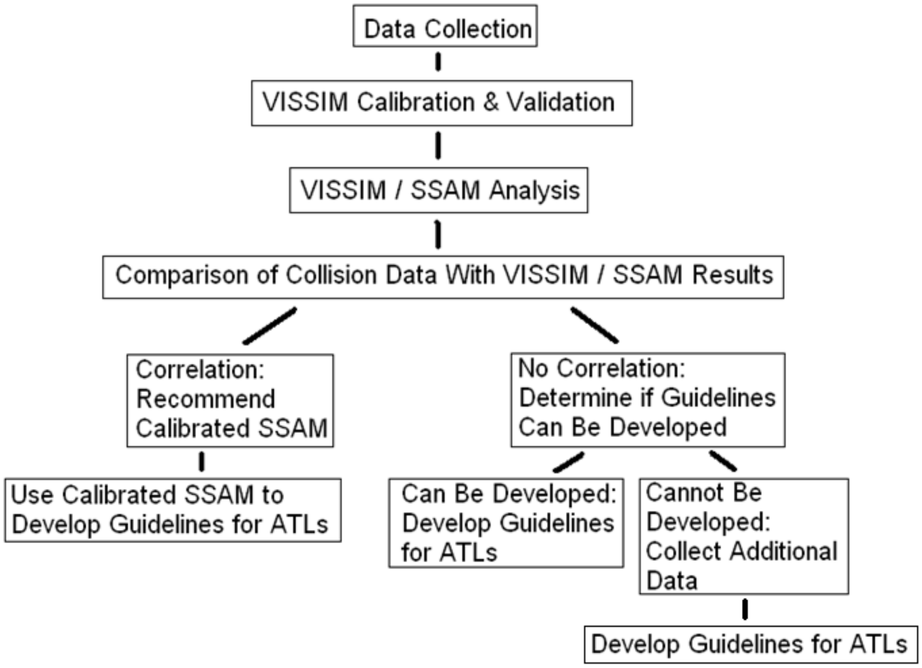


Figure 4. Procedure for developing guidelines for ATL design.

After data collection, the next step is to calibrate and validate VISSIM for ATLs. Once VISSIM is calibrated, SSAM is applied to simulate all test intersections. The next critical step in the process is a comparison of SSAM output to collision data from each test intersection. If a strong correlation exists, SSAM can be used to predict the safety performance of ATL intersections. If not, guidelines can be developed from available collision data.

DESIGN APPROACH

The research team developed a design approach that considers geometric design, pavement marking, and signing needs on a segment basis. The design approach is intended to encourage designers to consider design needs from the perspective of the motorists as they travel from the upstream approach to the ATL through and beyond the downstream merge. The description of each segment and its design-related needs are as follows:

- **Approaching ATL**: This segment informs the approaching driver of the start of an additional through lane at the next intersection. Supplementing signage and pavement markings should encourage drivers to use all intersection through lanes.
- **Approaching Signal**: Traveling along the through lanes [CTL(s) and ATL] approaching the signal, the driver should be reminded that the current lane configuration is continuing through the intersection. Supplementing side-mounted and/or overhead signage and pavement markings should encourage drivers to treat both lanes as through lanes.
- **Leaving Intersection**: Immediately downstream of the intersection the driver should be reassured that the through lanes are continuing to some distance beyond the intersection.

- **Merge at End of ATL**: Towards the termination of the ATL and the taper section the driver needs to know that the extra lane is ending and that appropriate merge behavior is encouraged.

The most common question asked by practitioners and Panelists throughout the research effort was: how much length is needed for the downstream ATL? The research team examined national resource documents, published research, and local highway design guidelines and found limited guidance on how to determine the length of the downstream ATL. Thus, the team evaluated the field data collected at the 22 study ATL sites and developed an analytical approach for estimating the required length needed for the downstream ATL under ideal conditions. The approach requires the calculation of two factors: DSL_1 (interrupted flow condition) and DSL_2 (uninterrupted flow condition). The greater of the two values determines the minimum downstream length.

DSL_1 is defined as the downstream ATL length needed for a vehicle in queue in the ATL to reach a desired (design) speed at a given acceleration rate when starting from a stopped position. Thus, the downstream ATL storage length enables motorists to reach a desired speed prior to merging. This minimum design parameter assumes ideal conditions. The presence of non-ideal conditions such as driveways, horizontal or vertical curves, obstructed sight distance, etc. will likely require longer downstream ATL storage lengths and will be left to the engineer’s judgment to determine.

DSL_2 is defined as the downstream ATL length needed to enable a driver in the ATL to find an acceptable gap in the CTL when approaching the signal on green when no queue is present (similar to an uninterrupted flow condition).

OUTREACH ACTIVITIES

A measure of success of this research effort will be the degree in which the final products of this research (the Guidelines), are applied in the day-to-day work of transportation practitioners. To increase awareness of this research effort the team reached out on multiple occasions throughout the course of the project to request feedback and to disseminate early findings.

The first outreach activity occurred as part of the User Survey in Task 2. The survey was distributed through the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Traffic Engineering (SCOTE), AASHTO Standing Committee on Research (SCOR) Research Advisory Committee (RAC), and Transportation Research Board (TRB) committees. A total of 42 out of 249 agency contacts responded to the survey for a response rate of 17 percent.

Research findings from the project were presented at the TRB Annual Meetings in 2010 and 2011. This included a podium presentation and paper publication (6); a poster session (7); a presentation to the Signalized Intersection Subcommittee of the Highway Capacity and Quality of Service Committee (HCQS); and a podium session on NCHRP progress updates (8).

These outreach efforts helped inform the user community of on-going work. It also enabled feedback which helped to inform and shape the Guidelines.

PANEL COORDINATION

Four meetings were held with the research panel throughout the course of the project to keep the panel informed of progress and enable feedback at key points in the project:

- An in-person meeting in Irvine, California in September 2009 to discuss the interim report;
- An in-person meeting at the TRB annual meeting in Washington, D.C. in January 2010 to discuss initial findings from the data collection effort;
- A webinar in September 2010 to present results on the operational and safety analysis methods; and
- An in-person meeting at the Keck Center in Washington, D.C. in April 2011 to review the draft Guidelines.

Agendas were circulated prior to each panel meeting and notes were recorded and distributed shortly following the meeting. The research team responded in writing to all questions and comments posed by the panel following the meetings.

CHAPTER 3: LITERATURE REVIEW

This chapter summarizes the results from a literature review that was conducted to identify issues related to the operations, safety, and design of ATLs. Specifically, the literature review:

- Provides a summary of capacity and operational effects of ATLs;
- Focuses on the safety effects of ATLs paying particular attention to the safety analysis methodology proposed in this study using the Federal Highway Administration's (FHWAs) Surrogate Safety Assessment Model (SSAM) (10); and
- Discusses design effects of ATLs.

Results from the literature review revealed a lack of any unified guidelines that account for operational effects of specific design configurations of auxiliary lanes at signalized intersections. Nevertheless, the literature reviewed speaks to many of the design and operational parameters that impact capacity and operations of ATLs. *Note that this literature review was completed in May 2009 and does not reflect publications since that time.*

LANE UTILIZATION

The US Highway Capacity Manual (HCM) 2010 suggests the use of the lane utilization factor, f_{LU}, to estimate capacity and LOS at signalized intersections (2). The HCM values, however, appear to consistently overestimate the level of "evenness" in lane utilization, even in the absence of any auxiliary lanes at the intersection. This is evident from the findings of multiple studies (12, 13, 14, 15, 16). Another shortcoming of the HCM method is the lack of sensitivity of the design features of the auxiliary lanes on the ultimate capacity of intersection approach, which is typically accounted for through the lane utilization factor. Specifically, the HCM advises the analyst to realize that short additions or drops before and after intersections, respectively, may not function as full through lanes. It also informs the reader that the lane utilization factor with respect to auxiliary lanes approaches a value of 1 as demand approaches capacity. Beyond this advice, the HCM falls far short from providing any quantitative impacts other than those for a full lane. The HCM only offers a rudimentary estimate for a lane utilization factor (with 0 being unused and 1 being fully used). This is shown in Exhibit 18-30 in the HCM 2000, duplicated below as Exhibit 2-5.

Table 2. Default lane utilization factors in the HCM 2000.

Lane Group Movements	No. of lanes in lane group	Percent of traffic in most heavily traveled lane	Lane utilization adjustment factor (f_{LU})
Through or Shared	1	100.0	1.000
	2	52.5	0.952
	3	36.7	0.908
Exclusive left turn	1	100.0	1.000
	2	51.5	0.971
Exclusive right turn	1	100.0	1.000
	2	56.5	0.885

The aaSIDRA intersection model (17) offers some means of estimating lane underutilization as a function of the length of a *downstream* short lane (i.e., at the point when traffic in the auxiliary lane must merge into the continuous lane). While this model represents a marked improvement over the HCM it still suffers from certain drawbacks, most notably the lack of sensitivity to the auxiliary lane configuration, the actual signal timing at the intersection, and, more importantly, the prevailing congestion level and amount of "captive" users in the auxiliary lane. Captive users include right-turning drivers at the signalized intersection from a shared right/through auxiliary lane, and through vehicles that are planning to turn right just downstream of the intersection at nearby driveways. It is also unclear whether aaSIDRA has been calibrated with field data, particularly for US driving conditions.

Probably the most comprehensive study of lane utilization in conjunction with a downstream short lane was that by Lee et al. (12). That study consistently showed more-even lane utilization as overall traffic flow increased, irrespective of the downstream lane design. Furthermore, Lee et al. identified 15 candidate factors that appear to have an effect on the level of lane utilization:

(1) Lane drop type (physical drop vs. lane trap)

(2) Downstream short-lane length

(3) Taper length

(4) Right-turn volume in shared lane

(5) Heavy-vehicle percentage

(6) Number of signs indicating the presence of a lane drop

(7) Number of markings

(8) Location of the first lane-drop signage

(9) Density of driveways on the upstream left side

(10) Density of driveways on the upstream right side

(11) Density of driveways on the downstream left side

(12) Density of driveways on the downstream right side

(13) Existence of upstream mid-block left-turning accessibility

(14) Existence of downstream mid-block left-turning accessibility

(15) Average lane volume

Lee, et al. based their results on a large database collected from a diverse set of sites located in North Carolina.

A good corollary to Lee's work regarding the effect of downstream short lanes is that of Kikuchi (18), who explored the effect of upstream auxiliary lane length on lane utilization. Although he examined the data for an approach configuration that had one left-, one through, and one exclusive right-turn lane, the same predicament applied to auxiliary through lanes: they may be underutilized and thus not filled to capacity if the driver does not have ready access to the choice lanes. Kikuchi states that the driver's choice can be hindered by two factors: (a) overflow or (b) blockage of the intersection. Kikuchi defines overflow as the situation in which a desired lane is fully occupied and, therefore, cannot accept any more vehicles; blockage, on the other hand, occurs when the auxiliary lane has available space but is inaccessible to the driver because of vehicle overflow in the contiguous lane. His work included the use of the VISSIM micro-simulation model (9) with different approach volumes to estimate the recommended lane lengths as a function of turning-movement volumes. This model appears to be readily modifiable to incorporate auxiliary through lanes. Kikuchi theorizes that "many operational problems of transportation fall into the category of a single queue leading to multiple service channels, and the congestion at the neck of the service channels prevents the units from reaching the desired channel."

A paper by Steyn and Vandehey (19) provided the results from a case study in Oregon in which an auxiliary lane was developed to receive two left-turn lanes. The auxiliary lane was designed with a 100-foot downstream length and a 150-foot taper. The authors anticipated that the short downstream distance would produce disparate lane utilization on the upstream end, but, surprisingly, the results revealed very even lane utilization. The authors hypothesized, in concert with similar findings by Feldblum (20), that proper management of short lanes with alternative merge control (e.g., "zipper" merge) may result in superior lane utilization. They also concluded that as traffic volume increases, so too does the balance of lane utilization—a finding consistent with that of Lee et al (12).

Hurley's (13, 21) research encompassed both auxiliary through lanes and double left-turn lanes with downstream lane drops. He approached the double left-turn lanes by mathematically modeling lane utilization based on choice and captive users. His results agreed with the findings of Lee, et al. (12) on a few of the factors that affect auxiliary through lane use. He listed the following characteristics as influencing factors:

1. Through-lane flow rate
2. Right-turn movements off the facility (within 500 feet)
3. Total left turns off the facility downstream of the intersection
4. Downstream auxiliary lane length
5. The size of the given urban area
6. The existence of left-turn bays or two-way-left-turn-lanes downstream

Hurley's data—collected at multiple sites with full lanes upstream and lane drops downstream—showed an average heavy-lane use of 64% at flow rates greater than 700 vph, compared to the HCM value of 52.5%. However, as Lee et al pointed out (**12**), Hurley's study sample size was small and did not include three-lanes-to-two lane drops. Thus, his input is useful but not enough to serve as the basis for general guideline development.

McCoy and Tobin (22) collected data at five intersections with auxiliary through lanes. They concluded that the auxiliary lane length and green time had impacts on its utilization rate; while taper length, lane congestion in the continuous lane, and right turn traffic volume had no substantial impact on auxiliary lane use, a somewhat surprising finding that is inconsistent with other studies. Meanwhile, the findings of Tarawneh (14) at eight sites were consistent with earlier ones in that auxiliary lane use increased with lane length and with overall volume levels. The study also revealed that through drivers appeared to have avoided the use of the auxiliary lane when right-turning traffic beyond the intersection increased. This is an area that deserves further exploration and improved definition.

Last, the Highway Safety Manual (HSM, 4) noted that restricting driveways to locations away from the intersection functionally allows vehicles more time and space to turn or merge. The HSM hypothesized that lane utilization is likely to increase with more-readily available lane movement.

ATL'S AND DELAY

The HCM 2010 (2) computes control delay for lane groups without considering auxiliary lane length or its other design attributes. However, the HCM does recognize the effect of lane underutilization on capacity and delay by adjusting the lane group capacity downward to account for lane utilization. Since the HCM control delay is a direct function of the lane group v/c ratio, and "c" (capacity) reflects the effect of lane utilization, then some mechanism exists within the HCM construct to incorporate, in an indirect fashion, the effect of ATL configuration on utilization and delay.

Several researchers have taken up the topic of delay with auxiliary lanes in recent years. Similar to Kikuchi's approach (18), micro-simulation traffic models have been used to generate data where the analyst is able to control facility and traffic parameters, and can directly estimate measures of effectiveness associated with various operational scenarios. From a traffic–flow-theory perspective, the following algorithms must be carefully assessed if a simulation model is to be used for the purpose of developing operational guidelines for auxiliary lanes: (a) upstream lane selection; (b) downstream platoon dispersion, and (c) mandatory and discretionary lane changing. As an example, Shen applied the CORSIM model (23) and found that as the downstream auxiliary lane length increased, delay decreased. However, Shen's model was not calibrated or validated with any field data, which undermines the validity of the findings.

An interesting application by Lee et al. (12) was the development of a set of CORSIM models to investigate the impact of lane utilization and downstream lane length on delay, for various demand volume levels and lane drop configurations. They found, like Shen, that delay began to spike at low v/c ratios when the downstream auxiliary lane became shorter.

The VISSIM (9) user manual describes a general methodology for lane changes, which is paramount to auxiliary through lane utilization. It states, in part that "….*if overtaking is possible, any vehicle with a higher desired speed than its current travel speed is checking for the opportunity to pass- without endangering other vehicles, of course*." There are two forms of lane changes simulated in VISSIM, necessary lane changes and free lane changes. Necessary lane changes result from the need to stay on a particular route and thus would create an auxiliary through lane's "captive" users. This could represent the path of right-turning vehicles that must enter an auxiliary shared through right lane. Free lane changes, on the other hand, result from a desire to accelerate and could create "choice" users on lanes. VISSIM, however, does not allow the adjustment of the level of "aggressiveness" with these lane changes. However, in using routing decisions as opposed to using directional decisions, the simulation should provide an adequate portrayal of lane usage, irrespective of factors that VISSIM cannot simulate.

Lee et al. (12) explored the effects of altering the green time on delay at intersections with auxiliary lanes. They concluded that, similar to the mechanism by which lane utilization affects delay in the HCM, unbalanced lane volumes that resulted from the lane drop negatively affect delay. However, they also found that at the extreme points of low and high traffic volume, providing additional green time had an insignificant effect on delay. In fact, control delay due to the lane drop eclipsed the benefit that providing the extra green time had on delay. Buckley (16) hypothesized (in a paper that focused mostly on downstream design) that, to avoid unnecessary delay, minimum cycle length, green time, and the overall capacity of the downstream road segment must be adequately configured.

ATL'S AND SAFETY

Clearly safety should play a role in the development of guidelines on the use of auxiliary lanes through signalized intersections. It is likely that auxiliary lanes would decrease the safety of an intersection when compared to the same intersection without auxiliary lanes, due to the potential for additional sideswipe-type collisions. One could make the case that a reduction in congestion due to auxiliary lanes could mean that some rear-end and other collisions would be prevented. However, the collisions prevented would not likely equal or exceed the collisions caused. In addition, the collisions prevented would most likely be less severe on average, so the expectation of a net negative safety impact is not unreasonable.

Estimating the magnitude of the safety impact is the key issue. On auxiliary lanes, the RFP states that, "*their impact on...safety…has yet to be determined*." This is largely true. The research reported by Lee, et al. (12) is the only paper we know of on this issue that reported empirical results. That analysis of 94 lane drop sites (almost exclusively of Types B and C) in North Carolina sheds a little light on collision patterns that one might expect with auxiliary lanes. Exhibit 2-6 provides a sample of the results, for downstream roadway segments. The notation "2LR" and "2LS" at the top of the graphs refers to different configurations of lane drops which occur for dual LT lanes at on-ramps (2LR) and surface streets (2LS). The results from that effort revealed weak relationships between the downstream length of the lane being dropped (i.e., "short lane") and the collision rate. The type of lane drop evidently affects collision rate as well, based on Figure 5.

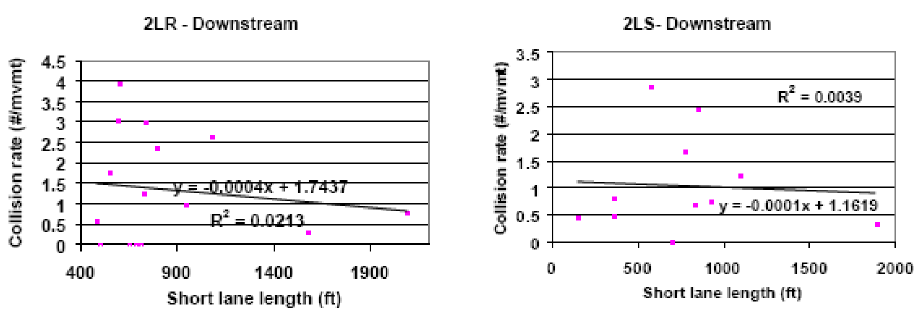


Figure 5. Downstream short lane length vs. collision rate (12).

The evidence in Figure 5 and others like it in the previous research is ultimately unconvincing for the purposes of this research, though. The relationships are weak, the models are unsophisticated (considering only one variable at a time, for instance), and the lane drop configurations are different from the auxiliary lanes of interest in this work (Type A in particular).

Bared's 2008 evaluation of the Surrogate Safety Assessment Model (10) informs the reader how, and how reliably, SSAM assesses safety as a modeling program. SSAM is a program recently developed by the FHWA that analyzes output from microscopic traffic simulation packages like VISSIM to provide quantitative safety measures. SSAM calculates the following surrogate safety measures:

- Minimum time-to-collision (TTC)
- Minimum post-encroachment (PET)
- Initial deceleration rate (MaxD)
- Maximum speed (MaxS)
- Maximum speed differential (DeltaS)
- Classification as lane-change, rear-end, or path-crossing event type
- Vehicle velocity change had the event proceeded to a crash (DeltaV)

In a recent evaluation of SSAM (10), the authors point to unusual lane-changing behavior among simulated drivers generating quite a few collisions. The authors went through a three-step process to validate SSAM including theoretical validation, field validation, and sensitivity analysis. In validating SSAM, the authors found good overall correlation between SSAM-generated conflicts and field collisions, with the exception of the underrepresentation of path-crossing maneuver conflicts. Although the correlation is not as close as that of the volume-based prediction models, SSAM would likely be a good starting point for our Task 3.

Furthermore, SSAM discerned geometric adjustments and subsequent changes in safety in otherwise similar situations, which makes it useful in evaluating the safety of traffic situations that do not yet exist. However, the report also noted several areas that need improvement in SSAM:

- **Improvement in driver behavior**. There were some inherently unrealistic driving actions that resulted in collisions that need correcting.
- **Development of a composite "safety index."** There needs to be a means by which to account for trade-off of surrogate safety measures.
- **Study of the underlying nature of conflicts in real-world data.** The SSAM-generated conflicts are, on the whole, less severe than recorded collisions, but it is not easily discernible whether the error is SSAM's or the flaws of recorded data.
- **Collection of real vehicle trajectory data sets.** Video recording must improve to allow recording of vehicle trajectory data on a more microscopic level.
- **Investigation of conflict classification criteria.** There is a blurry line between what constitutes a "lane change" conflict versus a "rear end" conflict, as the impetus of the conflict may not always be clear. This must be better defined.

Figure 6 shows a screenshot from SSAM illustrating a lane-change conflict.

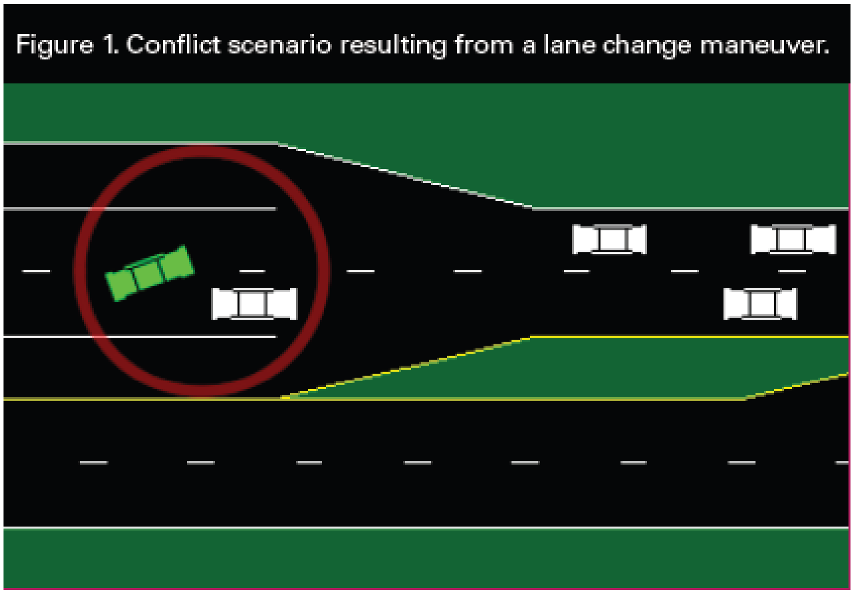


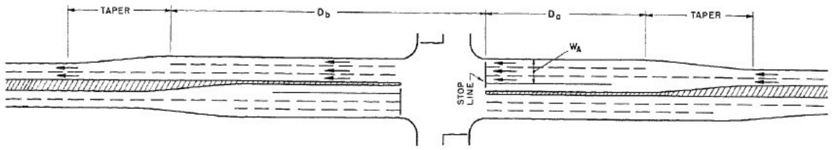
Figure 6. SSAM modeling of lane change conflict (20).

The *Highway Safety Manual* (4) advises on driveway location within the intersection functional area. The text suggests that restriction of driveways reduces conflicts and thus leads to a reduction in related rear-end and angle crashes. Furthermore, the *Manual* specifically states that "it is generally accepted that access points located within 250-ft upstream or downstream of an intersection are undesirable." The *HSM* says nothing directly on the subject of auxiliary through lanes at intersections.

Steyn's paper (19) suggests that proper management of short auxiliary lanes and alternative merge control (e.g., "zipper" merge) may improve the safety of a particular lane drop. It also pointed out that longer receiving lanes result in higher-speed merges and, potentially, the risk of collisions in the merge area.

ATL DESIGN EFFECTS

The AASHTO *Policy on Geometric Design of Highways and Streets* (i.e., the "Green Book", 1) specifies that auxiliary lanes can be used to increase capacity and improve safety at an intersection. The design guidelines recommend that an auxiliary lane be at least 10 feet wide, preferably equal to the width of other continuous lanes. The total length of an auxiliary lane (upstream and downstream) should be the sum of that which allows for entering taper, deceleration length, and storage length, as detailed in the next few sections. The sections that follow discuss design elements on the approach and exit sides of the intersection. Figure 7 depicts these elements.



D_a= length from end of taper to stop bar

W_A= Total through width at intersection, including shared lanes

D_b = Length measured from the stop bar to the beginning of downstream taper (ft)

Figure 7. Auxiliary Through Lane schematic by Leisch (24).

Elements on the Approach to the Intersection

Three functionalities need to be met in the design of the upstream auxiliary lane. These include: shifting, storage, and deceleration. Lane shifting is usually handled using an appropriate taper length to access the auxiliary lane at the design speed. The storage function is intended to accommodate the largest (or 95th percentile) queue expected, while the deceleration function should enable vehicles to decelerate from the design speed to come to a stop. The Green Book recommends designing an auxiliary through lane with provision for deceleration clear of the through lanes. Its recommendations can be found in Table 3, based on design speeds.

Table 3. AASHTO Green Book recommended deceleration lanes lengths.

Design Speed (mph)	Deceleration Length (ft)
30	170
40	275
45	340
50	410
55	485

However, the Green Book also states that designing for the full deceleration length as indicated above may be impractical, thus forcing the designer to use storage length as the criterion instead. Storage in an auxiliary through lane should be long enough to provide for the amount of vehicles that will accumulate during a critical period. This length is taken to be a function of the signal cycle and phasing, and the Green Book recommends allowing for 1.5 to 2 times the normal (or average) amount of vehicles stored. For example, using HCM notation, it can be shown that the average back of queue (in vehicles) during a signal cycle can be estimated as:

$$N_b = \frac{(C-g)}{1-v/S}$$

Where C= cycle length, g= effective green time, v is the volume in the auxiliary lane (captive and non-captive users), and S is the auxiliary lane saturation flow rate. Multiplying N_b by 2 would give a rough approximation of the size of the 95th percentile queue. Multiplying again by the average vehicle spacing at stop (typically 15-20 ft) would yield the required storage length.

With respect to taper lengths, the Green Book submits that short tapers are ideal for urban deceleration lanes because of the slow operating speeds at peak conditions. For straight-line tapers, the book recommends an 8:1 longitudinal to transverse (L:T) ratio for speeds up to 30 mph and 15:1 L:T for speeds up to 50 mph.

Perhaps the most cited paper on auxiliary lane design in the literature is that by Leisch in *Public Roads* (24). Much of what he concluded is either still the standard bearer for design or constitutes the basis for current designs. He recommended the design parameters in Table 4 for upstream deceleration and taper lengths.

Table 4. Upstream design elements recommended by Leisch (24).

Deceleration		**Storage**	**Taper (feet)**
Design Speed (mph)	**Da (feet)**		
40	150	Divide approach volume by number of lanes in W_A Use volume per lane, find D_2 = Da on desirable scale (refer to Exhibit 2-11)	175
50	200		225
60	250		275

The required upstream auxiliary lane length in this application must be the combination of taper and either deceleration or storage lengths. The designer should choose the larger of deceleration or storage length but that value should be no less than 200 feet. A representative nomograph of Leisch's approach for determining the appropriate auxiliary lane length (in this case an exclusive turn lane) based on the storage functionality is shown in Figure 8.

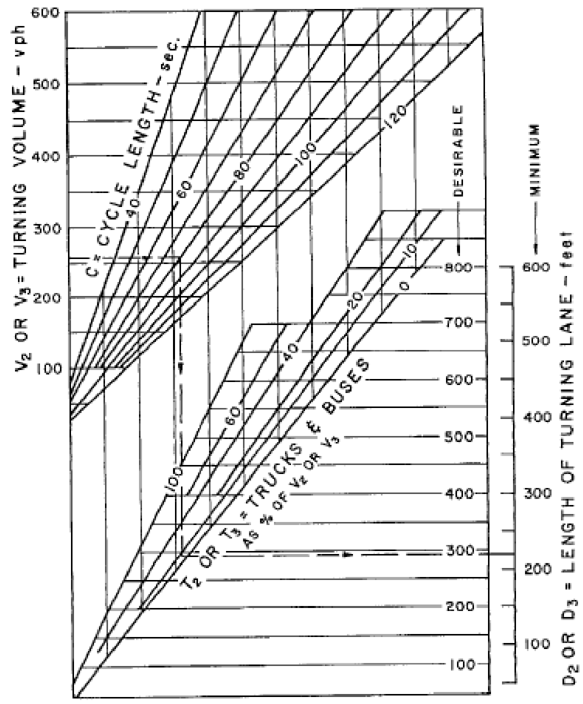


Figure 8. Length of right- or left-turn lane recommended by Leisch (24).

Elements Downstream of the Intersection

Similar to the upstream elements, the design components downstream of the intersection must satisfy the functionalities of merging, acceleration, and storage. Leisch's paper in *Public Roads* (19) also dealt with the downstream design elements. Table 5 shows his recommended *minimum* downstream lengths for ATLs based on the stated functionalities.

Table 5. Downstream design elements recommended by Leisch (24).

Acceleration		Merging	Taper (feet)
Design Speed (mph)	**Db (feet)**		
40	200	D_b = 12 * G	200
50	525		250
60	900		300

The downstream acceleration length is to be the sum of the required taper feet and the required acceleration or merging length. The designer is to pick the larger of the acceleration and merging length values but not less than 300 feet. In the context of the recommended merging length, the function is linear with the through green time, indicating that the higher the traffic volume (and thus the green time), the longer the storage that is needed for merging traffic. It is clear that for most modern arterials the merging requirement is likely to govern, with G values ranging anywhere from 40-100 seconds, far above the acceleration requirements at speeds 30-45 mph.

The work by Buckley et al. (15) represents perhaps the most important recent contribution to providing a theoretical foundation for the design of the receiving lane, by further elaborating on the downstream lane length requirements 40 years after Leisch's work was published. Using shock–wave-theory concepts due to the capacity reduction at the downstream lane drop, the authors determined the minimum length of the receiving lane as one that can accommodate the high density flow that occurs during the green phase. Based on a simplified triangular speed-density diagram used in the cell transmission model by Daganzo (25), an analytical solution to the length of the receiving lane resulted in the following formula:

$$L_{\min} = \frac{1.47 S_i G_{\max}}{K_j}$$

Where S_i is the saturation flow rate for all departing through traffic, G_{max} is the maximum green (pre-timed or actuated), and K_j is the jam density in the wider downstream section (across all lanes). By using default values for a two to one lane section with values of S_i=3,800 vph and K_i= 400 veh/mi, then L_{min} ~ 14 G which is quite close to the value published by Leisch. The reader should note, however, that Buckley's length is strictly measured from the edge of the cross street, not from the upstream stop line as assumed by Leisch. Finally, Buckley also estimated the minimum required cycle length that can prevent the shock wave experienced in one cycle from interfering with departures in the next cycle.

On that basis, he derived an analytical equation for cycle length as follows:

$$C_{\min} = \frac{G_{\max} S_i}{c_j}$$

in which c_j is the capacity of the *reduced* downstream segment capacity at the lane drop (veh/hr). This work shows the most potential for inclusion or modification in our research since it incorporates both geometric and signal aspects of the auxiliary through lane design.

The Manual on Uniform Traffic Control Devices (MUTCD) (21) provides interesting advice on the design of downstream taper lengths, allowing that "*longer tapers are not necessarily better than shorter tapers . . . because extended tapers tend to encourage sluggish operation and to encourage drivers to delay lane changes unnecessarily.*" Table 6 provides guidance on what the MUTCD describes as a merging taper.

Table 6. MUTCD recommended merging taper length (3).

Speed (S)	Taper Length (ft)
40 mph or less	$L = W*S^2 / 60$
45 mph or greater	L = W * S

[2]Where: W is width offset in the taper in ft and S is the speed in mph.

In Table 6, the separate models based on speed are advised because drivers need substantially less distance to accelerate to the proper merge speed at areas of lower speed control.

Feldblum's research for the Connecticut DOT (20) investigated the use of customized signage downstream of an ATL. The sign depicted in Figure 9 was implemented in lieu of the MUTCD W4-2 "Lane Ends" signs in spots beyond an intersection with an auxiliary lane drop.

Figure 9. Alternate merge sign downstream of ATL.

In their findings, the authors stated that, "After placement of this sign, the number of desirable merges, with no visible change in speed from any of the merging vehicles, increased from 56 percent to 66 percent. The number of undesirable merges decreased from 9 percent to 5 percent." They go on to recommend the use of this sign at lane drop situations and the adoption of this sign into the MUTCD. Although the data collection and statistical analysis were not rigorous, signing strategies deserve further consideration for the design of and efficient use of ATLs.

CONCLUSIONS

This literature review focused on the survey of the literature on the operational, design and safety properties of auxiliary through lanes (ATL) at signalized intersections. It is clear from the literature that no theoretical basis currently exists for the estimation of a lane utilization model for the approach side of the ATL. Such a model or variation thereof is a prerequisite to developing design guidelines for the length of the upstream and downstream ATL. There are similar approaches that have been used for estimating turn pocket lengths that can be adopted, but without a proper estimation of the demand on the auxiliary lane, these will not be very useful.

On the safety side, the findings were similar. Previous work in North Carolina remains the most extensive source of collision data at ATLs, although their crash modeling results were disappointing. The SSAM approach developed for FHWA, which uses simulated conflicts to gauge the safety effectiveness of operational treatments, appears to be the logical choice from our review of the literature, although questions remain on whether SSAM results will correlate well with field crash data. Part of the problem is of course the difficulty in attributing a specific crash (or crash avoidance) occurrence to the presence or absence of an ATL. Until crash reporting becomes more accurate and sophisticated, we remain convinced that SSAM is the most effective tool for the safety analysis.

On the ATL design side, our literature was able to pinpoint the critical design functionalities of ATLs, which include lane shifting, storage and acceleration/deceleration. These are repeated in most if not all sources and design guides. Moreover, the literature and research by team members speaks of innovative merging implementation strategies (such as the zipper merge) which appear to induce more even lane utilization, although the sample sizes in those studies were obviously limited.

CHAPTER 4: WEB-BASED SURVEY

This chapter summarizes the results from a web-based survey that was conducted to identify the location of signalized intersections with ATLs across the United States and to gather information, data, and experiences of agency staff on the performance of ATLs. The online survey included the following features:

- Examples of different types of ATL configurations;
- A Google Maps interface that allowed respondents to geographically locate an ATL site by "pinning" it on the interactive map; and
- A download tool to automatically export the raw survey results into an Excel spreadsheet along with longitude/latitude data for each identified ATL.

The survey was distributed through the American Association of State Highway and Transportation Officials (AASHTO) Subcommittee on Traffic Engineering (SCOTE), AASHTO Standing Committee on Research (SCOR) Research Advisory Committee (RAC), and Transportation Research Board (TRB) committees.

Figure 10 shows a screen capture from the website interface and Table 7 identifies the questions asked in the survey.

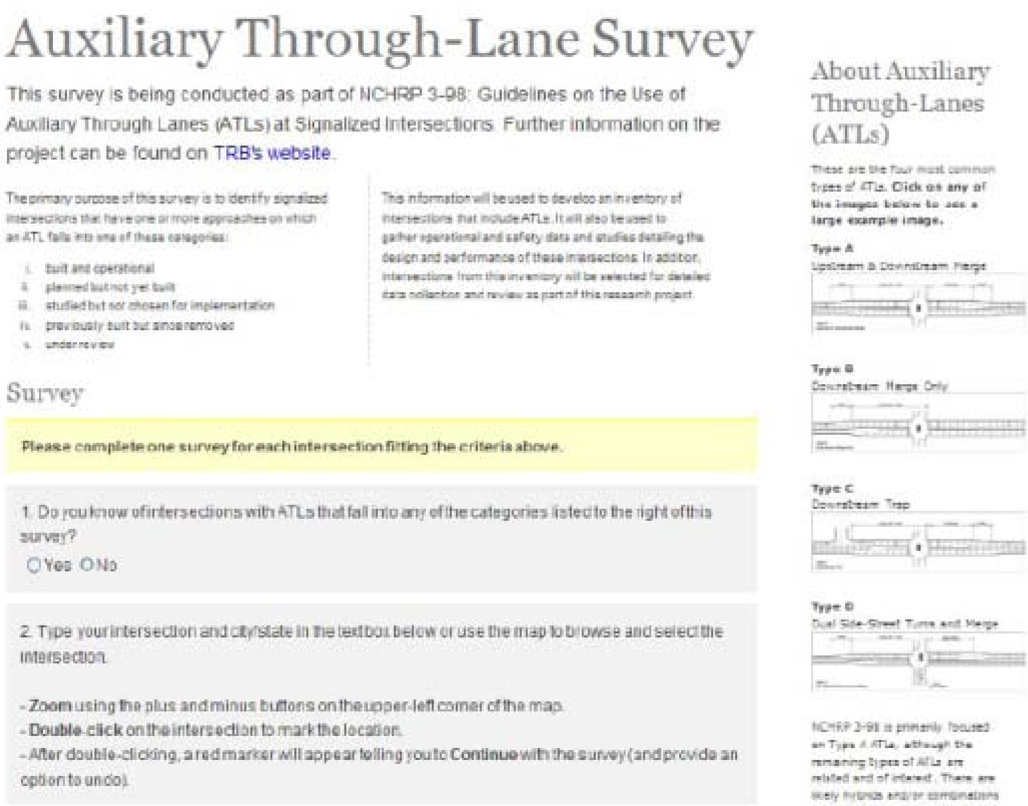


Figure 10. Survey website interface.

Table 7. Survey questionnaire.

Question		Options
1	Do you know of intersections with ATLs that fall into any of the categories listed to the right of this survey?	Yes / No
2	Type your intersection and city/state in the text box below or use the map to browse and select the intersection.	Pin on the Google Map
3	Which type of ATL does this intersection have, had, or is potentially going to have?	Primary Type A
		Related Type B
		Related Type C
		Related Type D
		Other / Hybrid
4	Which category does this intersection fall into?	Built and operational
		Planned but not yet built
		Studied but not chosen for implementation
		Previously existing but since removed
		Under Review
		Other
5	Please indicate whether any of the following data for this intersection are available and could be shared with the research team:	
	Traffic Counts	Yes / No / Maybe
	Crash Data	Yes / No / Maybe
	Signal Timing	Yes / No / Maybe
	Speed Information	Yes / No / Maybe
	Previous Study	Yes / No / Maybe
	Anecdotal Evidence	Yes / No / Maybe
	Other Information	Yes / No / Maybe
6	If you answered yes to any part of question 5, who may we contact regarding details of this evaluation?	Me
		Other (provide contact information)
7	Does your organization have written analysis or design guidelines regarding the use and evaluation of ATLs?	Yes / No
8	Is there anything else you would like to share with the research team regarding ATLs?	(open)

A total of 42 out of 249 transportation agency contacts responded to the survey resulting in a response rate of 17%.

Table 1 summarizes the number of ATL approaches identified from the survey respondents. Note that a single intersection can include one or more ATLs.

Table 8. Summary survey statistics.

Metric	Count
Total ATLs	144
Total Intersections with ATLs	117
Number of States with identified ATLs	22
Number of ATL configuration types	11

As shown in the exhibit, a total of 144 ATLs were identified in the survey across 117 intersections in 22 states of the United States. Figure 11 displays the identified ATL locations geographically. These locations provided the basis for determining the field data collection sites.

The remainder of this chapter presents findings from the survey organized by the following topics:

- Organization type of respondents
- Region
- Upstream and downstream ATL configuration
- Right-turn treatment
- Number of continuous through lanes
- Surrounding land use
- Available traffic and safety data
- Anecdotal comments

ORGANIZATION TYPE

The survey respondents represent state departments of transportation (DOT), counties, cities, universities, and consultants. Respondents from the academic and private sector generally provided a public agency contact. Figure 12 displays the number of respondents by agency type for all 42 participants.

Figure 11. Map of the ATL sites in the United States.

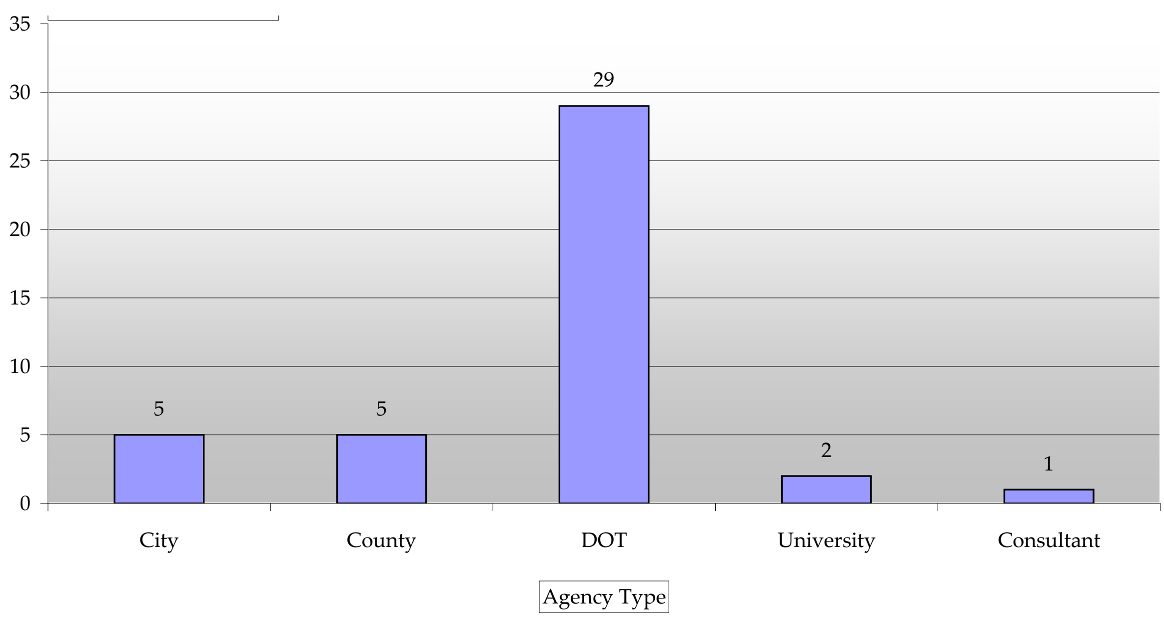


Figure 12. Respondents by agency type.

REGIONAL LOCATIONS

Sites were identified in all regions of the US. Figure 13 shows the number of ATL approaches by region as categorized by: Midwest (MW), Northeast, (NE), Northwest (NW), Southeast (SE) and Southwest (SW).

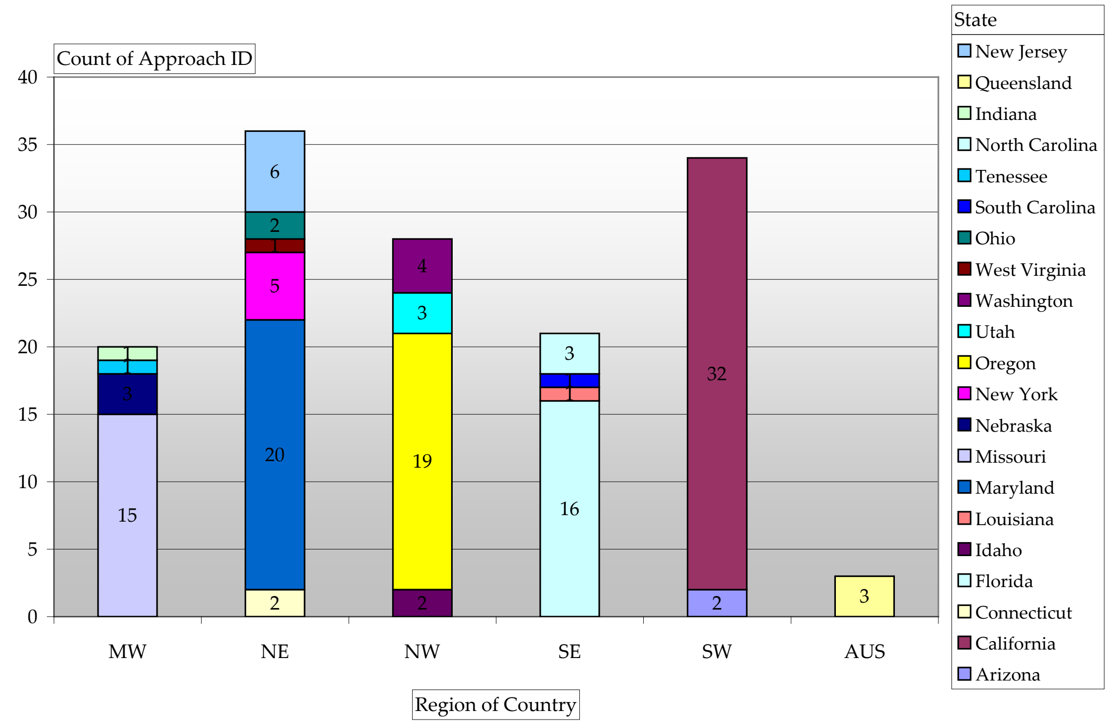


Figure 13. Count of sites by region of the United States.

The number of sites by region is relatively balanced, with the highest number identified in the southwest (led by California) and the second highest number of sites in the northeast (led by Maryland).

UPSTREAM AND DOWNSTREAM CONFIGURATION

Each ATL site has one of three possible types of upstream configurations:

- **Continuous through lane (CTL)**: the auxiliary lane directly originates from an upstream through lane at least 1,250 feet from the intersection.
- **Add lane**: one lane is added to the continuous through lane(s) by widening the roadway width.
- **Turn lane**: the auxiliary lane begins from multiple left-turn or right-turn lanes on the side street.

On the receiving end, ATL sites were classified based on one of four configuration/transition types:

- **Right-hand merge**: a reduction in the roadway width that tapers from the right-side of the roadway.
- **Left-hand merge**: a reduction in the roadway width that tapers from the left-side of the roadway.
- **Right-hand drop**: the ATL turns into a turn-only lane on the right-side of the roadway downstream of the intersection (trap lane).

- **Left-hand drop**: the ATL turns into a turn-only lane on the left-side of the roadway downstream of the intersection (trap lane).

Table 9 presents the proportion of the different downstream transitions per ATL upstream transition types.

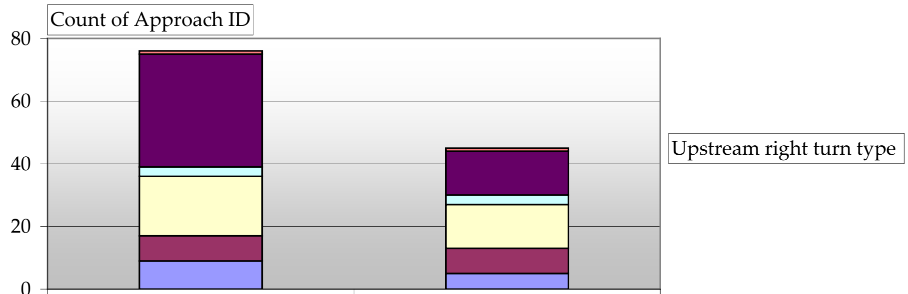
Table 9. Types of ATL by upstream and downstream transitions.

	Downstream transition			
Upstream transition	**LH drop**	**LH merge**	**RH drop**	**RH merge**
Side-street multiple turn lanes	0 (0%)	1 (1%)	7 (5%)	12 (9%)
Through add	3 (2%)	1 (1%)	9 (6%)	63 (45%)
Through basic	1 (1%)	5 (4%)	15 (11%)	24 (16%)
TOTAL	**4 (3%)**	**7 (5%)**	**31 (22%)**	**99 (70%)**

The majority of the site approaches represents a conventional lane merge from the right originating from an added through lane (45%). Two other common types of auxiliary lanes are the ATL originating from a continuous through lanes that either merge from the right (16%) or are trapped as a right-turn lane (11%). Finally multiple turn lanes from the side street that merge from the right (9%) and added through lanes that drop as a right-turn only lane (6%) are less common. The other types of auxiliary lanes are rare.

RIGHT-TURN TREATMENT

From the literature search, the configuration and presence of a right-turn lane has a significant effect on lane utilization. Various types of right-turn treatments were identified. The right-turn treatment could be a short pocket (approximately 50 feet of storage), channelized island (protects right-turn movements at the intersection), pocket and channelized island, shared with a through lane, or none (T intersection). Figure 14 shows the proportion of the different types of right-turn lanes for the ATL approaches identified in the survey.



	Th add	Th basic
Shared ThRT + RT pocket	1	1
Shared ThRT	36	14
RT pocket/channelized	3	3
RT pocket	19	14
RT channelized	8	8
No RT	9	5

Upstream transition

Figure 14. Right-turn lane treatment by upstream transition.

The majority of the ATL sites from the survey have either a shared through/right lane (50 sites) or an exclusive right-turn-pocket lane (33 sites).

NUMBER OF CONTINUOUS THROUGH LANES

The number of continuous through lanes on the approach for each of the ATL sites ranges from 1 to 4, and the number of downstream basic lanes ranges from 1 to 3. Table 10 provides a breakdown of sites based on the number of upstream and downstream basic lanes. This analysis was performed for through ATL sites (ATLs originating from side-street dual turns were not included).

Table 10. Number of basic lanes on ATL approach (through only).

Number of upstream continuous through lanes	Number of downstream continuous through lanes		
	1 basic lane	2 basic lanes	3 basic lanes
1 basic lane	51 (42%)	-	-
2 basic lanes	29 (24%)	25 (21%)	-
3 basic lanes		15 (12%)	-
4 basic lanes	-	-	1 (1%)

As shown in Table 10, the most common roadway approach type for the through ATL sites is one basic through lane upstream and one basic through lane downstream (42%). The second most common roadway approach type is two basic through lanes upstream and one basic through lane downstream, indicating that the ATL is used as a transition from a four-lane road to a two-lane road. Other roadway types include two basic lanes upstream and downstream, three basic lanes upstream and two downstream, and four basic lanes upstream and three basic lanes downstream.

LAND-USE/DEVELOPMENT TYPE

The surrounding land-use and development type for each ATL site was classified based on a visual inspection of the surrounding area using aerial photography. Sites were classified as urban, suburban, or rural. Figure 15 summarizes the proportion of sites by land-use context.

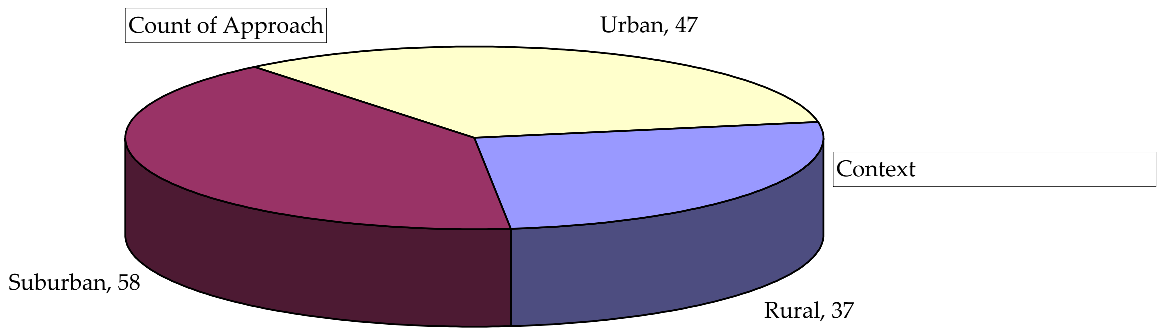


Figure 15. Proportion of ATL approaches by context.

The identified ATL approaches are located in a mix of rural, suburban, and urban contexts. The highest proportion of ATLs are located in suburban areas.

TRAFFIC/SAFETY DATA AVAILABILITY

Respondents were asked to indicate whether certain types of data are available for each ATL site. Figure 16 shows the answers, when positive, given by the respondents.

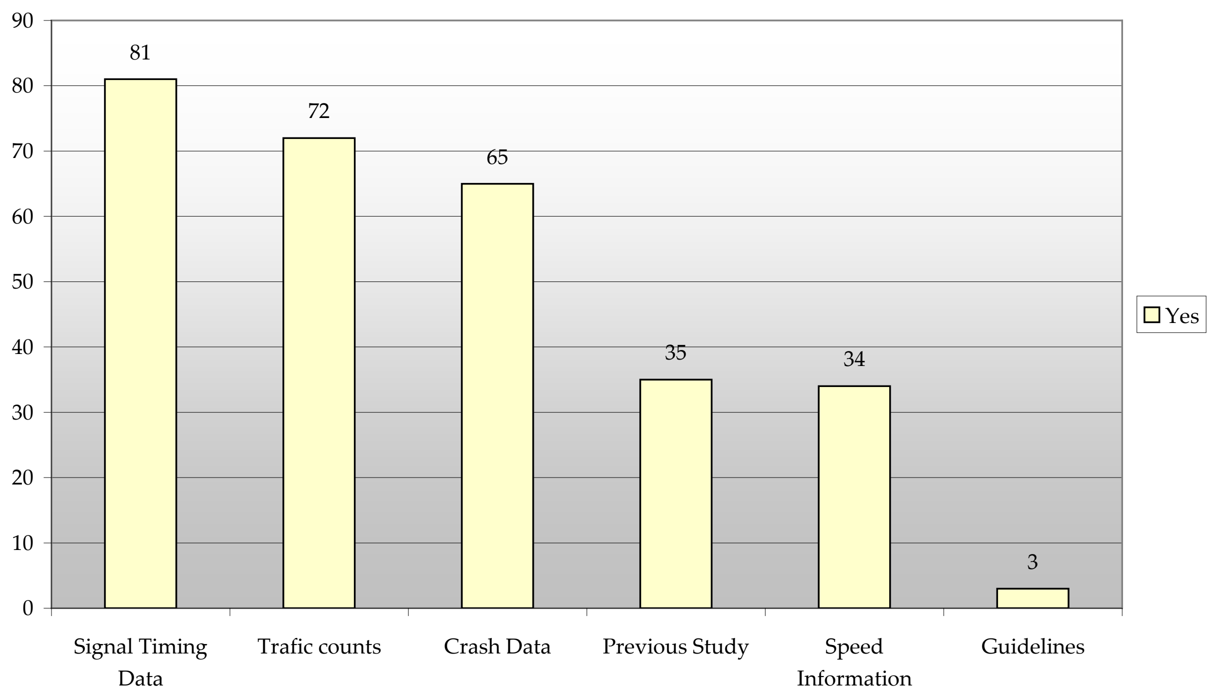


Figure 16. ATL data types available as indicated by survey respondents.

Signal timing, traffic counts, and crash data are the most readily available data with respectively 57%, 50%, and 46% of all the ATL sites having this type of data. Previous studies and speed information are available for a lesser proportion of sites.

Three agencies indicated that they have guidelines on the use of auxiliary through lanes: New York State DOT, Illinois DOT, and Caltrans.

ANECDOTAL COMMENTS

In addition, several respondents offered comments about their own experience with ATLs. Those are presented below:

- **New Hampshire DOT**. The intersection shown is one of dozens statewide at which traffic signals were added to a relatively high-volume highway and the ATLs were required to minimize delay. We do not have an inventory "of such instances but they are not uncommon. The most common complaint is that the merge following the intersection is too short or that aggressive drivers use the ATL to get ahead."
- **Pima County, AZ**. The Pima County/City of Tucson Pavement Marking Design Manual (Aug. 2008) has guidelines for signing and striping lane drops and trap lanes.
- **Connecticut DOT**. The contact person does not think any of these were specifically designed for the purpose of getting more vehicles through intersection and that they just got designed that way.
- **Ada County, ID (no ATL sites)**. The contact person expressed concerns regarding types A and B, as they may gain capacity at the intersection, but merging back into few lanes downstream can present problems at higher traffic volumes. Types C and D are less of a concern, and are more common due to potential future widening of roads in developing areas.
- **Mississippi DOT**. Mississippi has intersections that use Type C and Type D elements. However, these were designed this way due to project geometric parameters/constraints and not specifically as an ATL project.
- **Arkansas DOT**. The only type of intersection that would approximate this design is at the end of a project where the widening is carried through the intersection in anticipation of another project. It would function as an ATL for a time until the adjoining project was constructed.
- **Missouri DOT.** Missouri found these to be mostly ineffective with local drivers. If they know the lane ends, they don't use it, so there is very little capacity benefit.
- **Louisiana DOT.** They tend to be underutilized by motorists who know the lane ends beyond the intersection, creating a lane imbalance in the next lane over. There is an issue of fairness at play here. However, the imbalance may contribute to improved performance of the adjacent right-turn-on-red maneuver.
- **NYSDOT**. The need and lengths of ATLs in NYS are determined based on crash data and traffic simulation software.

CHAPTER 5: FIELD DATA COLLECTION

The research team identified a total of 46 candidate ATLs for study across 32 intersections. The sites considered included those identified in the Interim Report (as a result of the web survey) and at the Panel Meeting, field observations, as well as through additional research and discussions following the Panel Meeting. The following were the primary factors were considered in the selection of sites for data collection:

- **Congestion**. It is important to obtain observations under periods of both high and low congestion levels.
- **Number of CTLs**. A range of both ATL approaches with both one and two CTLs is desired.
- **ATL Length**. A range of lengths are desired to compare the effects of ATL length on lane utilization.
- **Right Turn Treatments**. A mix of both shared and exclusive right turn lanes are desired.
- **Geographic Location**. While previous traffic data collection research has shown that geographic location does not significantly influence results, it is desired to have observations from sites in various locations across the country. Priority was also given to sites located in close proximity to the project team's offices.

Based on a review of potential sites and the selection criteria, a total of 22 ATL approaches were chosen for study. Two types of studies were conducted: Basic and Special. Basic studies were conducted across all ATL study approaches. Special studies were performed at six of the 22 ATL approach sites. Table 1 summarizes the selected sites for field data collection.

Table 11. ATL field data collection sites.

ATL #	Intersection Name	Location	Approach	Right Turn Treatment	ATL Length[1]	# of CTLs
1	SW Murray Blvd and SW Walker Rd	Beaverton, OR	EB	Shared	< 1000	1
2			WB	Shared	< 1000	1
3	NC 54 and Fayetteville Rd	Durham, NC	EB	Exclusive	> 1500	1
4	MD 2 and Arnold Rd	Annapolis, MD	NB	Exclusive	> 1500	2
5			SB	Exclusive	> 1500	2
6	La Canada Dr and Magee Rd	Tucson, AZ	NB	Shared	1000-1500	1
7			SB	Shared	1000-1500	1
8			EB	Shared	1000-1500	1
9			WB	Shared	1000-1500	1
10	La Canada Dr and Orange Grove Rd	Tucson, AZ	SB	Shared	1000-1500	1
11			NB	Shared	1000-1500	1
12	NW 185th Ave and NW Walker Rd	Beaverton, OR	EB	Exclusive	< 1000	1
13			WB	Shared	< 1000	1
14	Sunset Lake Road and Holly Springs Rd	Holly Springs, NC	SB	Shared	1000-1500	1
15	Garrett Rd and Old Chapel Hill Rd	Durham, NC	NB	Exclusive	< 1000	1
16			SB	Exclusive	< 1000	1
17	MD 214 and Kettering Drive	Upper Marlboro, MD	EB	Exclusive	1000-1500	2
18	N 1st Ave (IL 171) and W North Ave (IL 64)	Melrose Park, IL	NB	Shared	> 1500	2
19			SB	Shared	> 1500	2
20	N 1st Ave (IL 171) and Roosevelt Rd	Maywood, IL	NB	Shared	< 1000	2
21			SB	Exclusive	< 1000	2
22	US 1 and New Falls of Neuse Rd	Wake Forest, NC	SB	Exclusive	> 1500	2

BASIC STUDIES

Table 12 lists the data elements that were collected as part of the basic field studies. "ATL use" is quantified as either ATL utilization (the percentage of all approach through traffic that uses the ATL) or in terms of ATL hourly flow rate (which includes right turn movements if the lane is shared). The later variable is included because it does not consider the total number of through vehicles and is therefore easier to distinguish from through congestion and flow rate. The independent variables are organized into the following categories:

- **Traffic Volume** – based on turning movement counts and traffic composition recorded from the intersection
- **Geometric Design** – describes the geometric elements of the intersection and ATL
- **Signal Timing** – based on signal timing characteristics per lane on an average basis

Table 12. Dependent and independent variables considered.

Dependent Variables	Independent Variables: Traffic Volume	Independent Variables: Geometric Design	Independent Variables: Traffic Signal
ATL Flow Rate (per hour) $ATL\ Utilization = \frac{ATL\ Flow}{ATL\ Flow + CTL\ Flow} \times 100\%$	Total Through Flow Rate (per hour) $X_T = \frac{ATL\ Flow + CTL\ Flow}{Capacity\ of\ CTL('s)}$ $X_R = \frac{Right\ Turn\ Flow}{.85 \times Capacity\ of\ ATL}$ Delay Savings = CTL Delay (assuming no ATL) – ATL Delay[1] Delay for Through Traffic Only[1]	ATL Length[2] Number of CTL's (1 or 2) Type of ATL (Shared / Exclusive)	Cycle Length Effective Green[3] Through Saturation Flow Rate[4] Right Turn Saturation Flow Rate[4]

[1] Delay computed using 2000 HCM Chapter 16, $d_1 + d_2$. The CTL Delay (assuming no ATL) is the delay in the CTL('s) assuming all through traffic uses the CTL('s)

[2] Measured from beginning of storage to end of taper

[3] Effective green taken as the maximum effective green for actuated control signals

[4] Saturation flow rate calculated using the 2000 HCM field procedure

Note: All "Flow" refers to through flow rate only unless otherwise noted.

Camera Placement

All data were collected using video recorded for each approach. Camera number and placement varied from approach to approach, but at each approach, at least one camera was placed upstream of the intersection and turned back toward the intersection to enable a clear view of the signal head and all approach lanes. An additional upstream camera was used where necessary to capture queues in order to account for the true demand (in the case of cycle failures), as opposed to volume counts alone. A downstream camera was normally used to count "captive" ATL users (i.e., ATL users who turned into driveways downstream of the intersection). These vehicles were not included in the ATL user counts.

Data Extraction

For each single continuous through lane (1-CTL) approach, the effective green time *g*, cycle length *C*, and continuous through lane (CTL), auxiliary through lane (ATL), and right turn demand counts were extracted on a cycle-by-cycle basis. These variables were used to calculate operational parameters such as the volume-to-capacity ratio for the CTL assuming all through demand is in the CTL (X_T), delay, and demand flow rates by lane. Due to the high amounts of usable amber time observed from the field, the total lost time per each green phase was taken to be two seconds. Additionally, the through saturation flow rate was calculated for each 1-CTL approach using the field procedure outlined in the Highway Capacity Manual (HCM), averaged over 30 cycles.

Due to highly congested conditions and the width of the roadway, data for two continuous through lane (2-CTL) approaches were extracted differently from the other data in the following ways:

- True demand could not always be accounted for from the video, in part due to long moving queues and high volumes at certain sites. The through-movement counts per cycle were computed instead.
- The HCM analytical method was used to consider the effects of heavy vehicles on saturation flow rate. For each cycle, the saturation flow rate was computed as

$$SFR\ (vph) = \frac{1900\ vph}{1 + \%HV}$$

  where %HV is the percentage of heavy vehicles during that cycle, expressed as a decimal. Other effects such as lane width, grade, and area type were not included in this computation because they did not vary from cycle to cycle. When this method was applied, the resulting saturation headways were reasonable (e.g. 1.8 to 2.2 sec/veh).
- The total lost time per phase was increased to three seconds for the Illinois sites due to blockage from previous phases.

For a shared ATL, the ratio of the right-turn saturation flow rate to the through saturation flow rate was assumed to be 0.85, also consistent with the HCM. In this working paper, data from individual cycles were aggregated into 15-minute averages in order to retain both the trend and variability in the data.

Results from Basic Studies

Table 13 summarizes the basic data collected at each study approach. Although data were collected on a cycle-by-cycle basis, the results presented here reflect the data aggregated into 15-minute intervals. The following are some observations about the 1-CTL approaches:

- ATL utilization ranged from 9% to 40%, with an average utilization of 21% and a median of 19%
- The right-turn percentages ranged from 9% to 32% for the shared ATL approaches
- Although X_T ranged from 0.26 to 1.26, a v/c of over 1.0 was only observed at 4 out of the 12 recorded approaches. This was likely because of the 15-minute aggregation, which tended to hide individual cycle failures
- The total length of the ATL ranged from 910 to 2,700 feet

There were fewer 2-CTL approaches than 1-CTL approaches, but there was also a wider representation of data for the 2-CTL approaches. The following are some observations about the 2-CTL approaches:

- ATL utilization ranged from 5% to 26%, with an average utilization of 16% and a median of 18%. The low utilization at the MD 214 site (#17 in Table 11) was likely due to a large portion of arrivals on green enabled by a high progression bandwidth
- The right-turn percentages ranged from 16% to 22% for the shared ATL approaches
- X_T ranged from 0.42 to 1.2.
- The total length of the ATL ranged from 810 to 2,980 feet

Appendix B includes data plots that show relationships between different combinations of the following variables based on 15-minute observations for each of the study ATL approaches:

- ATL Volume
- Total Through Volume
- X_T (volume-to-capacity ratio for the CTL assuming all through demand is in the CTL)
- Delay Savings (calculated using HCM as the difference between the delay in the CTL assuming no ATL use and ATL delay)
- ATL % (percent of total through traffic in the ATL)
- X All (volume-to-capacity ratio inclusive of through and right-turning traffic where right-turns are shared)

These graphs provide useful insights for understanding the relationships between the dependent and independent variables and serve as the foundation for the development of the statistical model that will predict the use of the ATL for prevailing traffic and geometric conditions. Appendix B also includes the raw data summaries from the 15-minute observations for each ATL approach.

Table 13. Summary operational statistics for ATL Sites.

				ATL Characteristics				Summary Results from Data Reduction						
#	Intersection	Location	Approach	ATL Type	Total ATL Length [1]	# CTL's	# 15-min Data Points	Special Study	Average Through Volume (vph)	ATL Utilization % Through	Average RT Volume (vph)	Approach RT %	X_T Range	Average Cycle Length (s)	Average Effective Green (s)
1	SW Murray Blvd and SW Walker Rd	Beaverton, OR	EB	Shared	< 1000	1	15		589	28	152	20	.44 - 1.26	95	27
2			WB	Shared	< 1000	1	6		551	29	102	16	.93 - 1.30	125	28
3	NC 54 and Fayetteville Rd	Durham, NC	EB	Exclusive	> 1500	1	12	X	323	23	350	52	.42 - 1.06	98	19
4	MD 2 and Arnold Rd	Annapolis, MD	NB	Exclusive	> 1500	2	18	X	2011	19	87	4	.85 - 1.22	178	97
5			SB	Exclusive	> 1500	2	13	X	1420	20	25	2	.57 - .90	158	86
6	La Canada Dr and Magee Rd	Tucson, AZ	NB	Shared	1000-1500	1	9		683	19	95	12	.63 - 1.03	89	39
7			SB	Shared	1000-1500	1	8		576	18	144	20	.52 - .74	90	46
8			EB	Shared	1000-1500	1	13		407	19	37	9	.47 - .85	90	21
9			WB	Shared	1000-1500	1	7		367	14	168	31	.49 - .77	90	23
10	La Canada Dr and Orange Grove Rd	Tucson, AZ	NB	Shared	1000-1500	1	13		436	19	95	18	.46 - .82	120	51
11			SB	Shared	1000-1500	1	6		330	19	154	32	.31 - .41	120	59
12	NW 185th Ave and NW Walker Rd	Beaverton, OR	EB	Exclusive	< 1000	1	8		424	40	245	36	.73 - 1.26	125	29
13			WB	Shared	< 1000	1	6		334	15	204	38	.46 - .81	109	29
14	Sunset Lake Road and Holly Springs Rd	Holly Springs, NC	SB	Shared	1000-1500	1	5	X	350	9	94	21	.37 - .68	113	28
15	Garrett Rd and Old Chapel Hill Rd	Durham, NC	NB	Exclusive	< 1000	1	9	X	270	19	466	62	.26 - .58	126	28
16			SB	Exclusive	< 1000	1	5	X	257	23	89	25	.32 - .44	125	29
17	MD 214 and Kettering Drive	Bowie, MD	EB	Exclusive	1000-1500	2	12		2114	5	116	5	.75 - 1.09	150	94
18	N 1st Ave (IL 171) and W North Ave (IL 64)	Melrose Park, IL	NB	Shared	> 1500	2	9		708	18	135	16	.91 - 1.23	159	28
19			SB	Shared	> 1500	2	9		850	18	301	26	.80 - 1.15	159	40
20	N 1st Ave (IL 171) and Roosevelt Rd	Melrose Park, IL	NB	Shared	< 1000	2	10		860	6	242	22	.78 - 1.02	120	31
21			SB	Exclusive	< 1000	2	10		793	26	110	12	.76 - 1.15	121	28
22	US 1 and New Falls of Neuse Rd	Wake Forest, NC	SB	Exclusive	> 1500	2	5		1559	13	218	14	.53 - .77	187	83

1. Defined as the upstream ATL storage length plus the downstream ATL storage length not including tapers or intersection width.

SPECIAL STUDIES

Two special field studies, **downstream merge behavior** and **acceleration profile**, were performed to calibrate the driver behavior model in VISSIM. Two additional special studies were performed in order to test the accuracy of the calibrated VISSIM model: **maximum queue length** and **travel time by lane**. In addition to their significance to ATL use as previously hypothesized, these variables were used because they were relatively easy to manually record in the field and/or extract from DVDs.

The special studies were performed for the ATL approaches listed in Table 14.

Table 14. Special study approach sites.

ATL #	Approach	Location	ATL Length (ft)[1]	ATL Type	# of CTL's
3	EB NC 54 at Fayetteville Rd	Durham, NC	> 1500	Exclusive	1
4	NB MD 2 at Arnold Rd	Annapolis, MD	> 1500	Exclusive	2
5	SB MD 2 at Arnold Rd	Annapolis, MD	> 1500	Exclusive	2
14	SB Sunset Lake Blvd at Holly Springs Rd	Holly Springs, NC	1000-1500	Shared	1
15	NB Garrett Rd at Old Chapel Hill Rd	Durham, NC	< 1000	Exclusive	1
16	SB Garrett Rd at Old Chapel Hill Rd	Durham, NC	< 1000	Exclusive	1

1. Defined as the upstream ATL storage length plus the downstream ATL storage length not including tapers or intersection width.

Downstream Merge Behavior

The location and behavior of each downstream merge from the ATL into the outer CTL (if more than one CTL) was measured during the peak hour at each approach. Each merge was classified as one of the following three types:

- Gap – The merge was essentially free-flow; the driver in the ATL either met no resistance in the CTL (i.e., there were no drivers in the CTL near the merge point) or made use of natural gaps between drivers in the CTL
- Yield – The driver in the CTL slowed to form a gap so that the driver in the ATL could merge safely
- Forced – The driver in the ATL merged into the CTL without considering nearby drivers in the CTL; this was usually identified by a sudden jerk in the steering of the driver in the ATL and/or braking by the driver in the CTL

In addition to the type of merge, the location of each downstream merge was recorded by roughly partitioning the downstream length of the ATL into thirds. The area closest to the intersection was labeled "early," with "middle" and "late" representing the next two areas of the downstream taper, as shown in Figure 17.

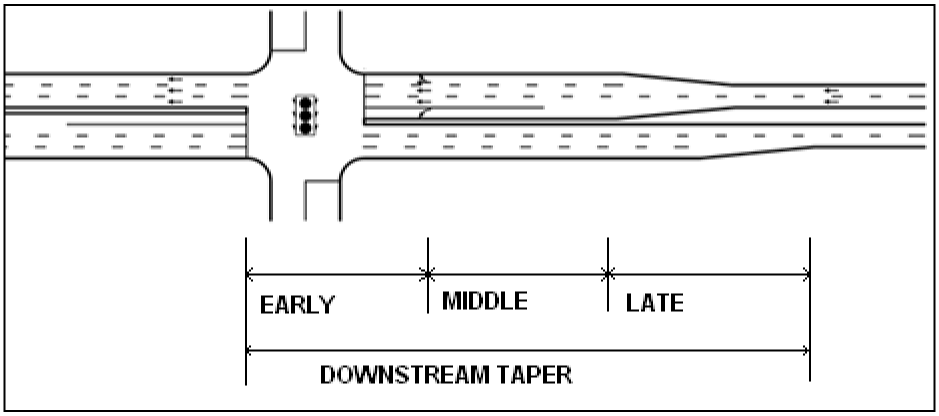


Figure 17. Downstream merge location.

Since the length of the downstream taper varies by approach, the distance from the stop bar covered by each of these areas also varies by approach. Table 15 lists the early, middle, and late merge distances by approach.

Table 15. Downstream merge area distance from stop bar (feet).

Approach	Early	Middle	Late
NC 54	0-300	300-600	600-900
NB MD 2	0-300	300-600	600-900
SB MD 2	0-400	400-800	800-1200
NB Garrett	0-150	150-300	300-450
SB Garrett	0-150	150-300	300-450
Sunset Lake	0-400	400-800	800-1200

Figure 18 summarizes the downstream merge behavior recorded in the field. As shown in the figure, the vast majority of merges occur through a gap in traffic at all locations of the ATL. However, merges are more likely to be yield or forced merges if they occur late in the ATL--this mainly reflects that ATL users will take longer to merge in a congested approach and will eventually force their way into the CTL. This is similar to VISSIM logic in that there is a "point of no return" beyond which a driver will stop until a driver in the CTL yields or they force their way into the CTL. In general, the high occurrence of gap merges across a wide range of ATL lengths indicates that the downstream length of the ATL did not affect driver’s merge behavior at the observed sites.

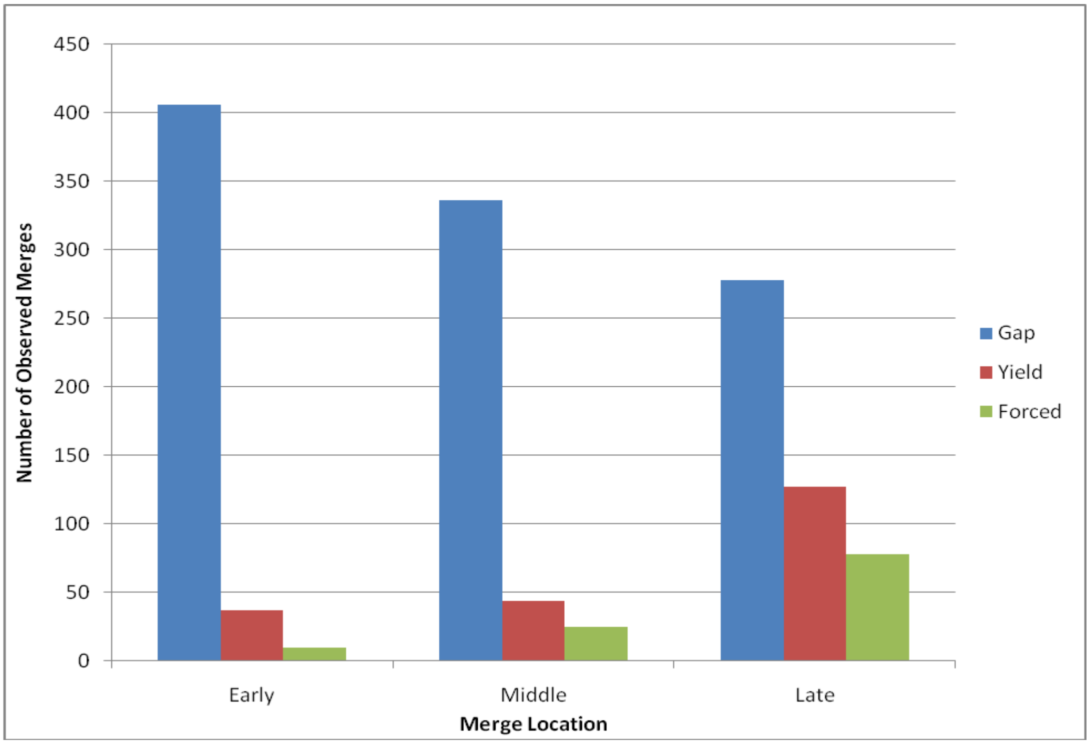


Figure 18. Downstream merge behavior results.

Table 16 contains a breakdown of this merge behavior by approach. This table contains a breakdown of the merge behavior data by approach and supports the claim that the majority of merges occur at normal gaps in traffic, regardless of downstream ATL length or the location of the merge. The merge location seems to be nearly evenly-split between the early, middle, and late segments of the downstream ATL.

Table 16. Downstream merge behavior results by approach.

Approach	Gap	%	Yield	%	Forced	%	TOTAL
Early Merging							
NC 54	121	49%	15	6%	6	2%	142
NB MD 2	140	34%	4	1%	0	0%	144
SB MD 2	138	21%	17	3%	4	1%	159
Garrett	7	28%	1	4%	0	0%	8
Subtotal	***406***	***30%***	***37***	***3%***	***10***	***1%***	***453***
Middle Merging							
NC 54	47	19%	13	5%	15	6%	75
NB MD 2	116	29%	9	2%	1	0%	126
SB MD 2	170	26%	20	3%	9	1%	199
Garrett	3	12%	2	8%	0	0%	5
Subtotal	***336***	***25%***	***44***	***3%***	***25***	***2%***	***405***
Late Merging							
NC 54	23	9%	5	2%	1	0%	29
NB MD 2	85	21%	28	7%	24	6%	137
SB MD 2	164	25%	92	14%	49	7%	305
Garrett	6	24%	2	8%	4	16%	12
Subtotal	***278***	***21%***	***127***	***9%***	***78***	***6%***	***483***
TOTAL	**1020**	**76%**	**208**	**16%**	**113**	**8%**	**1341**

Acceleration Profile

Average acceleration rates are an indicator of driver aggressiveness and were shown to affect ATL utilization in VISSIM. The acceleration rate of the leading vehicle in each lane was recorded by assuming a constant acceleration rate and then measuring the time for each vehicle to travel a fixed distance. The kinematic equation

$$a = \frac{2\Delta x}{(\Delta t)^2} \quad (1)$$

allows for this calculation. For each approach, the distance used for the calculation was 100 feet. Acceleration data were only recorded if there were vehicles in both the ATL and the CTL('s), as it would be unnecessary for a driver in the ATL to accelerate quickly if there were no downstream merging conflict with a CTL user, and vice versa.

Figure 19 shows a plot of the cumulative distribution of vehicle acceleration by lane. A K-S test concluded that at a 5% level of significance (α = 0.05), there is no significant difference between the acceleration distributions of the ATL and CTL vehicles. It should be noted that the sample size at this stage of the study is only 51 ATL vehicles and 97 CTL vehicles.

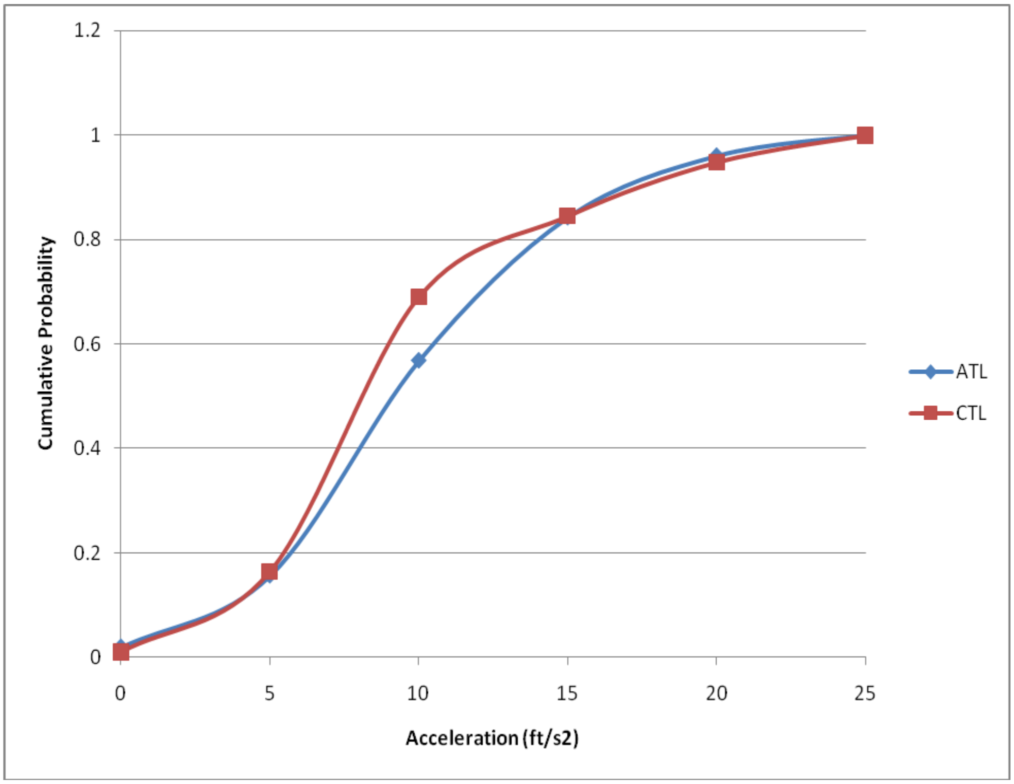


Figure 19. Cumulative distribution of acceleration from the stop bar in the ATL and CTL.

Maximum Queue Length

The maximum queue length (in vehicles) during each cycle was recorded for each lane. Since queue length is a rough indicator of congestion, it was hypothesized that longer queue lengths in the CTL would lead to higher ATL use. Additionally, this variable was useful for model validation due to the ease with which VISSIM can compute separate queue lengths in each lane. This observation was relatively accurate for the Durham sites, but the oversaturated approaches at MD 2 made accurate observation difficult, as there were several cycle failures. Figure 20 shows the cumulative distribution of maximum queue lengths in each lane.

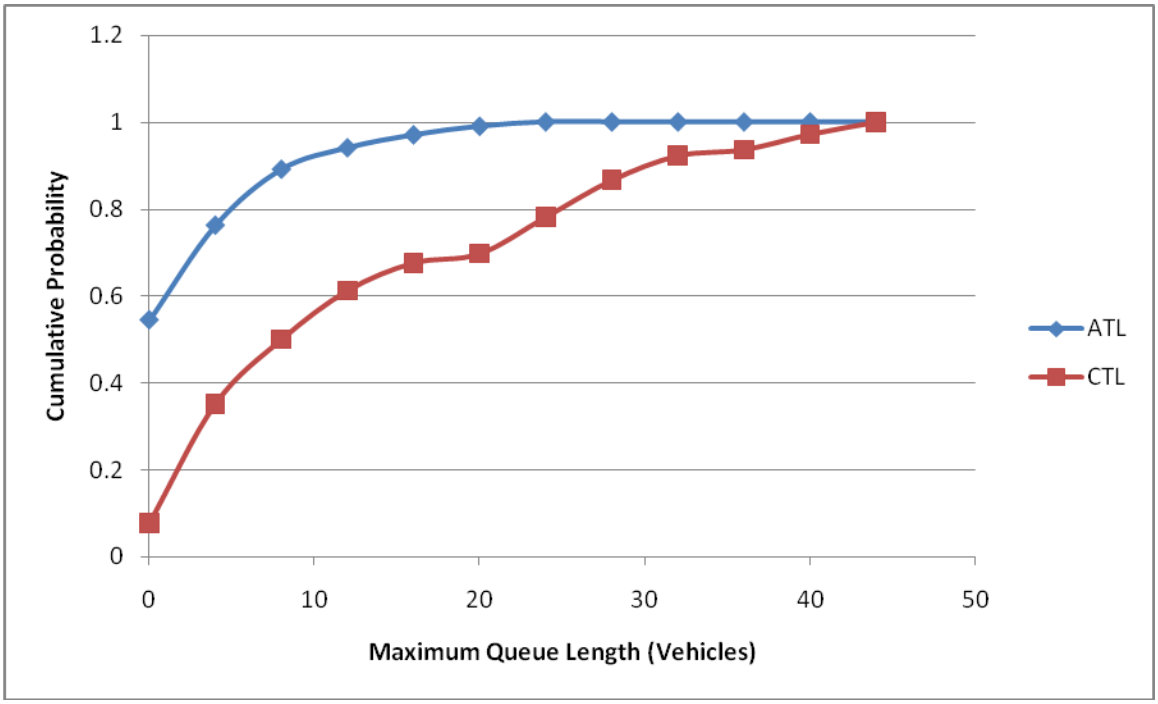


Figure 20. Maximum queue length cumulative distribution.

As shown in Figure 20, a total of 101 ATL queues were observed, and 142 CTL queues were observed. This figure shows that the average maximum queue length tends to be longer in a CTL than in the ATL. Both the Chi-Square goodness of fit test and the K-S test concluded that at a 5% level of significance, the distribution of maximum queue lengths in the CTL differed from the distribution in the ATL. This supports the hypothesis that ATL use is stimulated by a difference in queue lengths between lanes.

Travel Time

Hypothesized to be the chief stimulus for ATL use, the travel time over each lane was calculated using synchronized video recordings of the upstream and downstream segments of the approach. Full sampling of each lane was used at the Durham sites, but full sampling was deemed impractical at the Maryland site due to the high volume of traffic. At this site, six data points were recorded during each cycle: one travel time in each lane arriving on green (free flow) or red, joining the queue.

Each travel time observation was used to compute the travel time delay in each lane, defined as the difference between the observed travel time and the “free flow” travel time at the posted speed limit. Two techniques were used to compare the travel time delay in the CTL(‘s) with the travel time delay in the ATL. The first was to normalize the delay to a length of 1000 ft, as shown in the equation

$$\text{Delay / 1000 ft} = \frac{\textit{Observed Travel Time} - \textit{Free Flow Travel Time}}{\textit{Travel Time Segment Length (ft)}} \times 1000 \quad (2)$$

The second method of comparing the delay in each lane was to compute the Delay %:

$$\textit{Delay \%} = \frac{\textit{Observed Travel Time} - \textit{Free Flow Travel Time}}{\textit{Free Flow Travel Time}} \times 100\% \quad (3)$$

Figure 21 shows the cumulative distribution of travel time by lane for vehicles that arrive during the red phase. For this case, there were 66 ATL travel times and 174 CTL travel times recorded.

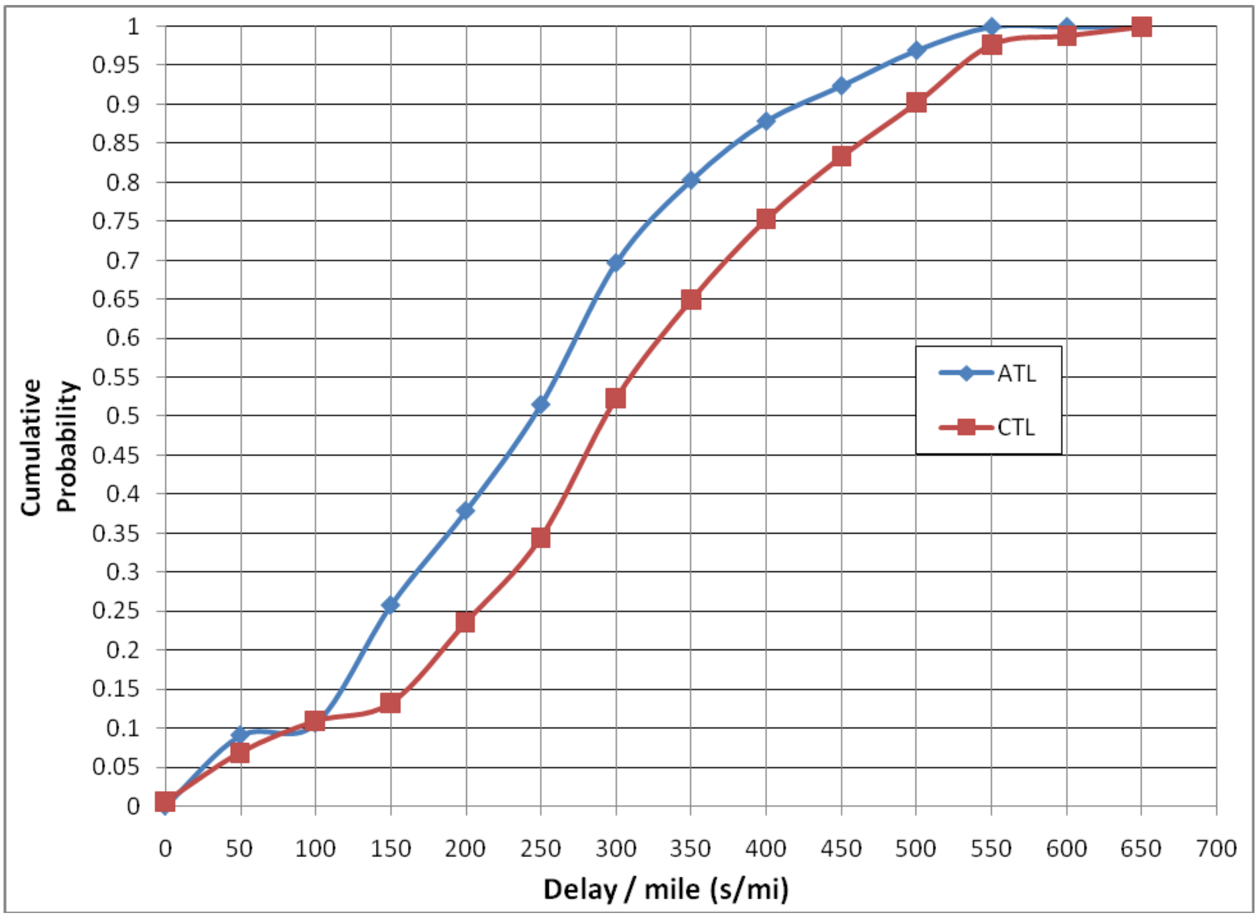


Figure 21. Cumulative distribution of travel time for vehicles arriving during red in the ATL and CTL.

Figure 22 shows the same plot for vehicles arriving during the green phase. For this case there were 43 ATL travel times and 109 CTL travel times recorded. For both sets of travel time distributions, a Chi Square goodness of fit test revealed that at a 5% level of significance (α = 0.05), the distribution of CTL travel times differs from the distribution of ATL travel times. A

K-S test for both sets of distributions yielded similar results at the 5% level of significance—the distribution of CTL travel dimes differed from the distribution of ATL travel times.

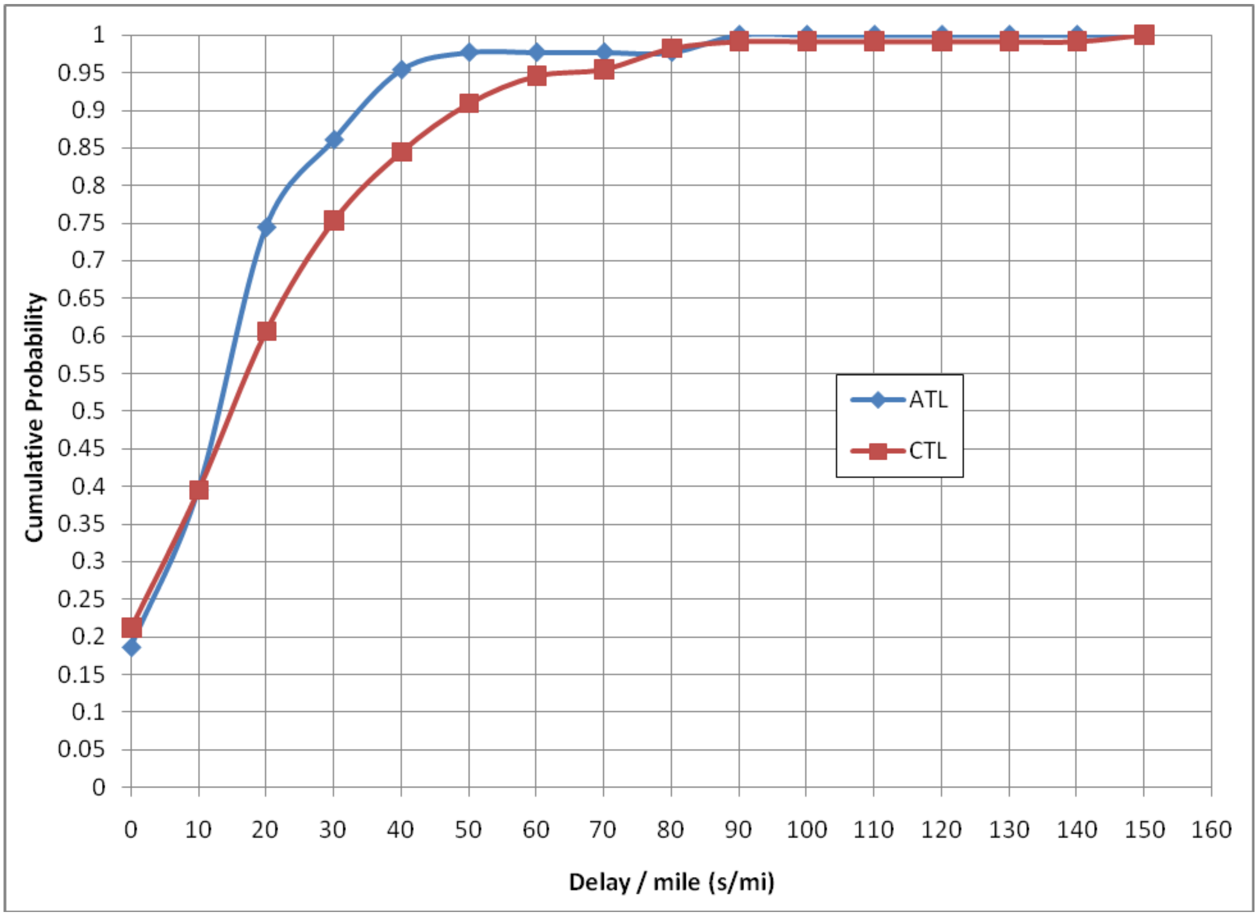


Figure 22. Cumulative distribution of travel time for vehicles arriving during green.

CHAPTER 6: OPERATIONAL ANALYSIS AND MODELING

The research team conducted a detailed analysis of the field data to determine the factors that influence ATL use and to identify appropriate method for predicting lane use in the ATL. As described in the Operations Concept Method section, a simulation-based approach and analytical-based approach were developed on the premise that drivers will choose to use the ATL based on a perceived travel time savings gained by using the ATL. However, a review of travel time data collected in the field showed no correlation between potential travel time savings by using the ATL and the volume of traffic in the ATL. Further, contrary to previous research, the length of the ATL was found to not have a significant influence on drivers' decision to use an ATL.

DESCRIPTIVE STATISTICS

Table 17 lists the range, mean, and standard deviation of several parameters extracted from field observations.

Table 17. Field descriptive statistics.

Variable	1-CTL	2-CTL	Overall
ATL Flow Rate (vph)			
Range	12 – 327	26 - 553	12 – 553
Mean	96	210	143
Standard Deviation	56	127	108
X_T			
Range	0.23 - 1.30	0.53 - 1.23	0.23 - 1.30
Mean	0.67	0.91	0.77
Standard Deviation	0.23	0.16	0.24
X_R			
Range	0 - 0.53	0 - 1.01	0 - 1.01
Mean	0.14	0.20	0.17
Standard Deviation	0.23	0.31	0.22
Delay Savings (s)*			
Range	3 – 139	7 - 150	3 – 150
Mean	19	44	30
Standard Deviation	25	27	29
Through Flow Rate (vph)			
Range	165 - 946	596 - 2492	165 – 2492
Mean	435	1377	825
Standard Deviation	152	583	608
Shared Right-turn Flow Rate (vph)			
Range	16 – 241	94 -384	16 – 384
Mean	118	227	144
Standard Deviation	56	78	77
*Using HCM Delay Models *(1)*			

It is evident from Table 17 that the 2-CTL sites exhibit the upper extreme of most of the variables, which justifies the development of separate models for 1-CTL and 2-CTL sites. Of these variables, through-movement flow rate and X_T appear to be the principle factors that influence ATL flow rate. Plots showing these relationships (broken down by approach) for 1- and 2-CTL approaches are shown in figures 23-26.

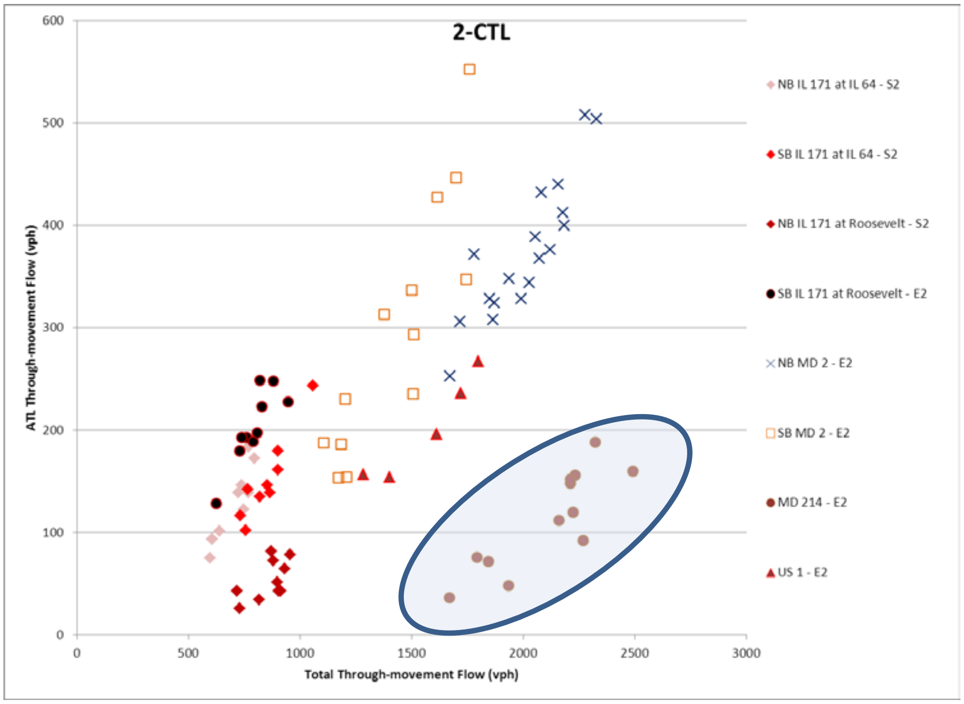


Figure 23. ATL through-movement flow vs. total through movement flow (1 CTL).

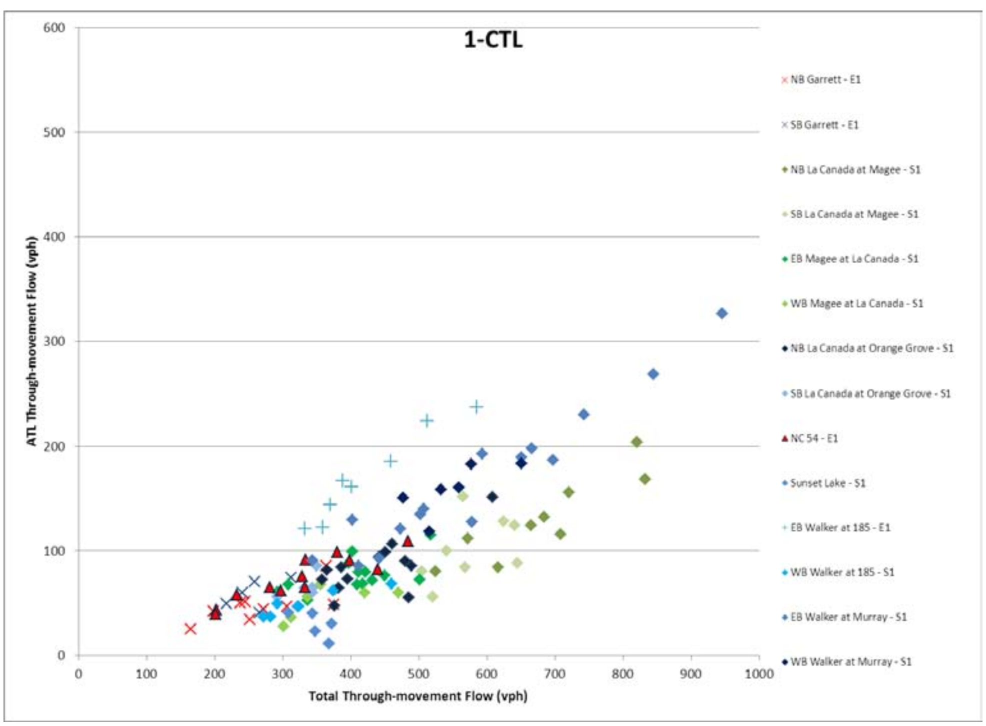


Figure 24. ATL through-movement flow vs. total through movement flow (2 CTL).

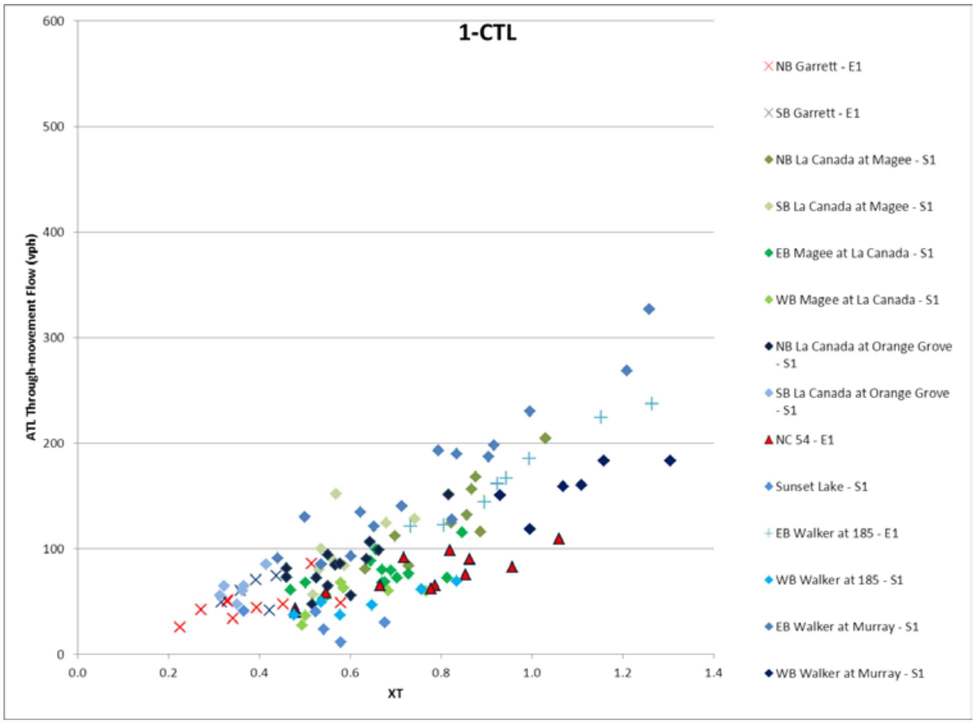


Figure 25. ATL flow vs. through-movement congestion (X_T) (1 CTL).

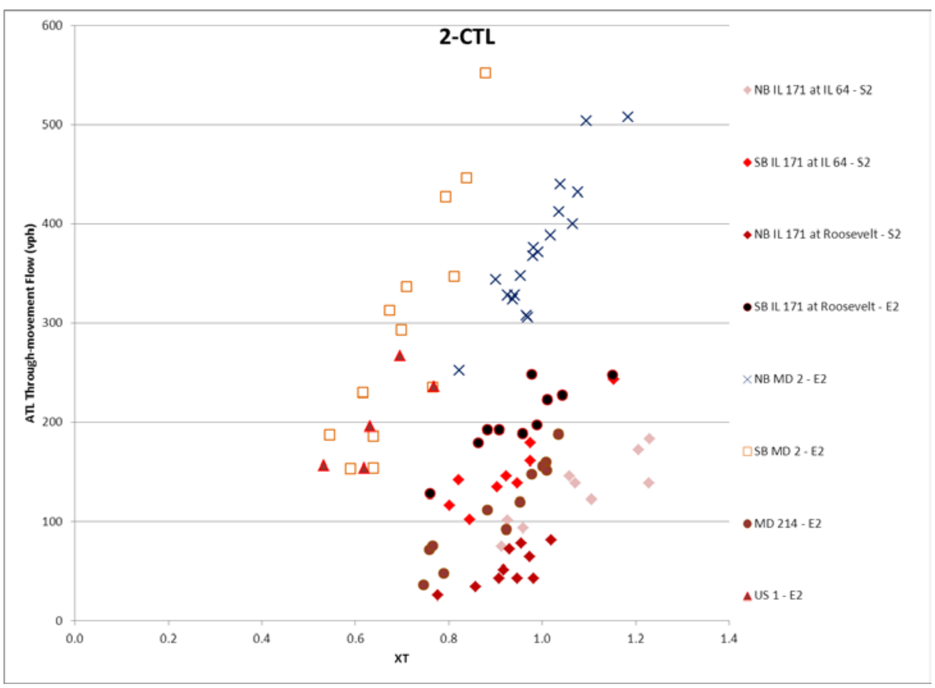


Figure 26. ATL flow vs. through-movement congestion (X_T) (2 CTLs).

The plots in figures 23-26 indicate that the relationship between ATL flow rate and either through-movement flow rate or X_T may be non-linear. While the 1-CTL data appears to be fairly consistent when compared across several sites, there is considerably more variation in the 2-CTL data by approach. In particular, the MD-214 approach experienced less ATL use than the other sites, even though the total through-movement flow on that approach was significantly higher. Field observation led to the conclusion that this site experienced very low ATL use because of extremely good signal progression for the through-movement that resulted in a low number of arrivals on red (and, consequently, less queuing, which likely detracted from ATL use).

STATISTICAL MODEL DEVELOPMENT

Table 18 lists the primary models developed from the calibration dataset. The first, Model 1, relates the ATL flow rate for 1-CTL data to X_T^2 and Through Flow Rate^2 (the later term was divided by 10,000 to more appropriately scale the coefficient). This model is logical in that it contains highly significant variables ($p < 0.0001$) that are consistent with the hypotheses that ATL flow rate increases with increasing through-movement flow rate and congestion level as indicated by X_T. Furthermore, it explains nearly 80% of the variation in the data.

Table 18. Summary of results from statistical model development.

	1-CTL Model	2-CTL Models	
Variable	**Model 1**	**Model 2A**	**Model 2B**
Intercept	20.226	132.55	29.240
X_T^2	81.791***		
Thru Flow / 100		7.49*	17.3***
Thru Flow^2/ 10000	1.65***		
X_R		-125.45*	-90.291*
R^2	0.780	0.349	0.768
Calibration Sample Size	122	86	74
p-value: *<0.01 **<0.001 ***<0.0001			

Variation in the 2-CTL data was more difficult to explain. The initial model, Model 2A, relates the ATL flow rate for the 2-CTL data to through-movement flow rate and X_R (the right-turn v/c for shared lanes—see Table 2 for formula). The model intuitively indicates that ATL flow rate increases with through-movement flow, and it also decreases the predicted flow if the ATL is shared with right-turns, as indicated by the negative coefficient of X_R. However, none of the model parameters are statistically significant, and it only explains 35% of the variability in the data. Model 2B explains significantly more of the data by removing the outlying MD-214 site. Although this may reduce the applicability of the model, Model 2B can still be used with the provision that it may not accurately predict the ATL use at a site with a high amount of arrivals on green (i.e. good signal progression). Since the MD-214 site had several characteristics that should otherwise encourage high ATL use (e.g. high through-movement volume, exclusive right turn lane, no driveways), it was considered a special case in this study.

ATL Design Elements

In addition to these three models, several other models were developed to explore the relationship between ATL use and the design elements of the ATL. Ultimately, the additional variability explained by these more complex models was small compared to the number of new variables introduced, so these design variables were removed from the final models. However, this exercise resulted in several conclusions about the design elements of the ATL:

- ATLs with higher posted speeds tended to experience slightly more use—although this may not be intuitive, it was hypothesized that a higher posted speed tended to create a speed differential between queued vehicles and those arriving at the beginning of green (who were more encouraged to use the ATL to pass queued vehicles in the CTLs);
- Driveways along the ATLs created a safety hazard and detracted from their use;
- A hill or other obstruction to the sight distance along the downstream ATL detracted from ATL use;
- Shared ATLs experienced less use than exclusive ATLs;
- Good signal progression led to shorter queues in the CTLs and consequently lower ATL use;
- Overhead signage of the ATL as a through-movement lane (as evidenced at each of the Oregon sites) encouraged ATL use.

As previously mentioned, early attempts to include each of these elements as variables in the model resulted in complex models with little gains in explaining more of the variability in the data. These models were deemed impractical for incorporation into the HCM and other manuals but could be alternatively summarized as guidelines for practitioners who wish to design or improve existing ATLs.

Model Validation

Of the original dataset containing 208 15-minute observations, only 75% was used for model development. Figure 27 shows a plot of the observed ATL flow rate for the remaining 56 data points versus the predicted ATL flow rate for each of the three models. Model 1 accurately predicted most of the validation data, while Model 2B was more accurate than Model 2A. Furthermore, the 2-CTL models appear to over-predict the underutilized ATLs and under-predict the ATLs with high flow. These errors may be attributed to the site-to-site variability.

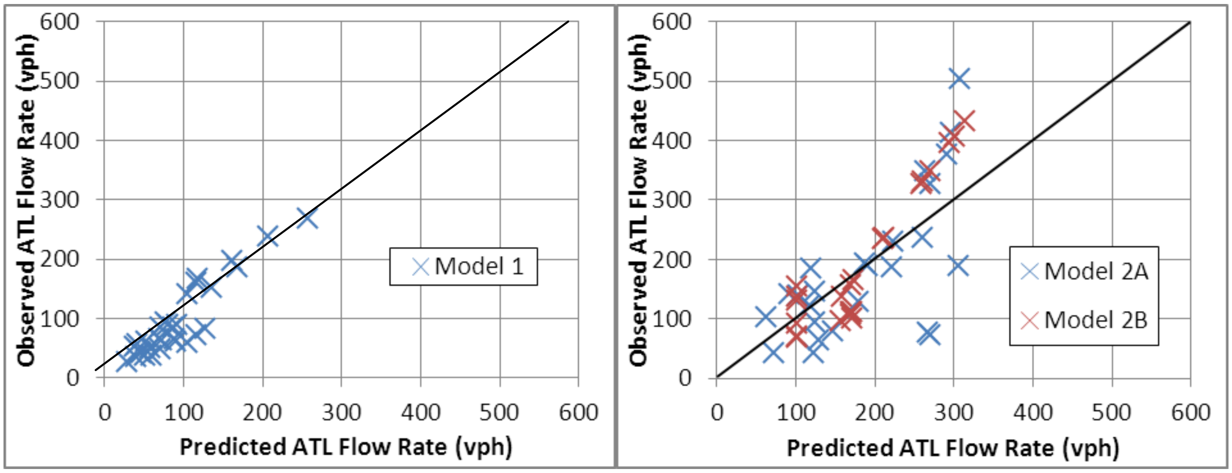


Figure 27. Model validation plots.

Note that this is not "validation" in its purest form, since the new data was extracted from the same approaches as the data used to calibrate the model. However, the trends shown in Figure 27 imply that the models work well at explaining the high degree of variability in ATL use from period to period. Table 19 contains a summary of the models' ability to predict the validation data when a linear trend line is set to each plot in Figure 27. The values of Validation Plot R^2, Intercept, and Slope describe these trend lines. Finally, the accuracy of each model was determined by calculating the number of validation points predicted within 50 vph (for each 1-CTL data point).

Table 19. Model validation results.

	1-CTL Model	2-CTL Models	
Parameter	**Model 1**	**Model 2A**	**Model 2B**
Validation Plot R^2	0.859	0.471	0.857
Validation Plot Intercept	-11.4	-13.2	-90.6
Validation Plot Slope	1.13	1.04	1.56
% Validation Points predicted ±50 ATL vph	100%	48%	68%
Validation Sample Size	31	25	22

Returning to the models themselves (Table 18), the terms in Models 1 and 2B are the most intuitive. The value of the intercept term in each of these two models (20.2 and 29.2 vph, respectively) indicates that at least one vehicle will use the ATL each cycle, regardless of the level of congestion—this is consistent with field observations. Furthermore, the inclusion of X_R in the 2-CTL model indicates that more right-turning vehicles in a shared ATL will detract from ATL use.

ESTIMATING ATL LENGTHS

A key design element of ATLs is the appropriate length of the ATL's upstream and downstream components. Although it may be hypothesized that a longer ATL promotes higher ATL use, extensive field observations of ATLs tend to contradict this theory.

Figure 28 displays a plot of the observed ATL flow rate for each study approach against the corresponding downstream length.

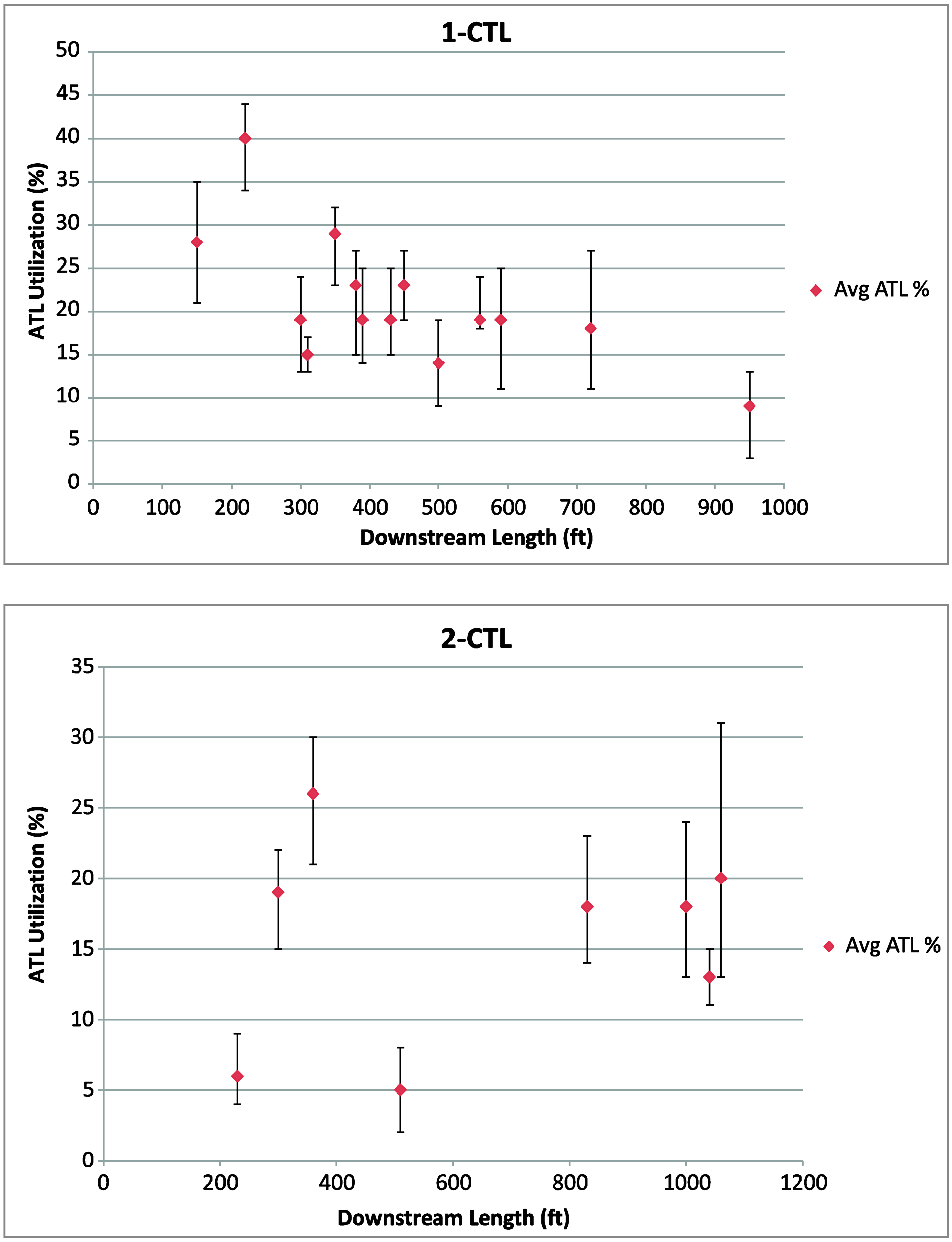


Figure 28. Minimum, average and maximum ATL utilization versus downstream length.

The figure shows the minimum, average, and maximum ATL utilization per site for 1- and 2-CTL approaches. It is clear from the data that downstream length plays little, if any, role in enticing drivers to use the ATL at the observed sites.

Instead of the length of the ATL, the primary motivator for using the ATL appears to be a defensive one: avoiding a cycle failure when traffic in the adjoining CTL is moderately to highly congested. Based on this premise, the required ATL upstream length is predicated on the provision of adequate storage for and access to the ATL from the neighboring CTL. The downstream length, on the other hand, is predicated on servicing the queued vehicles in the ATL so that they can accelerate to the approach free flow speed and smoothly merge before reaching the end of the downstream taper. Gap availability and acceptance in the CTL for ATL vehicles operating under relatively high-speed, uninterrupted conditions must also be considered. Therefore, the recommended minimum downstream length is the greater of the lengths determined from these two operating conditions.

Note that the lengths determined from this method represent a *guide* for practitioners for ideal conditions. Poor downstream sight distance, lack of proper signage (or existence of overhead lane signs), presence of downstream driveways, and significant right-turn-on-red (RTOR) flow from cross street traffic may all necessitate adjustments to the ATL length to accommodate those effects.

Appendix C provides the detailed method for estimating ATL lengths.

COMPUTATIONAL ENGINE

The research team developed computational engines in an Excel spreadsheet environment to implement the ATL operational procedures for pre-timed traffic signal control. The base geometric configuration for 1-CTL approaches is a shared CTL and an exclusive left-turn lane. Three possible design scenarios can be evaluated in the computational engine:

- Add an exclusive right-turn pocket (no ATL),
- Add an ATL with shared right turns, or
- Add an exclusive ATL and an exclusive right-turn pocket.

Additionally, the computational engines provide planning-level estimates of annual delay and dollar savings by switching from the base case to any of the three types of design scenarios. They also enable testing how much green time can be re-allocated from the subject approach to other critical movements at the intersection.

A similar set of design options is provided in a separate spreadsheet for the 2-CTL scenarios, including all three design scenarios described above.

The Input Dialog Box

The input dialog box for the 2-CTL case is depicted in Figure 29 below. Users enter all the required input data in the yellow-colored boxes. Particular attention should be paid to entries 2 and 3 following the case study ID. Their combination determines which of the 3 enhancements will be implemented. Thus an "N", "N" sequence represents a 2 exclusive CTLs plus a shared ATL configuration. A "Y", "N" sequence represents the NO-ATL, exclusive-right-turn pocket case and the "Y","Y" sequence represents the case of an exclusive ATL and an exclusive-right-turn pocket. Note that the sequence "N","Y" is not feasible. Entry 11 enables the testing of green time re-allocation and, thus, is always less than or equal to entry 9. All other input data items are self-explanatory.

2-CTL ONLY

COMPUTATIONS OF ATL LENGTHS (UPSTREAM AND DOWNSTREAM) -- FOR VARIOUS LANE CHOICES

	INPUT DATA HERE- CASE I IS THE BASELINE (THRU CTL + CTL SHARED LANE)			
	ALL CELLS EXCEPT INPUT CELLS ARE PROTECTED			
1	ENTER CASE STUDY ID OR TITLE IN YELLOW BOX		ANALYSIS OF SHARED- ATL SCENARIO	
2	ENHANCEMENT: EXCLUSIVE RIGHT TURN LANE (Y/N)?	N	Please enter data in CAPS for the first 2 entries	
3	ENHANCEMENT: ADDITIONAL EXCLUSIVE THRU ATL (Y/N) ?	N	This entry cannot be "Y" if previous enry is "N"	
4	TOTAL APPROACH THROUGH VOLUME=	1500	VPH	
5	RIGHT TURN VOLUME=	200	VPH	
6	APPROACH SPEED (MPH)=	35	MPH	
7	THRU SATFLOW (TOTAL)=	3600	VPH	
8	RIGHT SATFLOW PER LANE=	1530	VPH	
9	APPROACH EFFECTIVE GREEN=	60	SEC	
10	INTERSECTION CYCLE LENGTH=	120	SEC	
11	APPROACH EFFECTIVE GREEN WITH ATL / OTHER ENHANCEMENTS=	45	SEC	DEFAULT
12	AVERAGE VEHICLE SPACING AT STOP=	20	FT	20
13	AVERAGE ACCELERATION RATE FROM STOP=	10	FT/SEC/SEC	10
14	INTERSECTION WIDTH (STOPLINE TO FAR CURB)=	40	FT	40
15	CRITICAL GAP I NEIGHBORING CTL LANE=	6	SEC	6
16	DRIVER REACTION TIME	1	SEC	1
	BASED ON THE INPUTS- THIS IS ANALYSIS CASE	III	2-CTL'S+SHARED ATL	

Figure 29. Screen capture of requirements for ATL computational engine.

Note that items 12-16 are primarily related to determining the downstream ATL length, and can be all defaulted to the values shown on the right-hand side.

A final note regarding the required inputs; in the "CTL" worksheet, the user is prompted to select either the mean or a percentile of the number of rejected gaps. If the percentile option is selected, then the actual confidence level must be input as well. This value effects the computation of the DSL_2 term as explained in Appendix A. In the following example showing the summary results, the mean value of the number of rejected gaps option was used.

Summary Results Tabulation

Detailed calculations of lane volumes based on the NCHRP 03-98 and HCM 2010 lane-volume allocation models and the resulting delays and queue lengths are shown in the "ATL" and "1-

or 2-CTL" tabs. Otherwise, all model findings and design lengths are given in the SUMMARY spreadsheet, as shown in Figure 30.

ANALYSIS OF SHARED ATL SCENARIO

SUMMARY OF RESULTS FOR CASE III 2-CTL'S+SHARED ATL

CONDITION	EFFECTIVE GREEN TIME (SEC)	THRU VOL	RIGHT VOL	TOT VOL	XALL	d	LOS	95TH % queue (FT)	AVERAGE DELAY	LOS	UPSTREAM L. BASED ON STORAGE / ACCESS Q-LENGTH (FT)	DOWNSTREAM L. BASED ON STORAGE LENGTH (FT)	DOWNSTREAM L. BASED PN CTL-GAP ACCEPT LENGTH (FT)
BASELINE- THRU CTL CASE I	60	868	0	868	0.964	52.09	D	830					
BASELINE- SHARED CTL CASE I	60	632	200	832	0.964	51.90	D	830	51.999	D	830		
NO EXCLUSIVE RT- POCKET			0	0	0.000	0.00							
ATL SHARED**	45	362	200	562	0.887	51.86	D	600					
2-CTL'S	45	1138	0	1138	0.886	44.28	D	500					
COMBINED PERFORMANCE FOR INDICATED CASE									46.787	D	600	740	260
ATL -- DELAY SAVINGS PER HOUR (VEH-HRS OF DELAY)								veh-hrs	2.461				

APPROXIMATE SAVINGS PER YEAR (ASSUMING 2 PEAKS PER DAY , 5 DAYS, 50 WEEKS)	veh-hrs	1231
APPROIMATE DOLLAR SAVED PER YEAR ASSUMING THIS VALUE OF TIME PER HOUR	10	$ 12,305

*if ATL thru vol is color highlighted like this then the 3-98 model volume estimate governs otherwise it is based on equal v/s

**If ATL Shared carries no through trafic, then it operates as a defacto RT pocket

Figure 30. Screen capture of output summary from ATL computational engine.

The engine provides lane-by-lane analysis for the base case and for the enhanced scenario. Measures include the lane volume allocation, the degree of saturation, delay, LOS, and 95th percentile queue by lane (except for the 2 exclusive CTLs, which are treated as single lane group as per the HCM 2010). In the case shown in Figure 30, the through traffic in the ATL was based on equalizing the v/s ratio among lanes eligible to serve through traffic because the models will tend to allocate more through traffic to the ATL. For this example, the difference was minimal: the model predicted 371 through vehicles per hour using the ATL, while the equal v/s criterion assigned 362 vehicles per hour to the ATL. The summary report also shows that the addition of the ATL enables the reallocation of up to 15 seconds of original green time (60 minus 45 seconds) to other critical movements, without much of an effect on delay (52 seconds in the baseline vs. 48 seconds with the shared ATL).

The right-hand side of the results report shows the various design length recommendations. The upstream length needed to provide queue storage for and unimpeded access to the ATL is estimated at 600 feet, based on the ATL 95th percentile queue. In other words, storage is the governing criterion in this case. The required downstream length based on providing storage for vehicles to reach desirable spacing at the speed limit or free flow speed is estimated at 740 feet, measured downstream of the approach far curb. The distance needed for vehicles entering the intersection at the free flow speed to find an acceptable gap in the neighboring CTL is much shorter at 260 feet. This is not surprising given a low CTL lane volume of 562 vehicles per hour (or an average headway of 6.31 seconds between CTL vehicles). It is also interesting to note the tradeoffs between signal capacity reallocation and required ATL design elements. The following table shows the required ATL upstream and downstream lengths under different green re-allocation scenarios (for the same base case shown in Figure 29 and Figure 30).

Table 20. Required ATL storage length.

Revised Approach Green Time, s	Reallocated Approach Green Time, s	Minimum Upstream ATL Length, ft	Minimum Downstream ATL Length (ft) Based on		Annual Approach Delay Savings (veh-hrs)
			Storage	Gap Acceptance	
60	0	400	590	260	6329
55	5	500	640	260	5,130
50	10	500	690	260	3,590
45	15	600	740	260	1,231
40	20	700	800	260	-4,134
35	25	800	880	260	-16,204

It is also worth noting that the distance based on the gap acceptance criterion is insensitive to the green time changes, since it is based on the assumption of uninterrupted flow for isolated vehicles that have reached their desired speed upon crossing the stop line. That particular case rarely determines the downstream length unless the CTL is highly congested or a large percentile value of the number rejected gaps (90-95%) is selected.

HCM Implementation

One of the significant enchantments of the HCM 2010 (2) methodology for evaluating signalized intersections is that it replicates dual-ring actuated traffic signal control. Implementing this actuated method requires carry out an iterative procedure using software to determine the duration of each signal phase given the interdependence of demand and capacity across all (critical) lane groups at the intersection.

Incorporating the ATL lane use prediction equations described in this chapter within the actuated control methodology of HCM 2010 presents an additional complexity: the ATL lane use equations require capacity (X_T) and therefore effective green time as an *input*, whereas phase duration and effective green time for actuated intersection is an *output* of the HCM 2010 methodology. This results in a second iterative loop within the HCM methodology.

In the long term it is the research team's goal that the ATL lane use prediction equations be incorporated within the HCM 2010 computation engine for signalized intersections (and other HCM-faithful software tools). However, for the short term the research team developed a manual procedure that allows practitioners to apply the ATL lane use prediction model within HCM 2010 software tools such that the predicted through volume in the ATL converges to a value within 10 vehicles per hour.

The procedure is as follows:

1. Run the HCM 2010 computational engine *without* the ATL(s) and record g/C & X_T for the ATL approach(es).
2. Apply the ATL computational engine and record the predicted ATL through volume for all ATL approaches, considering the minimum value determined by the equal-degree-of-saturation upper bound.
3. Subtract the ATL through volume from the total through volume and re-run the HCM 2010 computational engine. Record the new values of g/C and X_T for the ATL approach(es).
4. Reapply Step 2 using the new signal timing parameters determined in Step 3.
5. Check to see if the new ATL through volume prediction converges to within 10 vph of the previous ATL through volume prediction. If so, proceed to calculating and reporting performance measures commensurate with HCM 2010 procedures. If not, repeat process starting with Step 2.

The research team applied the above procedure to "Example Problem 1: Automobile LOS" for from the HCM 2010 for two ATL cases and found that the results for both cases converged within three iterations. Appendix D documents the results from this example.

Simulation

The ATL volume prediction models described in this chapter are based on a deterministic analysis framework and are directly compatible with the Highway Capacity Manual (HCM) procedures.

The HCM recognizes that the use of alternative analysis tools, and specifically microsimulation approaches, have merit in a number of applications. In an effort to study the utility of microsimulation tools for ATL applications, guidelines were prepared for applying simulation to ATL evaluation.

The foremost goal in calibrating a simulated CTL-ATL system is to match the field-observed ATL utilization, or in the absence of field data, the ATL volume predicted from the models presented in this chapter. By varying the Lane Change Distance (LCD) parameter in VISSIM, the research team was able to successfully calibrate 19 of the 22 ATL studied ATL approaches. The remaining three approaches exhibited very low utilization (less than 10%). These low-utilization percentages could not be replicated without also making significant adjustments to the car-following logic, which in turn resulted in more simulated "crashes."

Appendix E describes the method for calibrating simulation models for evaluating ATLs.

CHAPTER 7. SAFETY ANALYSIS AND MODELING

The objective of the safety analysis and modeling effort was to determine the safety effects of ATLs, either by examining collision data, or, if possible, by exercising a validated SSAM model. In addition to trying to determine whether an ATL had a positive or negative effect on safety relative to the same intersection approach without an ATL, the study also examined the safety effects of key ATL design parameters.

DATA COLLECTION

This study analyzed the sixteen ATL approaches listed in Table 21. These sites were selected using the following criteria:

- A wide range of through-movement volume;
- A wide range of ATL length;
- Little or no effects from neighboring intersections; and
- A wide range of locations from across the United States.

Collision data for a nine-year period (2000-2008) were compiled for each study approach from the beginning of the upstream taper to the end of the downstream taper. Although collision reporting policies vary widely by jurisdiction, the analysts worked meticulously to make fair comparisons by reviewing crash reports and estimating the location of each crash within the ATL. For this analysis, only rear end and sideswipe collisions were considered to be related to the ATL, and collisions within the intersection (not along the upstream or downstream ATL) were assumed to be unrelated to the ATL. This assumption was necessary because SSAM does not model many collision types like fixed object, head-on, or animal collisions, and because including the numerous angle and left turn collisions in the analysis would tend to obscure any effects on the number of sideswipe collisions, which were far less frequent. Field data collection during 2009 and 2010 provided the estimated peak-hour ATL utilization percentage in Table 21, as well as turning movement counts and other data needed to calibrate each VISSIM model.

Table 21. Summary of SSAM-analyzed approaches (*Denotes 2-CTL Site).

#	Approach	Location	Upstream/ Downstream Length (ft)[1]	Average ATL Utilization %
1	EB Walker Rd at Murray Blvd	Beaverton, OR	500/225	28
2	WB Walker Rd at Murray Blvd	Beaverton, OR	250/310	29
3	EB NC-54 at Fayetteville Rd	Durham, NC	1650/450	23
4	NB La Canada Dr at Magee Rd	Tucson, AZ	690/410	19
5	SB La Canada Dr at Magee Rd	Tucson, AZ	425/695	18
6	EB Magee Rd at La Canada Dr	Tucson, AZ	475/470	19
7	WB Magee Rd at La Canada Dr	Tucson, AZ	650/490	14
8	NB La Canada Dr at Orange Grove Rd	Tucson, AZ	490/550	19
9	SB La Canada Dr at Orange Grove Rd	Tucson, AZ	550/495	19
10	EB Walker Rd at 185th St	Beaverton, OR	325/225	40
11	WB Walker Rd at 185th St	Beaverton, OR	575/400	15
12	SB Sunset Lake Dr at Holly Springs Rd	Holly Springs, NC	400/1075	9
13	NB Garrett Rd at Old Chapel Hill Rd	Durham, NC	330/285	19
14	SB Garrett Rd at Old Chapel Hill Rd	Durham, NC	335/425	23
15	NB MD-2 at Arnold Rd *	Annapolis, MD	1545/690	19
16	SB MD-2 at Arnold Rd *	Annapolis, MD	1630/1050	20

1. Does not include taper.

VISSIM CALIBRATION

After initial testing it was found that the VISSIM logic that drives ATL use is based upon the *lane-change distance* of the VISSIM connector downstream of the ATL. Essentially, this lane-change distance measures the point that simulated drivers react to the lane-drop. By modifying this parameter, the researchers were able to accurately replicate the field-observed ATL utilization for most of the sites, achieving an R^2 greater than 0.9 between the model predictions and field observations. To simulate a day's worth of traffic activity, a VISSIM model was constructed for each of four time-of-day plans (AM peak, midday, PM peak, and night). Additionally, each model was simulated for the time periods 2001, 2004, and 2007 to represent the three-year periods 2000-2002, 2003-2005, and 2006-2008, respectively. However, several of the study approaches were not built as ATLs during the earliest time period, so this time period was not simulated for all sites. The following assumptions were assumed when preparing each VISSIM model:

- When not provided by state DOT's, the researchers estimated turning movement counts using the available AADT and K-factors for each time of day, assuming that turning percentages would remain constant from 2000 to 2008;
- AADT data were used to determine traffic growth or decline, assuming a linear rate from 2000 to 2008 for purposes of interpolation;
- Signal timing plans were kept in "free" mode (fully actuated with no coordination) for off-peak periods and were kept constant at each site for each three-year time period;
- One simulation run from each model was manually observed to verify that signal timing plans effectively moved traffic (the team used Synchro to optimize those that did not perform adequately);
- Traffic composition (% heavy vehicles, etc.) and ATL utilization were assumed to be consistent with that observed in the field during the 2009-2010 data collection; and
- Driveways and un-signalized side streets were not simulated.

This daunting list of assumptions stems from the fact that the study tried to reproduce traffic conditions from every time of day for each day over a nine-year time span.

Since the field ATL utilization was only obtained for one peak hour period per site, the researchers let the VISSIM logic determine the ATL utilization during the other time periods and then checked to make sure each file produced a realistic level of ATL use. Each time-of-day VISSIM model was simulated for ten one-hour replications and then analyzed with SSAM. The number of conflicts per day was estimated by weighting the results from the four time-of-day models as follows: 1 day = 2 * AM peak results + 7 * Midday results + 2 * PM peak results + 13 * Night results. The maximum default time-to-collision (TTC) threshold (1.5 seconds) was used to filter the SSAM output because it resulted in the greatest sample size for comparison to collision data and because the study found that lower TTC thresholds produced similar results (there was a high correlation between the results for TTC thresholds of 1.5 seconds, 1.0 second, and 0.5 second). The weighted SSAM results were then compared to the collision data by examining plots of conflicts versus collisions and using Pearson's chi-square test.

SSAM SENSITIVITY ANALYSIS

After the validation exercise, a sensitivity analysis was used to investigate design and operational elements of ATLs that might produce trends in SSAM. Ten replications were performed for each combination of the following levels:

- Downstream storage length: 200, 400, 600, 800, and 1,000 feet;
- Through-movement demand to capacity ratio (X_T): 0.75, 1.00, and 1.25; and

- Right Turns: 0 and 200 vph.

The levels of each variable reflected those observed in the field. Lower levels of X_T were not included because ATL use was very low during time periods when X_T was below 0.75. The upstream ATL length was fixed at 600 feet. Both shared and exclusive ATL scenarios were simulated with these variables.

ANALYSIS

Collision Data Trends

Figure 31 displays a breakdown of the collision data obtained for all 16 sites. The total number of crashes reported at all 16 sites was 1,014—this amounts to approximately eight crashes per site per year. Although the majority of collisions (52%) were rear-end crashes, only 10% were sideswipe crashes, which might be expected to be higher in ATLs.

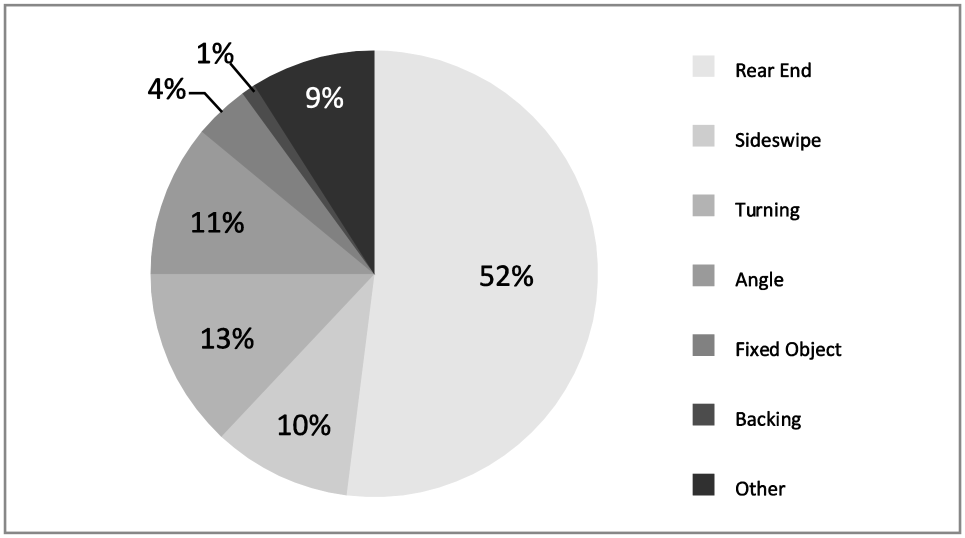


Figure 31. Field collision data trends.

The rear end and sideswipe collision data were aggregated by relative location within the ATL, as Figure 32 indicates. The line for total collisions is simply the sum of rear end and sideswipe collisions. Note that the distribution of sideswipe collisions is spread more evenly over the length of a typical ATL than the distribution of rear end collisions. This suggests that while rear end collisions usually occur in the queuing areas near the intersection, sideswipe collisions are more likely to occur in other areas of the ATL. Also note that almost exactly half of these collisions were upstream of the intersection, while about half were downstream.

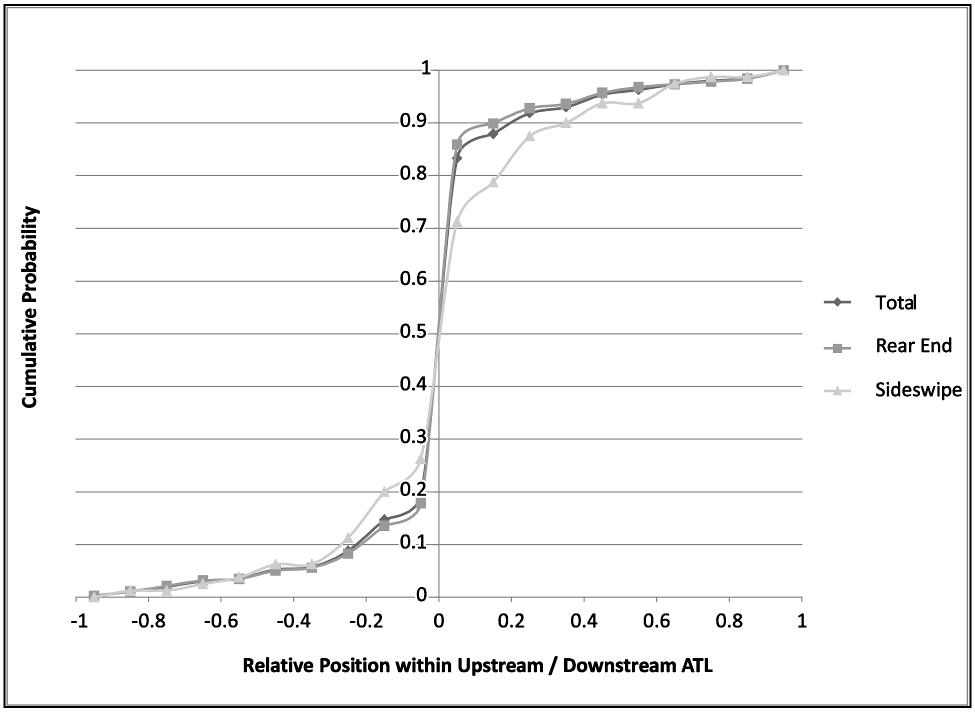


Figure 32. Collision data distribution within the ATL.

Three more relationships were explored using a comparison of the collision data with operational data. Figure 33 plots the number of rear end collisions from 2006 to 2008 vs. the maximum X_T obtained from field data collected in 2009 and 2010—this value indicates the level of congestion in the through-movement lanes. X_T was estimated using field data collected in the peak hour for each approach (the observed peak hour varied from site to site due to varying traffic patterns). The line in Figure 33 is the best-fit linear relationship between the maximum X_T observed and rear end crash frequency for each of the 16 sites. Note that only the last three years of collision data were used in order to shorten the time period between safety and operational data collection (all of the operational data were obtained in 2009 and 2010).

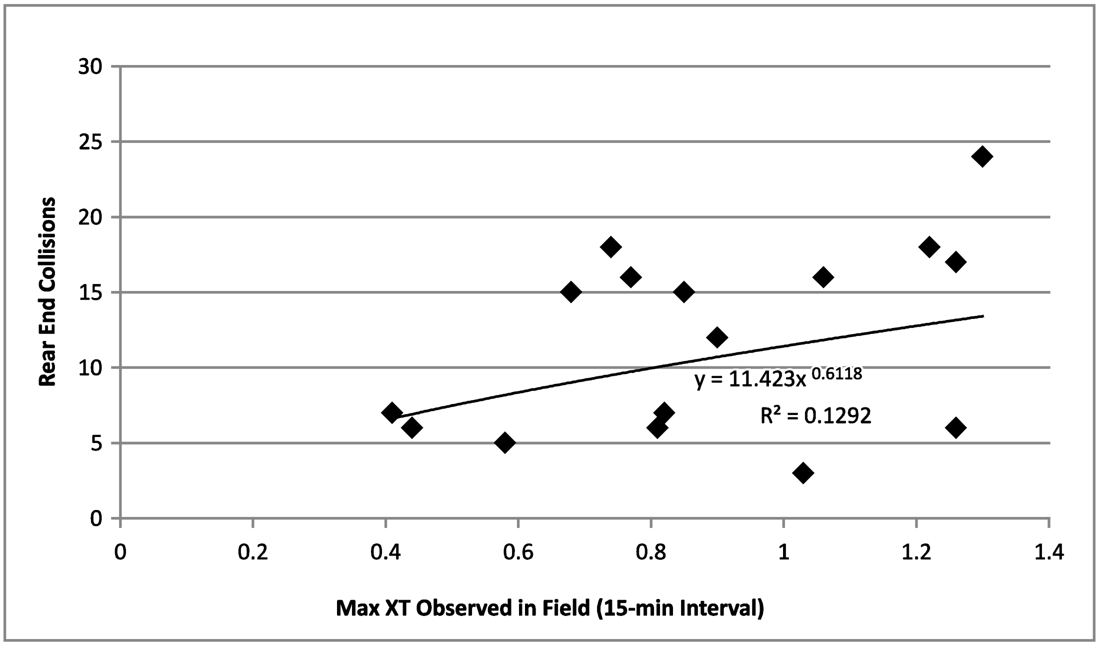


Figure 33. 2006-2008 collision data trends versus maximum X_T observed.

This relationship loses most of its strength when ATL utilization is used in place of the maximum X_T, as displayed in Figure 34. This figure suggests that ATL utilization is not strongly correlated with an increase in rear end collision frequency. Note that modeling efforts in the related operational study of ATLs revealed that the relationship between ATL utilization and operational measures is weak, so it may be difficult to relate this variable with any collision data as well.

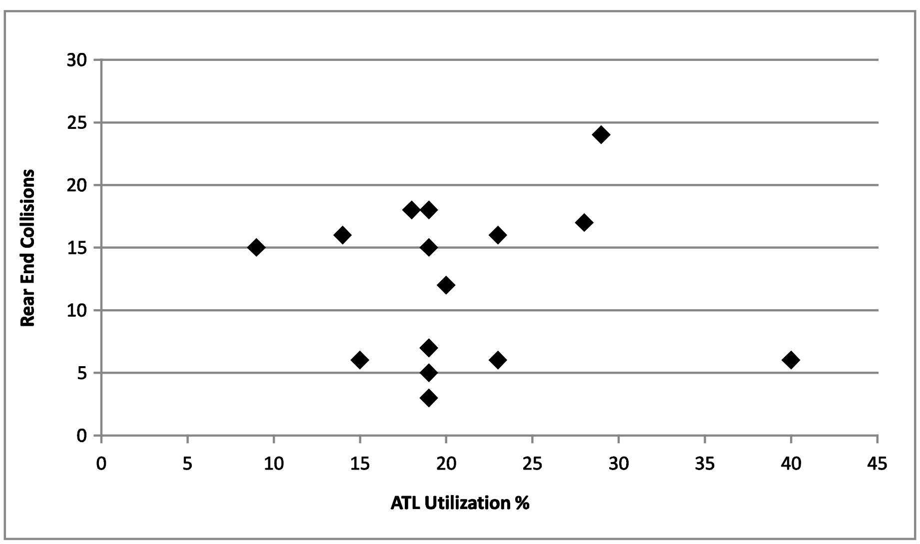


Figure 34. 2006-2008 rear-end collisions versus average ATL utilization

Figure 35 displays the trend between rear end collisions and average ATL flow observed in the field for each of the 16 sites. This shows that ATL flow is positively correlated with total volume; this trend is intuitive and does not necessarily indicate that well-utilized ATLs are unsafe. Also note that the two influential points in the upper right portion of the figure with very high ATL flow are the sites with two CTLs.

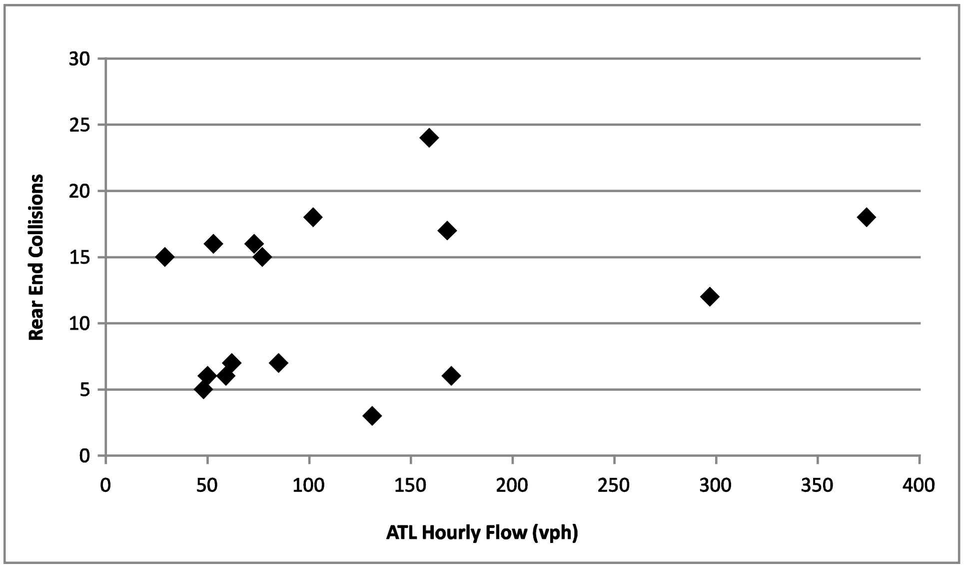


Figure 35. 2006-2008 rear-end collisions versus average ATL hourly flow.

Figure 36 displays the final relationship explored in this section of the analysis, which compares sideswipe collisions at each site to total ATL length. It may be hypothesized that longer ATLs would allow for safer merging, but it is also possible that more exposure to merging areas would lead to more frequent sideswipe collisions. Based on the direction of the linear relationship shown in Figure 36 (again the best-fit line), it may be that the latter hypothesis is stronger than the former.

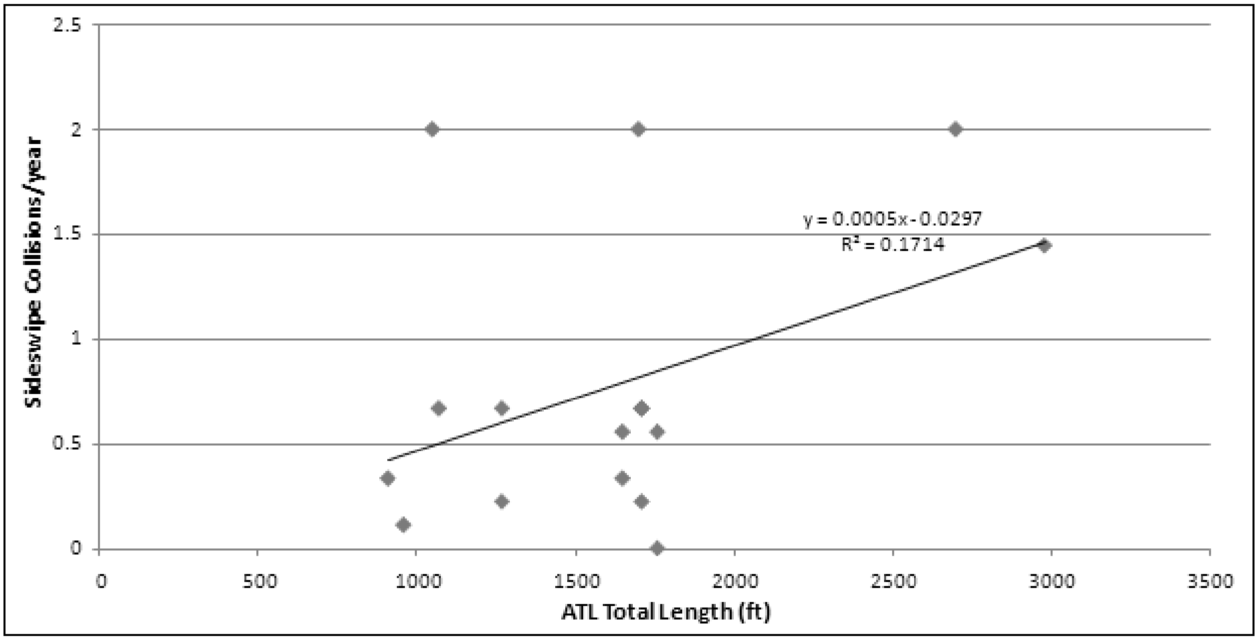


Figure 36. Sideswipe collisions/year versus ATL total length.

In summary, although there was a weak relationship between collisions and congestion level at the ATLs, the data presented here did not show that ATL use and collision frequency were related, and the data did not show that shorter ATLs were less safe.

Comparison of SSAM to Collision Data

The key to this study was the comparison of the SSAM simulation results to the collision data. Table 22contains a breakdown of collisions and SSAM conflicts by approach. Note that the SSAM data were aggregated to adjust for time of day as described earlier.

Table 22. Summary of collision and SSAM data (*denotes site with two CTLs).

#	Approach	Number of Years Analyzed	Rear End Collisions	Sideswipe Collisions	SSAM Rear End Conflicts	SSAM Sideswipe Conflicts	Possible Explanation for Collisions
16	SB MD-2 at Arnold Rd *	9	57	13	4894	460	Downstream bottleneck
15	NB MD-2 at Arnold Rd *	9	45	6	5892	658	
5	SB La Canada Dr at Magee Rd	9	42	5	4419	87	
3	EB NC-54 at Fayetteville Rd	6	41	12	3261	65	Driver confusion
2	WB Walker Rd at Murray Blvd	6	34	2	6492	1967	Downstream bottleneck
8	NB La Canada Dr at Orange Grove Rd	9	33	6	6972	783	
7	WB Magee Rd at La Canada Dr	9	29	5	5055	198	
10	EB Walker Rd at 185th St	9	28	1	3360	510	
11	WB Walker Rd at 185th St	9	27	2	4741	649	
6	EB Magee Rd at La Canada Dr	9	27	3	8493	73	
9	SB La Canada Dr at Orange Grove Rd	9	24	2	11640	672	
1	EB Walker Rd at Murray Blvd	6	23	4	7948	337	
13	NB Garrett Rd at Old Chapel Hill Rd	6	20	12	2976	204	Short ATL with driveways
12	SB Sunset Lake Dr at Holly Springs Rd	6	17	12	5217	270	Collisions reflect left turns into ATL
4	NB La Canada Dr at Magee Rd	9	15	0	5328	222	
14	SB Garrett Rd at Old Chapel Hill Rd	6	12	4	4626	440	
	Total	**126**	**474**	**89**	**91314**	**7594**	

In Table 22 the sites are ranked from the highest to lowest number of rear end collisions. It is interesting to compare the collision data with the SSAM conflicts across sites—in a few instances, conflicts roughly followed collisions. In other comparisons, the lack of a strong relationship can be explained by intersection elements that could not be accounted for in the VISSIM model used for SSAM (see the "Notes" column). For example, a heavy weaving area downstream of approach 1 made for an unusually high number of both rear end and sideswipe collisions. However, in many cases, the reason for a lack of relationship between collisions and SSAM conflicts can most likely be attributed to the low frequency of collisions at these intersections. Indeed, the last row in the table indicates that only 563 collisions were observed over all 16 approaches in nine years (an average of 4.5 collisions/year at each site). Although this low sample size makes it difficult to determine a relationship between SSAM conflicts and collisions, it also indicates that none of these 16 approaches appears to be particularly unsafe. Even the worst site, approach 3, only exhibited 8.8 collisions/year, many of which can probably be attributed to the driver unfamiliarity observed during field data collection, when the team observed several instances of last-minute lane changes and illegal turns during less than six hours of field observation.

Figure 37 compares rear end conflicts with rear end collision data when each data point is aggregated over a 3-year period. The approaches within each state are marked with the same shape—while the data from Arizona and Oregon appear to be widely scattered, there appears to be a weakly negative relationship for the data from Maryland and North Carolina. This suggests that, for the data obtained, SSAM does not strongly predict rear end collision data at this level of aggregation.

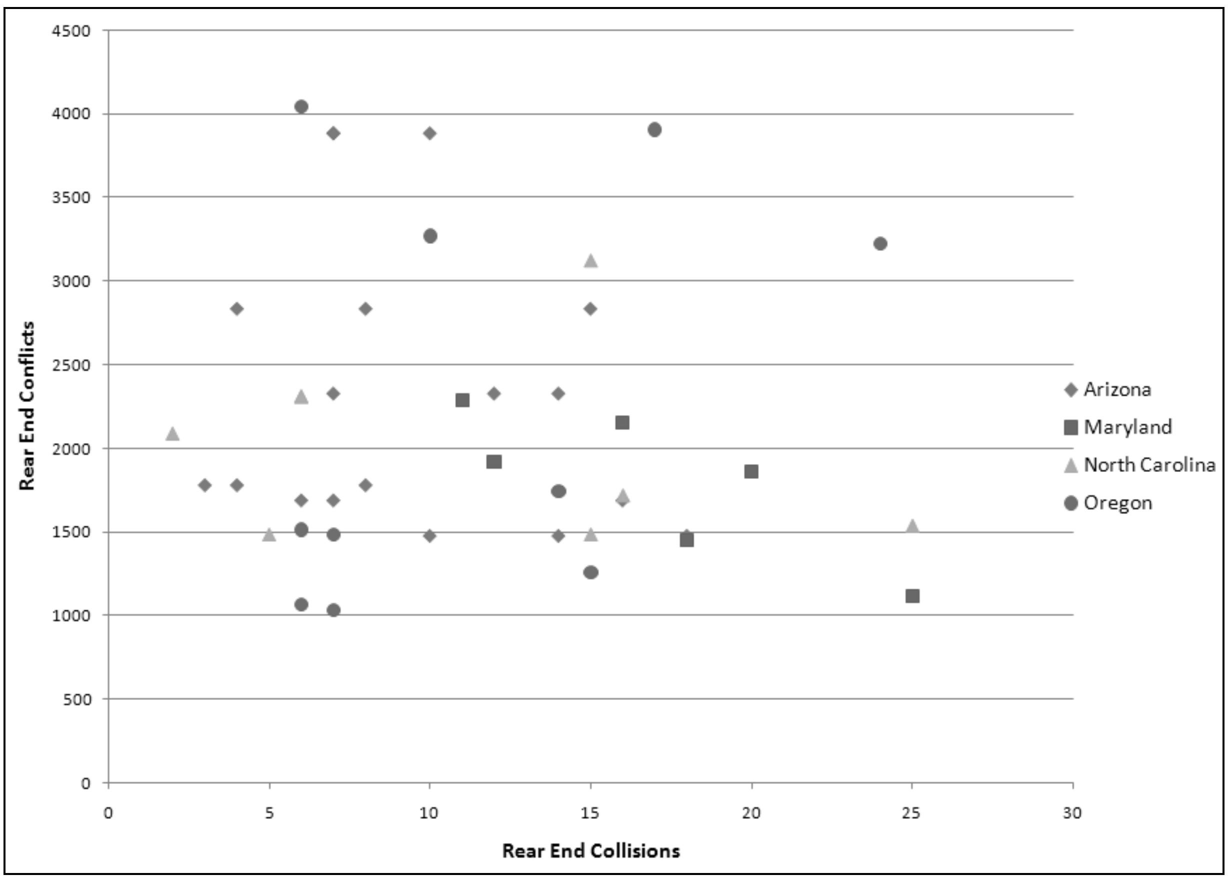


Figure 37. Rear-end conflicts versus collisions.

Figure 38 displays the same plot for sideswipe conflicts and sideswipe collisions. With the exception of the two outlying data points from Oregon, the relationship between conflicts and collisions appears to be a flat line. Note that the low number of observed sideswipe collisions may limit the strength of this comparison.

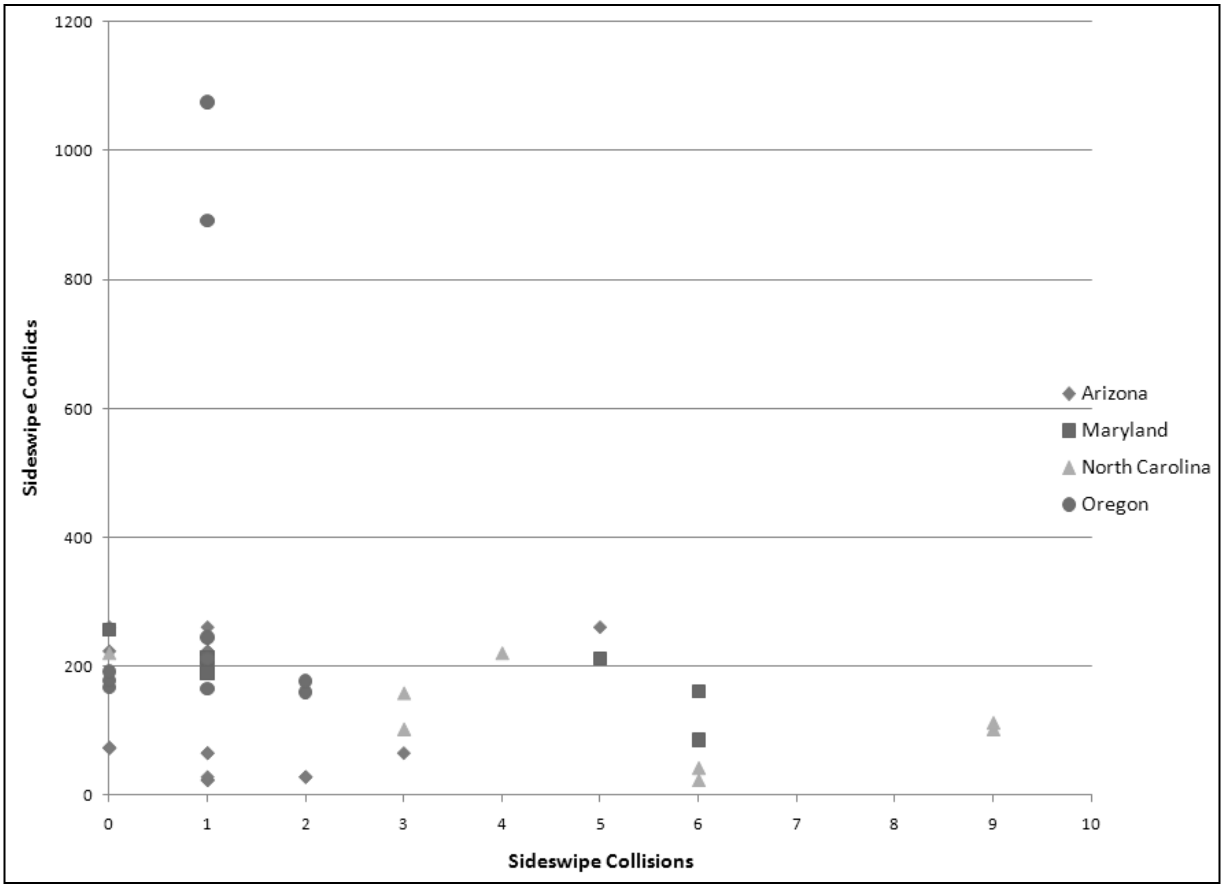


Figure 38. Sideswipe conflicts versus collisions.

A relationship between SSAM and collision frequency only becomes visible at a high level of aggregation. Figure 39 and Figure 40 show conflicts versus collisions (rear end and sideswipe, respectively) using three different methods of aggregation. The first method combines all sites from each state, the second method combines all observations from each time of day, and the third method combines all observations from each 3-year period.

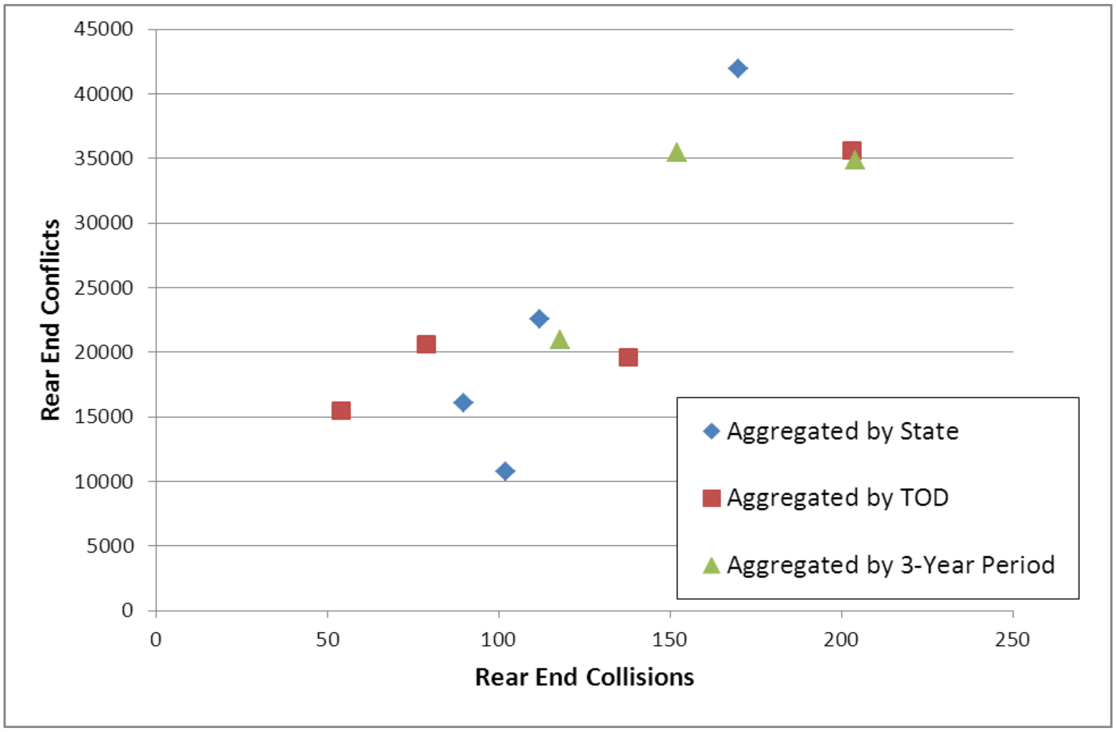


Figure 39. Aggregated rear-end conflicts versus collisions.

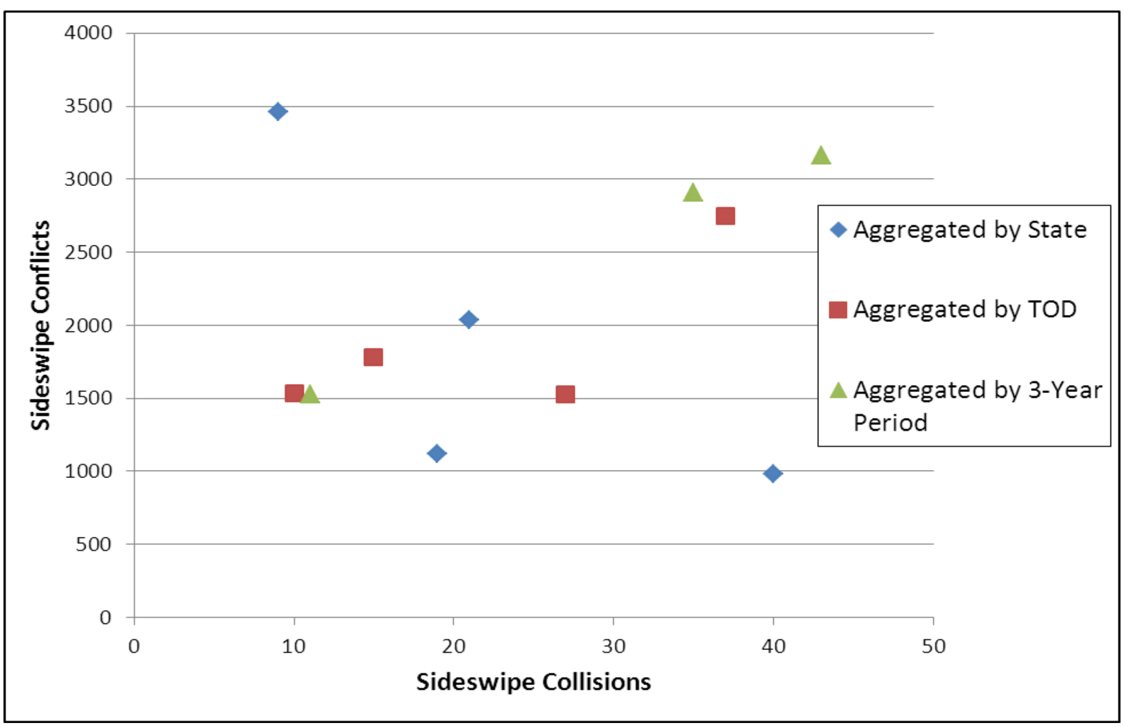


Figure 40. Aggregated sideswipe conflicts versus collisions.

The figures display several linear relationships, the strongest of which is the comparison of sideswipe conflicts versus collisions when aggregated by 3-year period. This relationship was statistically significant at the 95 percent confidence level according to Pearson's Chi-square test (χ^2=3.6, df=2). The low collision frequencies at the studied sites may have made it difficult to achieve statistical significance.

Figure 41 shows the distribution of conflicts by relative position within the ATL. The figure indicates that, in general, SSAM distributed sideswipe conflicts more evenly than rear end conflicts throughout the ATL and shows that there were relatively more upstream rear end conflicts and more downstream sideswipe conflicts.

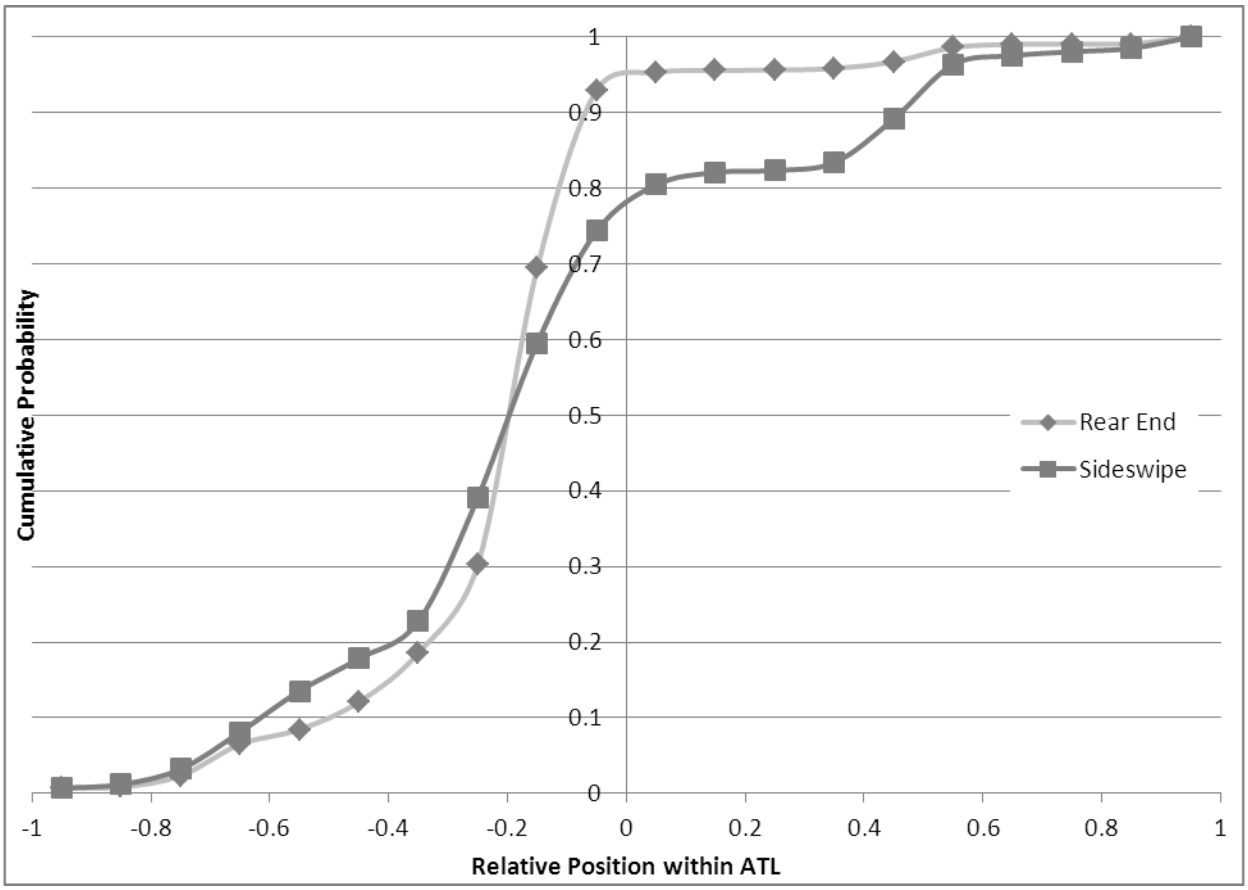


Figure 41. SSAM conflict distribution versus relative position within ATL.

Figure 42 compares the distributions of conflicts and collisions over the downstream ATL length only. Note that conflicts in SSAM occur farther away from the intersection than the reported collisions.

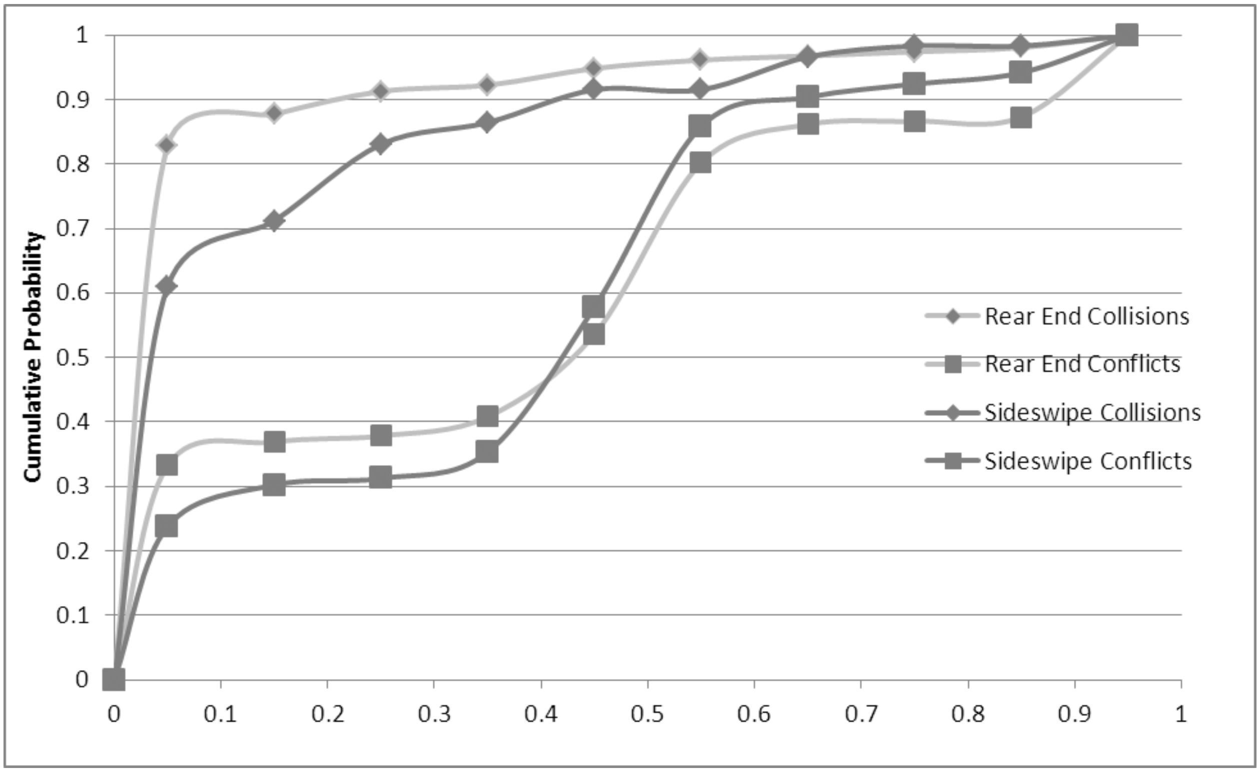


Figure 42. Comparison of conflict and collision distribution over relative downstream ATL length.

In summary, the comparison of SSAM conflicts with field collision data shows that while SSAM can roughly predict collisions at a high level of aggregation (i.e., when there were plenty of collisions to compare with), it was difficult to find a relationship between the collision data and SSAM output when the sample size was low or when the data were compared by position within the ATL.

SSAM Trends

The final part of this work on the safety implications of ATLs involved an analysis of the sensitivity of SSAM conflicts to several design variables. Figure 43 and Figure 44 show how the number of SSAM rear end and sideswipe conflicts, respectively, changed for an exclusive lane with respect to downstream ATL length and v/c. Note that the v/c here is analogous to the value of X_T presented previously. Rear end conflicts remained relatively consistent as downstream length increased, but the number of sideswipe conflicts spiked at a downstream length of 800 feet. This may be due to some quirk in the VISSIM or SSAM logic and the low samples sizes of sideswipe conflicts. Conflicts tended to increase fairly steadily with increasing v/c, as might be expected.

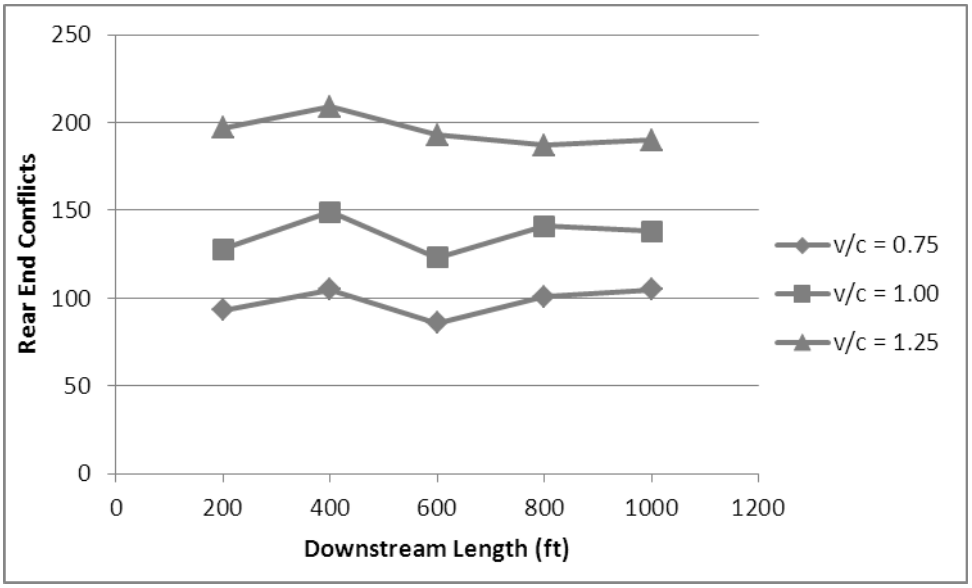


Figure 43. SSAM rear-end conflict comparison (no right turns).

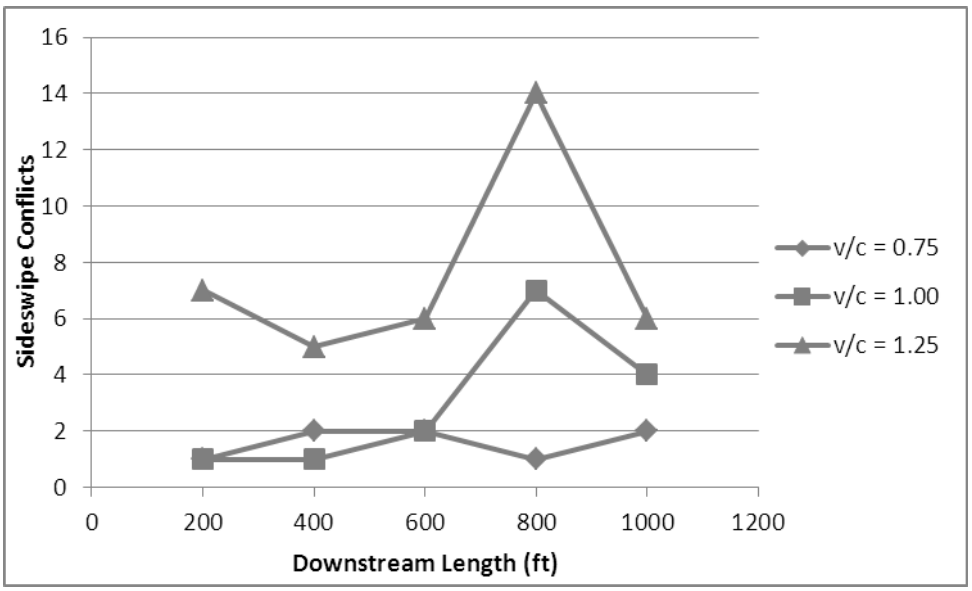


Figure 44. SSAM sideswipe conflict comparison (no right turns).

Figure 45 and Figure 46 show the same comparisons, but for a shared ATL with 200 right turns per hour. The figures indicate that low-to-moderately congested approaches had low levels of conflicts when compared to those simulated at v/c = 1.25. While rear end conflicts remained relatively unaffected by changes in downstream length, the number of sideswipe conflicts tended to increase as downstream length increased. This could be explained by the exposure, as a greater downstream length tended to generate more conflicts in SSAM simply because the conflict area was lengthened. Note that if the number of conflicts was normalized by downstream length, a decreasing trend would emerge. Also note that there were many more conflicts generated by the shared lane than by the exclusive ATL scenario.

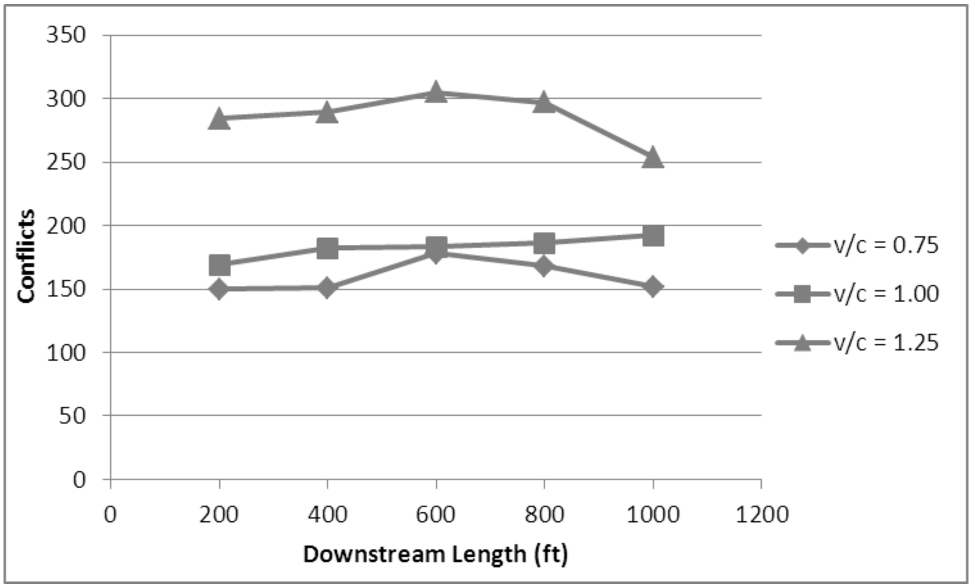


Figure 45. SSAM rear-end conflict comparison (200 right-turning vph).

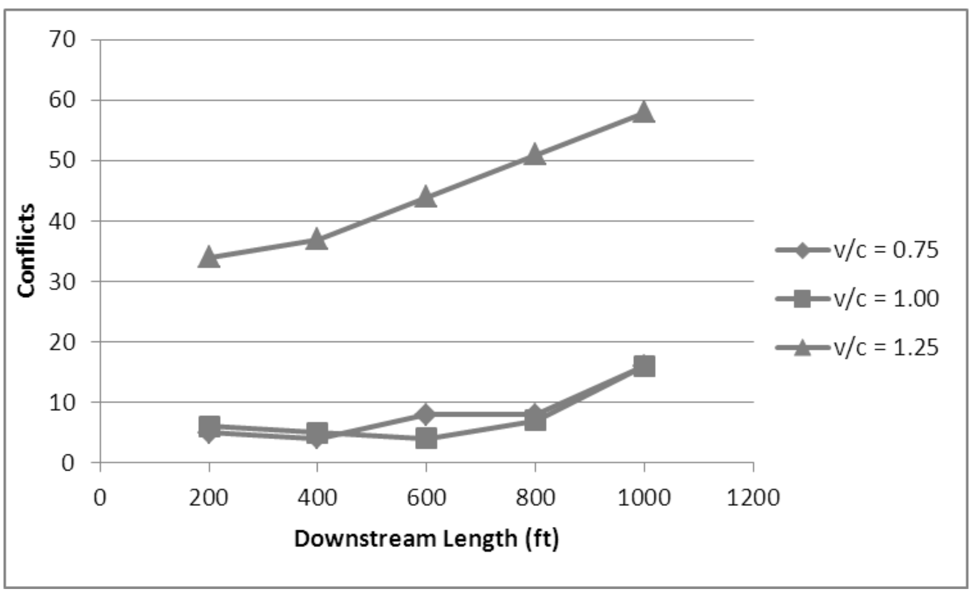


Figure 46. SSAM sideswipe conflict comparison (200 right-turning vph).

Analysis of variance for the rear end conflict data across all levels tested indicated that there were statistically significant interactions between the right turn volume and v/c and between the right turn volume and the downstream length. The former interaction is intuitive, as the effect of right-turning vehicles as a detriment to safety magnifies when there is more through traffic in the shared ATL. In terms of the sideswipe conflicts, multiple significant interactions existed—some of these were unexpected and may be due to the low sample size for sideswipe conflicts. In general, it appears that the SSAM logic confirms intuition. First, it appears that downstream length has little effect on safety. Second, shared ATLs (at least those with 200 or more right-turning vehicles per hour) tend to have more conflicts than exclusive ATLs. Finally, the crash increase with the increase in X_T shown in Figure 33 was supported by the increase in conflicts with v/c shown in Figures 43 through 46, particularly as the v/c increased beyond 1.0.

CONCLUSIONS

The objective of this study was to determine the effects of ATLs on safety and to explore which ATL design parameters had an effect on the safety of the ATL. The key finding of the study is that there was a relatively infrequent occurrence of rear end and sideswipe collisions at each of the sixteen analyzed approaches. This low sample size limited the conclusiveness of the results, but, since ATL-related collisions were hypothesized to consist of only rear end and sideswipe events, this also indicates that the analyzed ATLs do not appear to be unsafe.

The analysis of ATL design and operation with respect to collision data yielded the following conclusions:

- ATL length does not appear to affect collision rate;
- ATL utilization (%) does not appear to affect collisions, although more congested intersections tend to have more collisions as well as higher ATL use;
- Shared ATLs with at least a moderate amount of right-turning vehicles appear to be less safe than exclusive ATLs, and
- Downstream bottlenecks cause safety concerns due to traffic spilling back into the ATL merging area.

The results of the SSAM validation exercise indicate that SSAM conflicts were related to collision data only when highly aggregated, but a validated SSAM model for use at ATLs could potentially be made available to designers if there was enough collision data to constitute a large sample size. To provide more conclusive results, this study should be repeated with a greater number of study sites. An ambitious before-and-after study of ATLs would still be the preferred method for directly determining their effects on safety at signalized intersections if enough qualifying sites could be found. This type of study could be performed at new ATLs with proper foresight or at existing ATLs if historical collision and operational data are still readily available. If such a project is undertaken, researchers should be aware that many ATLs are often constructed as part of new development, and they should be cautious to account for changing traffic patterns with the development.

CHAPTER 8: GUIDELINES

The research team prepared guidelines for practitioners on how to evaluate and design ATLs. The guidelines are accessible via: http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2492.

This chapter describes the scope of the guidelines, their limitations, and provides a summary of chapter contents.

SCOPE OF THE GUIDELINES

Consistent with the overall research effort, these guidelines apply to auxiliary lanes for through movements that begin upstream of a signalized intersection and end downstream of the intersection. It focuses on ATLs that begin with a right-hand add lane upstream of the signal and end with a right-hand tapered merge downstream of the signal.

The guidelines provide practitioners with the tools and guidance needed to answer the following questions:

- What factors affect the use of ATLs?
- How much traffic is likely to use an ATL?
- What is the safety performance of ATLs?
- What tools are available to evaluate operational and safety performance of ATLs?
- What minimum length is needed for the upstream and downstream components of the ATL?
- What signs and pavement markings should be applied on ATLs?
- How can simulation be used to supplement a deterministic analysis of ATLs?

LIMITATIONS OF THE GUIDELINES

The ATL guidelines do not address the following conditions:

- Non-signalized intersections
- Intersections that serve as transitions from either four-lane to two-lane roadways or six-lane to four-lane roadways
- Left- or right-turn lanes with an upstream addition and downstream drop
- Approaches that have more than two CTLs
- Approaches that include shared left–through lanes or downstream facilities where queues extend into the ATL
- Approaches that experience blockage due to downstream conditions

- Approaches that operate within a well-coordinated signal system such that the majority of vehicles arrive during the green phase of the traffic signal.

In addition, the guidelines do not provide statistical or analytical models to predict the number of crashes or conflicts on an ATL. Rather, a summary of crash data obtained for ATL approaches is provided.

Lastly, these guidelines do not provide guidance for applying ATLs relative to other capacity-enhancing intersection treatments.

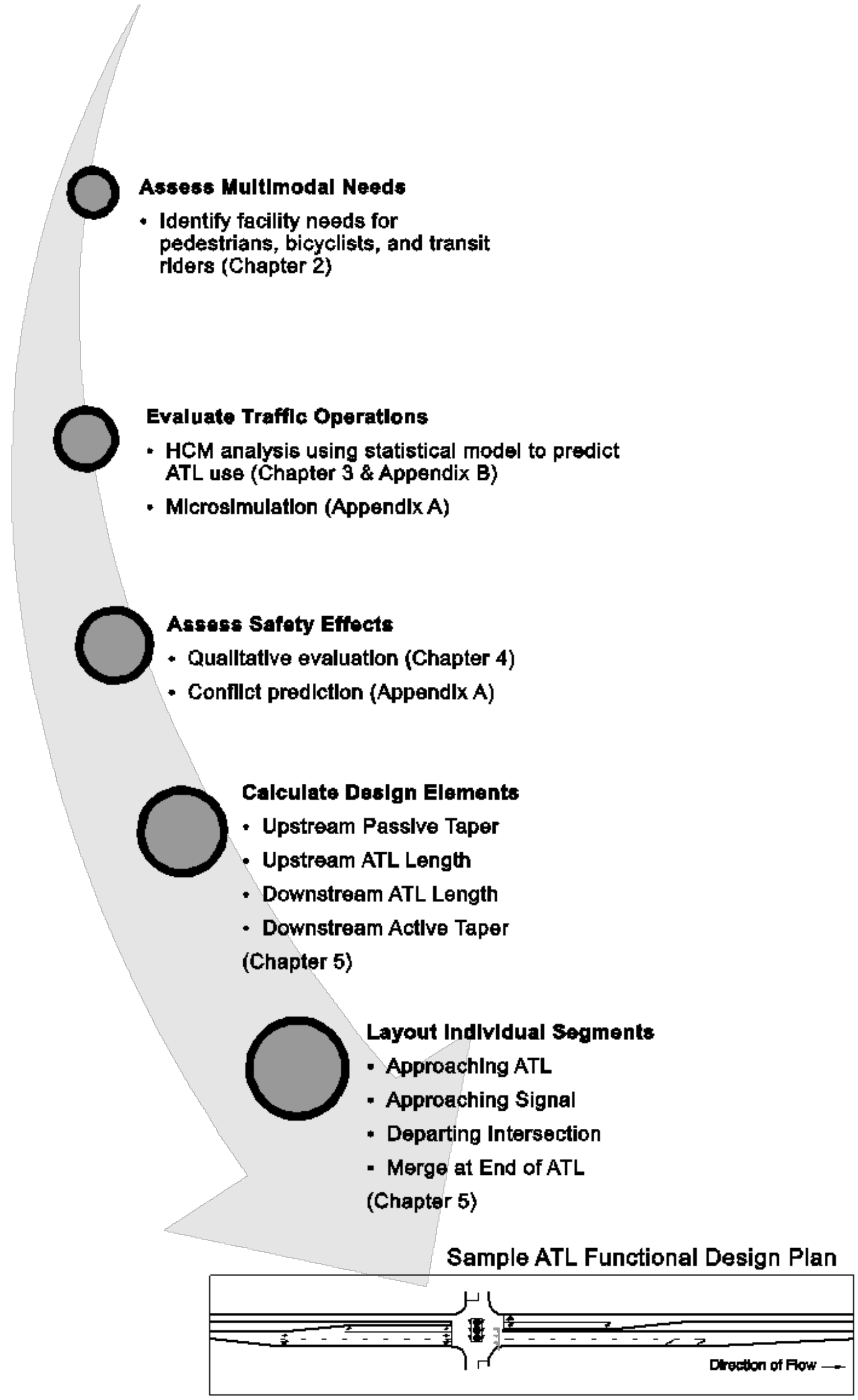
ORGANIZATION OF GUIDELINES

The guidelines are organized to follow a typical analysis and design process for ATLs as shown in Figure 47. Also included is the corresponding chapter that documents the information and procedures needed to carry out the appropriate step in the process.

The title and content for all chapters and appendices are described below:

- **Chapter 1: Introduction.** Describes background, scope, and limitations.
- **Chapter 2: ATL Characteristics.** Describes the operational, safety, and design characteristics of ATLs, as well as needs and considerations for potential ATL user types.
- **Chapter 3: Operational Analysis.** Presents a statistical model for predicting the amount of traffic that will use an ATL for approaches with one or two CTLs.
- **Chapter 4: Safety.** Documents the results from an evaluation of field crash data and discusses geometric and operational factors expected to impact the safety performance of an ATL.
- **Chapter 5: Design**. Describes an approach for preparing a functional design plan for an ATL, provides a method for determining the minimum upstream and downstream ATL length, and presents guidance on signing and pavement markings for ATLs.
- **Chapter 6: Application.** Demonstrates how to apply the operations, safety and design tools, methods, and guidelines to a practical example.
- **Appendix A.** Describes how analysts can use traffic simulation models to estimate the operational performance and, to a limited extent, the safety performance of ATL designs.
- **Appendix B.** Describes the computational engine that carries out the deterministic operational analysis procedure described in Chapter 3.
- **Appendix C.** Describes the method and equations for calculating the minimum required upstream and downstream ATL lengths.

The Guidelines include illustrations showing the placement and type of signs and markings, along with references to the relevant sections of the MUTCD.



NOTES

- No additional data required beyond traditional intersection analysis
- Applicable to approaches with one or two continuous through lanes and an exclusive or shared right-turn lane

Figure 47. Organization of guidelines.

CHAPTER 9: CONCLUSIONS AND RECOMMENDATIONS

This section documents the key conclusions from the research, a plan for implementing the research, and recommendations for future research.

KEY CONCLUSIONS

- ATLs are applied at signalized intersections throughout the United States as a congestion-relief treatment to mitigate bottlenecks along arterials. While many city, county, and state agencies have implemented ATLs, prior to this research project there was a lack of guidance to enable practitioners to estimate the amount of traffic expected to use the ATL and methods to calculate key design elements such as upstream and downstream ATL lengths and to develop signing and pavement marking plans specific to ATLs.

- Field data collected across 22 ATL approaches revealed that ATL use is most heavily influenced by the <u>volume of through traffic</u> on the approach and the <u>demand-to-capacity ratio for through movements</u> assuming all through demand across the CTL(s). Other factors suggested in previous literature and hypothesized by the research team, such as travel time savings gained by using the ATL and length of ATL was not found to significantly influence the use of ATLs at the study sites.

- ATL lane-use prediction models were developed for 1-CTL and 2-CTL approaches. These models explain approximately 80 percent of the variability from the observed study ATL approaches. While the validation exercise was performed using data collected from the same data set that the lane use prediction models were developed from, they do not represent the same data because approximately 25 percent of data were withheld for validation purposes.

- The operational analysis procedure described in the Guidelines predicts the volume of traffic in the ATL and can be used to estimate performance measures for the ATL and CTL(s).

- The ATL computational engine developed as part of this project enables practitioners to apply the ATL operational and design procedures to estimate capacity, delay, level of service, and queuing in the ATL and CTL as well as estimate the minimum upstream and downstream length of the ATL.
- Microsimulation tools such as VISSIM (10) can be applied to evaluate the operational and safety performance of ATLs with appropriate modifications to lane changing behavior to account for under-utilization in the ATL. The Guidelines describe an approach to calibrate the Lane Change Distance parameter in VISSIM to approximate the expected lane utilization of an ATL. Further, FHWA's SSAM model can be applied to a calibrated microsimulation model to predict conflict frequency and type. The results of the SSAM validation exercise indicate that SSAM conflicts were related to collision

data only when highly aggregated, but a validated SSAM model for use at ATLs could potentially be made available to designers if there was enough collision data to constitute a large sample size.

- The results from a safety analysis found a relatively infrequent occurrence of rear end and sideswipe collisions at each of the sixteen analyzed approaches. This low sample size limited the conclusiveness of the results, but, since ATL-related collisions were hypothesized to consist of only rear end and sideswipe events, this also indicates that the analyzed ATLs do not appear to be unsafe.
- There are no current methods available to practitioners (based on the research team's knowledge) to estimate downstream ATL length. Given this gap, the research team developed a theoretical model to estimate the downstream ATL length based on two conditions. DSL1 estimates the downstream ATL length needed to allow vehicles to reach a desired merge speed when starting from a stopped position. DSL2 estimates the downstream ATL length needed to provide adequate gaps for a driver in the ATL to merge into the CTL when approaching the intersection on green without any queue present (i.e., uninterrupted flow conditions). The greater of the two values of DSL1 and DSL2 dictate the recommended minimum downstream ATL length. This method for estimating downstream ATL length based on the greater of two parameters, DSL1 and DSL2, is presented as an option to the practitioner and requires further research and validation before it is recommended to be incorporated into local agency design manuals and national resource documents such as the AASHTO Green Book.
- No changes are recommended to the AASHTO Green Book or MUTCD as part of this research effort.

IMPLEMENTATION STEPS

The following steps are recommended to implement the results from this research activity into practice.

- Send agency partners and survey respondents the Guidelines link (when published) and encourage practitioners to apply the Guidelines
- Incorporate the Guidelines into Volume IV of the HCM 2010 (near term) and into the next revision of the HCM (long term)
- Conduct webinars through Transportation Research Board to disseminate results to practitioners
- Present findings to TRB committees and subcommittees
    - Operational Effects of Geometric Design (AHB65)
    - Geometric Design (AFB10)
    - Highway Capacity and Quality of Service (AHB40)

- Present conference and journal papers at TRB and ITE conferences on the following topics :
    - High level guidelines approach
    - Operations Method (deterministic and simulation approach)
    - Safety Method
    - Design Method

RECOMMENDATIONS FOR FUTURE RESEARCH

This research effort is the first of its kind to provide practitioners with methods and procedures to assist in assessing the need for, and the evaluation and design of ATLs. As such, further application and testing is desired to validate and enhance the Guidelines and expand their application to other related treatment types.

The following items summarize our team's recommendations for future research to build and expand upon the work conducted as part of NCHRP 3-98:

- Test and validate the ATL lane-use prediction models for 1-CTL and 2-CTL sites beyond the 22 ATL approaches examined as part of the NCHRP 3-98 research effort. The sites should experience peak hour congestion such that X_T is greater than or equal to 1.0 and include a mix of shared and exclusive right-turn movements.
- Investigate the effects of factors that were found to lead to lower ATL use such as driveway activity, vertical grades, sight distance obstructions or restrictions, heavy right-turn movements at the intersection, and good signal progression (see Guidelines page 1-3 regarding limitations of prediction methods for well-coordinated signal systems). Incorporate adjustments as necessary to the 1-CTL and 2-CTL prediction equations.

- Test and monitor the effect of strategies intended to increase the use of ATLs. These strategies include:

    - Placement of side-mounted lane use signs on the upstream ATL approach and overhead signs on mast arms/span wires to encourage drivers to use the ATL as a through lane.

    - Implementation of alternate merge signs to reduce the use of ATLs as a passing lane and provide equal priority for merging to further incentivize use of the ATL.

  - Reduction of the green time on ATL approaches to encourage more traffic to use the ATL in order to avoid waiting multiple cycles. This can be done by reallocating green time to other movements at the intersection
- Incorporate the ATL lane use prediction method into the next update of the Highway Capacity Manual and embed the analytical routines within the computational engine for the signalized intersection chapter. This will streamline the analysis and evaluation process for software users and ease the software implementation process for software developers while at the same time increasing the accessibility of the method to a wider base of potential users.
- Develop a crash modification factor (CMF) for ATL installation. Methodologically, this should not be difficult, as ATLs are rarely installed as crash countermeasures and therefore regression to the mean should not be a great threat to the validity of a CMF estimated using before and after crash data. The difficulty in developing a CMF will be in finding a large enough sample of sites where the conversion was made from conventional geometry to ATL and where no other important design changes were made at the same time. During this study, the research team was not able to find any such sites, as ATLs were always installed in conjunction with other improvements. If researchers were able to complete the difficult task of estimating a CMF, planners and designers would be in a much stronger position to evaluate potential ATL installations than they are now.
- Increase the sample size of reported ATL-related crashes from this study and conduct additional SSAM analysis to determine if a relationship can be developed between SSAM conflict predictions and reported crashes for ATL approaches.
- Test and validate the models to estimate downstream ATL length (DSL1 and DSL2). The models should be tested on both 1- and 2-CTL approaches and across a range of congestion levels and approach speeds.
- Expand the Guidelines to address related ATL configurations to determine the extent to which the Guidelines are applicable to those conditions and whether adjustments are needed. Related configurations include:
  - Dual left-turn lanes that merge from two-to-one or three-to-two lanes (from major or minor streets)
  - Dual right-turn lanes that merge from two-to-one or three-to-two lanes (from major or minor streets)
  - "Half" auxiliary lanes
    - Full upstream lane with downstream merge
    - Upstream "add lane" with full downstream lane
  - Downstream trap lanes that end as a downstream right-turn lane only maneuver
  - ATLs located to the left of the CTL

- o “Offset” ATLs (ATL lanes that are added to the left of the CTL and merged to the right of the CTL downstream, or vice versa

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APPENDIX A: WEB-BASED SURVEY RESULTS

Contact 1			Location of Intersection											Type of ATL							Data available							
Name	Organization	Agency Type	Intersection ID	Approach ID	N/S Roadway	E/W Roadway	Approach	Latitude	Longitude	City	State	Region of Country	Context	Nb of Basic Downstream Through Lanes	Nb of Basic Upstream Through Lanes	Posted Speed	Upstream transition	Downstream transition	Upstream right turn type	Comment	Trafic counts	Crash Data	Signal Timing Data	Speed Information	Previous Study	Anecdotal Evidence	Other Info	ATLs Guidelines in agency?
Dr. Rashad M. Hanbali, PTOE	City of Cape Coral	City	1	1	Delprado Blvd	Cape Coral Prkwy	SBLT	26.562608	-81.943885	City of Cape Coral	Florida	SE	Urban	2	0		Left turns	RH drop	N/A		Yes	Maybe	Yes	Maybe	Maybe			No
Saed Rahwanji	MD -State Highway Admin.	DOT	2	2	Branch Ave (SR 5)	Surratts Rd	SB	38.7504018	-76.87957764	Prince George's County	Maryland	NE	Suburban	2	2		Th add	LH drop	RT pocket/channelized		Yes	Yes	Yes					No
Saed Rahwanji	MD -State Highway Admin.			3	Branch Ave (SR 5)	Surratts Rd	SBLT	38.7504018	-76.87957764	Prince George's County	Maryland	NE	Suburban	1	0		Left turns	RH drop	N/A		Yes	Yes	Yes					No
Saed Rahwanji	MD -State Highway Admin.		3	4	Three Notch Rd (SR235)	Patuxent Beach Rd (SR 4)	NB	38.30313938	-76.51908875	St Mary's county	Maryland	NE	Suburban	1	2		Th basic	RH drop	RT pocket/channelized		Yes	Yes	Yes					No
Saed Rahwanji	MD -State Highway Admin.			5	Three Notch Rd (SR235)	Patuxent Beach Rd (SR 4)	NBLT	38.30313938	-76.51908875	St Mary's county	Maryland	NE	Suburban	1	0		Left turns	RH drop	N/A		Yes	Yes	Yes					No
Saed Rahwanji	MD -State Highway Admin.		5	6	Montgomery Rd (SR 103)	Waterloo Rd (SR 104)	WB	39.23627528	-76.79778099	Howard County	Maryland	NE	Suburban	1	0		Left turns	RH merge	N/A	No left-turn striping	Yes	Yes	Yes					No
Brian Hadley	Caltrans	DOT	7	7	I-15 NB ramp	SR56 / Ted Williams Prkwy	WB	32.964706	-117.095384	San Diego County	California	SW	Suburban	1	2		Th basic	RH drop	RT pocket	Interchange	Maybe	Maybe	Yes	No	No	Yes	Yes	No
Erwin Madlangbayan	California Department of Transportation (Caltrans)	DOT	8	8	Rio Vista Hwy (SR12)	Summerset Rd	NB	38.18174009	-121.7232478	Solano County	California	SW	Rural	1	1		Th add	RH merge	RT pocket		Yes	Yes	Yes	Yes	Maybe	No	Maybe	Yes
Erwin Madlangbayan	California Department of Transportation (Caltrans)			9	Rio Vista Hwy (SR12)	Summerset Rd	SB	38.18174009	-121.7232478	Solano County	California	SW	Rural	1	1	50	Th add	RH merge	No RT									
Zhongren Wang	Caltrans	DOT	9	10	Stony Point Rd	Gravenstein Highway (CA116)	WB	38.334522	-122.738245	Sonoma County	California	SW	Rural	1	1		Th add	RH merge	RT pocket		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans			11	Stony Point Rd	Gravenstein Highway (CA116)	EB	38.334522	-122.738245	Sonoma County	California	SW	Rural	1	1	55	Th add	RH merge	Shared ThRT		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		10	12	Farmers Ln (SR12)	Fourth Street	EB	38.449672	-122.688458	Santa Rosa	California	SW	Urban	2	2		Th add	RH merge	RT pocket		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		11	13	Sonoma Hwy (SR12)	Los Alamos Rd	EB	38.458173	-122.63421	Santa Rosa	California	SW	Urban	1	2		Th basic	LH merge	RT channelized		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		12	14	Sonoma Hwy (SR12)	Verano Ave	NB	38.302699	-122.476102	Sonoma County	California	SW	Suburban	1	2		Th basic	RH merge	RT channelized	Distance between Verano & Ramon: 1620'	Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		13	15	Sonoma Hwy (SR12)	Ramon St	SB	38.298499	-122.47532	Sonoma County	California	SW	Urban	1	2		Th basic	RH drop	RT pocket	Distance between Verano & Ramon: 1620'	Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		14	16	Sonoma Hwy (SR12)	Napa St	SB	38.293667	-122.475809	Sonoma County	California	SW	Urban	1	0		Left turns	LH merge	N/A		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		15	17	Sonoma Hwy (SR12)	Airport Blvd (SR 29)	NB	38.22314	-122.257857	Napa County	California	SW	Rural	2	2		Th add	RH merge	RT pocket		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		16	18	Redwood Hwy (US101) SB ramps	Shoreline Hwy (SR1)	WB	37.880409	-122.517919	Marin County	California	SW	Suburban	1	1	35	Th add	RH merge	No RT		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans			19	Redwood Hwy (US101) SB ramps	Shoreline Hwy (SR1)	NBLT	37.880409	-122.517919	Marin County	California	SW	Suburban	1	0		Left turns	RH merge	N/A		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		17	20	I-5 NB on-ramp	SR4/Charter Way	EB	37.936583	-121.29827	San Joaquin County	California	SW	Urban	2	2		Th add	LH drop	RT pocket/channelized	Interchange	Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		18	21	Lincoln Str	SR4/Charter Way	EB	37.9375	-121.293807	San Joaquin County	California	SW	Urban	2	2	40	Th add	RH merge	Shared ThRT		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		19	22	I-5 SB ramps	SR 12	WB	38.116005	-121.400805	San Joaquin County	California	SW	Urban	1	2		Th basic	LH merge	No RT	Interchange	Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		20	23	Thornton Rd	SR 12	EB	38.115984	-121.392684	San Joaquin County	California	SW	Rural	1	2		Th basic	RH merge	RT pocket	Interchange	Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans			24	Thornton Rd	SR 12	EBLT	38.115984	-121.392684	San Joaquin County	California	SW	Rural	1	0		Left turns	RH merge	N/A		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		21	25	CA88	Eight Mile Rd	NB	38.057997	-121.188698	San Joaquin County	California	SW	Rural	1	1	55	Th add	RH merge	Shared ThRT		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans			26	CA88	Eight Mile Rd	SB	38.057997	-121.188698	San Joaquin County	California	SW	Rural	1	1		Th add	RH merge	RT pocket		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		22	27	CA88	CA12	NB	38.138641	-121.162394	San Joaquin County	California	SW	Rural	1	1	-	Th add	RH merge	Shared ThRT		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans			28	CA88	CA12	WBLT	38.138641	-121.162394	San Joaquin County	California	SW	Rural	1	0		Left turns (1 shared thru)	RH merge	N/A		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		23	29	Jack Tone Rd	CA120	EB	37.797667	-121.143161	San Joaquin County	California	SW	Rural	1	1		Th add	RH merge	RT channelized	Rural: ATLs of 1100'	Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans			30	Jack Tone Rd	CA120	WB	37.797667	-121.143161	San Joaquin County	California	SW	Rural	1	1		Th add	RH merge	RT channelized	Rural: ATLs of 1100'	Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		24	31	Escalon Bellota Road	CA120	EB	37.797376	-120.996832	San Joaquin County	California	SW	Suburban	2	2		Th add	RH merge	RT pocket		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans			32	Escalon Bellota Road	CA120	WB	37.797376	-120.996832	San Joaquin County	California	SW	Suburban	2	2		Th add	RH merge	RT pocket		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans			33	Escalon Bellota Road	CA120	SB	37.797376	-120.996832	San Joaquin County	California	SW	Suburban	2	3		Th basic	RH merge	RT channelized		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		25	34	CA108	CA219	NB	37.711005	-120.994909	Stanislaus County	California	SW	Suburban	2	3		Th basic	RH merge	RT pocket		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Zhongren Wang	Caltrans		26	35	commercial dvwy (near Maag Ave)	CA120	EB	37.773577	-120.828774	Stanislaus County	California	SW	Suburban	1	2		Th basic	RH merge	RT channelized		Yes	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Inga Note	City of Spokane Valley	City	29	36	Sullivan Rd	Sprague Ave	EB	47.65703409	-117.1966338	Spokane County	Washington	NW	Suburban	2	3		Th basic	RH merge	Shared ThRT		Yes	Yes	Yes	No	No	No	No	No
Inga Note	City of Spokane Valley		30	37	Argonne Rd	Trent Ave	NB	47.6810353	-117.2827864	Spokane County	Washington	NW	Suburban	2	3		Th basic	RH merge	RT pocket		Yes	Yes	Yes	No	No	No	No	No
Inga Note	City of Spokane Valley		31	38	Thermane St	Sprague Ave	WB	47.65696182	-117.3148656	Spokane County	Washington	NW	Suburban	3	4		Th basic	LH merge	Shared ThRT		Yes	Yes	Yes	Yes				No
Inga Note	City of Spokane Valley		32	39	US27	32nd Ave	EB	47.62777266	-117.2235847	Spokane County	Washington	NW	Rural	1	1		Th add	RH drop	Shared ThRT		Yes	Yes	Yes	No	No	No	No	No
Dirk Gross	Ohio DOT	DOT	34	40	Riverside Drive	Hayden Run Road	NB	40.06821	-83.105102	Columbus	Ohio	NE	Suburban	1	1	50	Th add	RH merge	Shared ThRT		Maybe	Yes	Maybe	Maybe	Maybe	Maybe	Maybe	No
Dirk Gross	Ohio DOT	DOT		41	Riverside Drive	Hayden Run Road	SB	40.06821	-83.105102	Columbus	Ohio	NE	Suburban	1	1		Th add	RH merge	RT pocket		Maybe	Yes	Maybe	Maybe	Maybe	Maybe	Maybe	No
Peter J. Yauch, P.E.	Pinellas County Public Works	County	35	42	119th Street	Ulmerton Road (SR 688)	EB	27.894623	-82.803601	Largo	Florida	SE	Suburban	2	3		Th basic	RH drop	Shared THRT		Yes	Yes	Yes	No	No	Maybe		No
Peter J. Yauch, P.E.	Pinellas County Public Works		36	43	101st Street	Ulmerton Road (SR 688)	EB	27.894617	-82.78306	Largo	Florida	SE	Suburban	2	3		Th basic	RH drop	Shared ThRT	2610' ATL lanes in front of a regional mall	Yes	Yes	Yes	No	No	Maybe		No
Peter J. Yauch, P.E.	Pinellas County Public Works		37	44	Seminole Boulevard (SR 595)	Ulmerton Road (SR 688)	WB	27.894617	-82.78306	Largo	Florida	SE	Suburban	2	3		Th basic	RH merge	RT pocket	2610' ATL lanes in front of a regional mall	Yes	Yes	Yes	No	No	Maybe		No
Peter J. Yauch, P.E.	Pinellas County Public Works		38	45	US 19	Ulmerton Road (SR 688)	EB	27.893733	-82.722471	Largo	Florida	SE	Suburban	2	2		Th add	RH merge	RT pocket	Interchange	Yes	Yes	Yes	No	No	Maybe	Maybe	No
Peter J. Yauch, P.E.	Pinellas County Public Works			46	US 19	Ulmerton Road (SR 688)	WB	27.893733	-82.722471	Largo	Florida	SE	Suburban	2	2		Th add	RH merge	RT pocket	Interchange	Yes	Yes	Yes	No	No	Maybe	Maybe	No
Peter J. Yauch, P.E.	Pinellas County Public Works		39	47	Missouri (US 19)	Court Street (SR 60)	WB	27.960814	-82.7876	Clearwater	Florida	SE	Suburban	1	1	-	Th add	RH merge	Shared ThRT									No

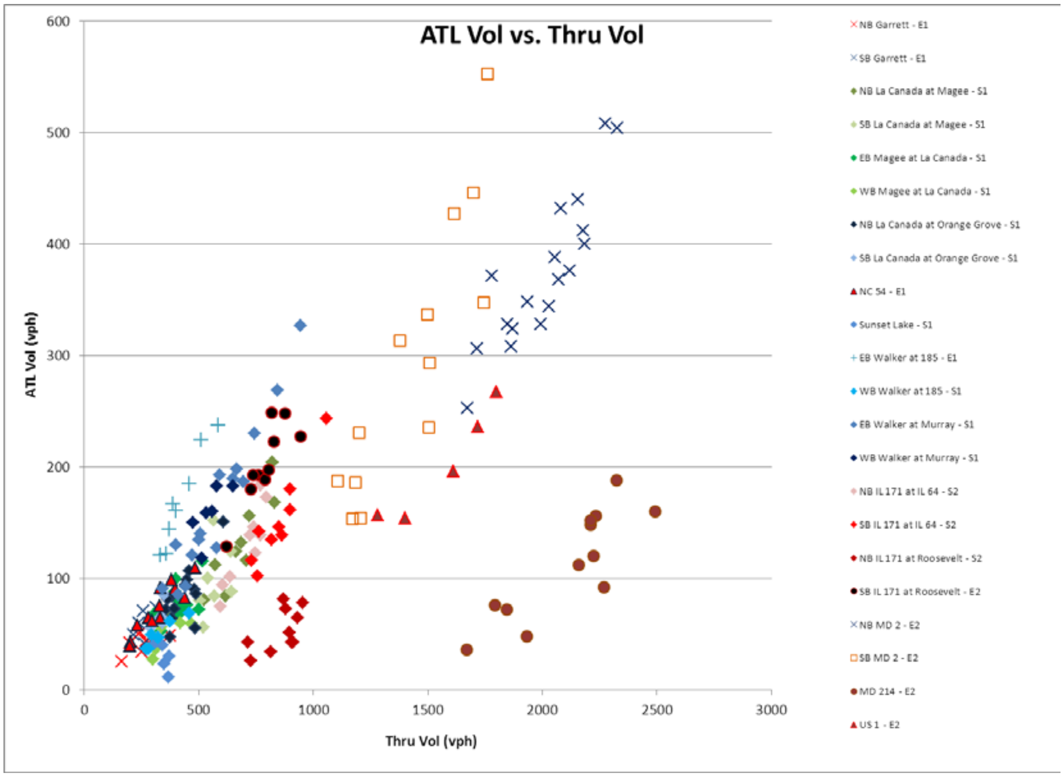
Contact 1			Location of Intersection											Type of ATL							Data available							
Name	Organization	Agency Type	Intersection ID	Approach ID	N/S Roadway	E/W Roadway	Approach	Latitude	Longitude	City	State	Region of Country	Context	Nb of Basic Downstream Through Lanes	Nb of Basic Upstream Through Lanes	Posted Speed	Upstream transition	Downstream transition	Upstream right turn type	Comment	Trafic counts	Crash Data	Signal Timing Data	Speed Information	Previous Study	Anecdotal Evidence	Other Info	ATLs Guidelines in agency?
Mo Farhat	Pima County Dept. of Transportation, Traffic Engineering Division	City	40	48	La Canada Dr	Orange Grove Rd	SB	32.32310604	-110.9951949	Tucson	Arizona	SW	Urban	1	1	45	Th add	RH merge	Shared ThRT		Yes	Yes	Yes	Yes				No
Diahn L. Swartz	City of Tucson Department of Transportation	City	41	49	Tucson Blvd	Speedway Blvd	NB	32.23608386	-110.9352958	Tucson	Arizona	SW	Urban	1	1		Th add	RH merge	RT pocket		Yes	Yes	Yes	Maybe				No
	FDOT District 7 Planning and Modal Development	Consultant	42	50	Nebraska Ave	Hillsborough Ave	WB	27.99605452	-82.45113194	Tampa	Florida	SE	Urban	2	3		Th basic	RH drop	Shared ThRT		Yes	Maybe	Yes	Maybe	Yes	Maybe	Maybe	
	FDOT District 7 Planning and Modal Development		43	51	Nebraska Ave	Floribraska Ave	EB	27.98144089	-82.45114803	Tampa	Florida	SE	Urban	1	2		Th basic	RH merge	Shared ThRT		Yes	Maybe	Yes	Maybe	Yes	Maybe	Maybe	No
John F. Carey	Connecticut Department of Transportation	DOT	44	52	Farmington Ave	Town Farm Rd	NB	41.73199144	-72.83243001	Hartford County	Connecticut	NE	Rural	1	1	40	Th add	RH merge	Shared ThRT		Maybe	Yes	Yes	Maybe				No
John F. Carey	Connecticut Department of Transportation			53	Farmington Ave	Town Farm Rd	SB	41.73199144	-72.83243001	Hartford County	Connecticut	NE	Rural	2	2	40	Th add	RH merge	No RT									
Saed Rahwanji	MD -State Highway Admin.		45	54	Telegraph Rd (MD170)	Reece Rd (MD174)	EB	39.13664097	-76.68577731	Baltimore	Maryland	NE	Urban	1	1		Th add	RH merge	RT channelized									No
				55	Telegraph Rd (MD170)	Reece Rd (MD174)	WB	39.13664097	-76.68577731	Baltimore	Maryland	NE	Urban	1	1		Th add	RH merge	RT pocket	SB RT channelized drop as a downstream trap								
				56	Telegraph Rd (MD170)	Reece Rd (MD174)	NB	39.13664097	-76.68577731	Baltimore	Maryland	NE	Urban	1	1		Th add	RH merge	RT pocket									
Saed Rahwanji	MD -State Highway Admin.		46	57	Ridge Rd	Anapolis Rd (MD175)	NB	39.13051174	-76.73834324	Baltimore	Maryland	NE	Urban	1	2		Th basic	RH drop	RT channelized									No
Saed Rahwanji	MD -State Highway Admin.		47	58	Anapolis Rd (MD 175)	Reece Rd (MD174)	NB	39.11358388	-76.72680974	Fort Meade	Maryland	NE	Urban	1	2		Th basic	RH merge	RT channelized									No
Saed Rahwanji	MD -State Highway Admin.		48	59	Crain Hwy (MD3) SB	Riedel Rd/ Waugh Chapel Rd	WB	39.03923632	-76.67524159	Anne Arundel County	Maryland	NE	Suburban	2	0		Left turns	RH merge	N/A									No
Saed Rahwanji	MD -State Highway Admin.		49	60	Telegraph Rd (MD170)	Patuxent Fwy (MD32)	NB	39.10022184	-76.69403851	Anne Arundel County	Maryland	NE	Suburban	1	2		Th basic	RH drop	No RT									No
Saed Rahwanji	MD -State Highway Admin.		50	61	Cedar Ln	Patuxent Fwy (MD32)	NB	39.1850839	-76.89566016	Howard County	Maryland	NE	Rural	1	2		Th basic	RH merge	No RT	2 lanes+merge markg before int. & 1 lane after intersection.								No
Saed Rahwanji	MD -State Highway Admin.		51	62	Sanner Rd	Guilford Rd	SB	39.1817491	-76.90038085	Howard County	Maryland	NE	Rural	1	2		Th basic	RH merge	RT pocket									No
Saed Rahwanji	MD -State Highway Admin.		52	63	Hartford Rd	Joppa Rd	NB	39.39475368	-76.52330518	Baltimore County	Maryland	NE	Suburban	1	2		Th basic	RH merge	RT pocket									No
Saed Rahwanji	MD -State Highway Admin.		53	64	Piney Orchard Pkwy	New Waugh Chapel Rd	SB	39.07186803	-76.71203882	Anne Arundel County	Maryland	NE	Suburban	1	0		Left turns	RH merge	Shared ThRT									No
Saed Rahwanji	MD -State Highway Admin.		54	65	Loch Raven Blvd (MD542)	commercial drwy near Taylor Ave	NB	39.38378777	-76.57778621	Baltimore County	Maryland	NE	Suburban	2	3		Th basic	RH drop	Shared ThRT									No
Saed Rahwanji	MD -State Highway Admin.		55	66	Hillen Rd	Cold Spring Ln	EB	39.34584776	-76.58607423	Baltimore	Maryland	NE	Urban	1	2		Th basic	RH merge	Shared ThRT									No
Saed Rahwanji	MD -State Highway Admin.		56	67	Anapolis Rd (MD175)	Ridge Rd (MD713) to Disney Rd	SB	39.12814604	-76.7358005	Anne Arundel County	Maryland	NE	Rural	1	1	40	Th add	RH merge	Shared ThRT	Th basic lanes length between Disney and Ridge: 1130'								No
Saed Rahwanji	MD -State Highway Admin.			68	Anapolis Rd (MD175)	Disney Rd to Ridge Rd (MD713)	NB	39.13049301	-76.73837543	Anne Arundel County	Maryland	NE	Rural	1	1	40	Th add	RH merge	Shared ThRT	Th basic lanes length between Disney and Ridge: 1130'								No
Alex Georgevitch	City of Medford			69	Springbrook Rd	E McAndrews Rd	EB	42.34199601	-122.8458059	Medford	Oregon	NW	Suburban	1	2		Th basic	LH merge	Shared ThRT		Yes	Yes	Yes	Yes	Yes	Yes	Maybe	No
Jon Cheney	County of Volusia Traffic Engineering	County	58	70	Williamson Blvd	Hand Ave	SB	29.249874	-81.109206	Ormond Bch	Florida	SE	Suburban	1	2		Th basic	LH merge	No RT		Yes	Yes	Yes	Maybe	No	No	Maybe	No
Jon Cheney	County of Volusia Traffic Engineering		59	71	Willamson Blvd	SR 400/Bellvue Ave	SB	29.158983	-81.067095	Daytona Beach	Florida	SE	Suburban	1	2		Th basic	RH merge	RT pocket		Yes	Yes	Yes	Maybe	No	No	Maybe	No
Jack D. Andrews	ITD	DOT	61	72	US95	Appleway Ave	SB	47.700781	-116.791791	Coeur d'Alene	Idaho	NW	Urban	2	3		Th basic	RH drop	Shared ThRT	Interchange	Maybe	Yes	Yes	Maybe	Maybe	No	Maybe	No
Brett Blackadar	Seminole County Engineering	County	62	73	S Orange Blvd (SR431)	SR46	WB	28.81177647	-81.3630724	Seminole County	Florida	SE	Suburban	1	2		Th basic	RH drop	RT pocket		Yes	Yes	Yes	No	No	No	No	No
Brett Blackadar	Seminole County Engineering		63	74	Markham Woods Rd	SR436	NB	28.68866767	-81.39195442	Seminole County	Florida	SE	Suburban	1	1		Th add	RH drop	Shared ThRT	Merge marking before trap lane turn, Th add (770') from a turn lane	Yes	Yes	Yes	No	No	No	No	No
Brett Blackadar	Seminole County Engineering		64	75	Longwood Lake Mary Rd	Lake Mary Blvd	WB	28.75607191	-81.33341789	Seminole County	Florida	SE	Suburban	1	0		Left turns	RH merge	N/A		Yes	Yes	Yes	No	No	No	No	No
Jason Sommerer	Missouri Department of Transportation	DOT	67	76	Route C/ Southwest blvd	Southridge Dr	EB	38.555049	-92.199823	Jefferson City	Missouri	MW	Suburban	1	2		Th basic	RH merge	RT channelized		Yes	Yes	Yes	No	No	Yes		No
Leo Colgna Jr	MoDOT	DOT	68	77	Jefferson Ave	Elm St	NB	37.67639903	-92.65834808	Lebanon	Missouri	MW	Urban	1	2		Th basic	RH merge	Shared ThRT	Based on MODOT drawings	Yes	Maybe	Yes	No	No	No	No	No
Brian Doubrava	MoDOT	DOT	69	78	Kansas Expwy (SR13)	James River Fwy (US60/160), NB on-ramp	SB	37.14150348	-93.31924438	Springfield	Missouri	MW	Suburban	1	0		Left turns	RH merge	N/A	Interchange ramp	Yes	Maybe	Yes	No	No	No	No	No
Eric Turner	MoDOT	DOT	70	79	S Nineteen St	W South St (US65)	WB	37.00450618	-93.22513103	Ozark	Missouri	MW	Rural	1	1		Th add	RH drop	No RT		Yes	Maybe	Yes	No	No	No	No	No
Eric Turner	MoDOT		71	80	Massey Blvd (US160/SR13)	Mt Vernon St (SR14)	SB	37.04349736	-93.30272198	Christian County	Missouri	MW	Suburban	1	2		Th basic	RH merge	RT pocket/channelized		Yes	Maybe	Yes	No	No	No	No	No
Brian Doubrava	MoDOT		72	81	US44/ SB ramp	West Byp	NB (right&left merge	37.24580564	-93.34980011	Springfield	Missouri	MW	Rural	1	2		Th basic	RH merge	Shared ThRT	Interchange ramp	Yes	Maybe	Yes	No	No	No	No	No
Brian Doubrava	MoDOT		73	82	S St, FF Hwy (US160)	James River Fwy (US60/160), NB on-ramp	SB	37.15067116	-93.36344719	Springfield	Missouri	MW	Rural	1	0		Left turns	RH merge	N/A	Interchange ramp	Yes	Maybe	Yes	No	No	No	No	No
Mike Bock	MoDOT	DOT	74	83	Republic Rd (US65)	James River Fwy (US60/160), NB on-ramp	SB	37.13815084	-93.25294018	Springfield	Missouri	MW	Suburban	1	0		Left turns	RH merge	N/A	Interchange ramp	Yes	Maybe	Yes	No	No	No	No	No
Jason Sommerer	Missouri Department of Transportation	DOT	75	84	Wildwood Dr	Missouri Blvd	EB	38.5816036	-92.24584579	Jefferson City	Missouri	MW	Suburban	1	1		Th add	RH drop	Shared ThRT		Yes	Yes	Yes	No	No	Yes		No
Jason Sommerer	Missouri Department of Transportation			85	Wildwood Dr	Missouri Blvd	WB	38.5816036	-92.24584579	Jefferson City	Missouri	MW	Suburban	1	1	45	Th add	RH merge	No RT									
Jason Sommerer	Missouri Department of Transportation			86	Wildwood Dr	Missouri Blvd	WBLT	38.5816036	-92.24584579	Jefferson City	Missouri	MW	Suburban	1	0		Left turns	RH drop	N/A									
Jason Sommerer	Missouri Department of Transportation		76	87	US 63	Grindstone Pkwy	EB	38.91284567	-92.2928381	Columbia	Missouri	MW	Suburban	1	2		Th basic	LH drop	No RT		Yes	Yes	Yes	No	Yes	Yes		No
Bruce Christensen	Idaho Transportation Dept District 4	DOT	77	88	Blue Lakes Blvd (US93)	Fillmore Ave	SB	42.59662868	-114.4551802	Twin Falls	Idaho	NW	Rural	1	0		Left turns	RH merge	N/A		Yes	Yes	Yes	Maybe	Maybe	Yes	Maybe	No

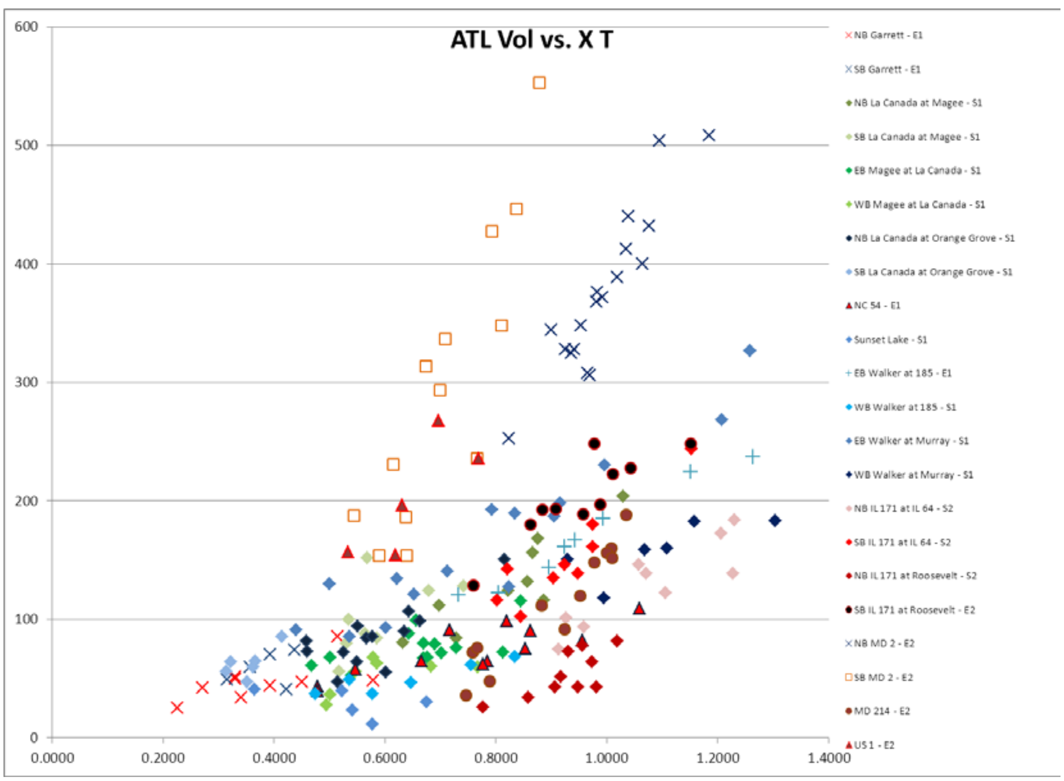
Contact 1					Location of Intersection									Type of ATL							Data available							
Name	Organization	Agency Type	Intersection ID	Approach ID	N/S Roadway	E/W Roadway	Approach	Latitude	Longitude	City	State	Region of Country	Context	Nb of Basic Downstream Through Lanes	Nb of Basic Upstream Through Lanes	Posted Speed	Upstream transition	Downstream transition	Upstream right turn type	Comment	Trafic counts	Crash Data	Signal Timing Data	Speed Information	Previous Study	Anecdotal Evidence	Other Info	ATLs Guidelines in agency?
Tony Sheppard	SCDOT	DOT	79	89	Corley Mill Road (S-32-68) / Ginny Lane (S-32-1834)	US 378	WB	34.01385	-81.152628	Lexington County	South Carolina	SE	Suburban	2	2		Th add	RH merge	RT channelized	Interchange	Yes	Maybe	Yes	No	Yes	No	No	No
Steven C. Strength, PE, PTOE	Louisiana Dept. of Transp. and Development, District 02	DOT	80	90	Williams Blvd (SR49)	Veterans Blvd	SB	30.00655389	-90.24046272	New Orleans	Louisiana	SE	Urban	2	3		Th basic	RH drop	RT pocket		Yes	Yes	Yes	Maybe		Maybe		No
Joel Leisch		University	81	91	NE Third St (US97)	NE Franklin Ave	EB	44.05596158	-121.302495	Bend	Oregon	NW	Urban	1	1	-	Th add	RH merge	Shared ThRT	Type A over 3 intersections: 850', ATL total length: 1400'	Maybe	Maybe	Maybe	Maybe	Maybe	Maybe	Maybe	No
Joel Leisch				92	NE Third St (US97)	NE Franklin Ave	WB	44.05596158	-121.302495	Bend	Oregon	NW	Urban	1	1		Th add	RH drop	Shared ThRT	Type A over 3 intersections: 850', ATL total length: 1400'								
Joseph Auth			83	93	SW McAdams Ave (SR43)	SW Miles St	SB	45.4712594	-122.6716179	Portland	Oregon	NW	Urban	2	2	35	Th add	RH merge	No RT		Yes	Yes	Yes	Yes	No	No	No	No
Dave MacDonald	MoDOT	DOT	84	94	SW Ward Rd (MD150)	Outer Belt Rd	SB	38.85327765	-94.39873695	Jackson County	Missouri	MW	Rural	1	2		Th basic	RH merge	RT pocket		Maybe	Yes	Yes	Maybe	No	Yes	Maybe	No
Dave MacDonald	MoDOT			95	SW Ward Rd (MD150)	Outer Belt Rd	SBLT	38.85327765	-94.39873695	Jackson County	Missouri	MW	Rural	1	0		Left turns	RH drop	N/A		Maybe	Yes	Yes	Maybe	No	Yes	Maybe	No
Cedrick Owens	MoDOT	DOT	85	96	Bass Pro Dr	US40	WB	39.03361949	-94.36479092	Jackson County	Missouri	MW	Rural	2	2		Th add	RH merge	Shared ThRT + RT pocket	Interchange	Maybe	Yes	Yes	No	Yes	Maybe	Maybe	No
Eric Rasband	Utah Department of Transportation	DOT	86	97	Bangerter Hwy (SR154)	W 134000 S	WB	40.50756723	-111.9826555	Salt Lake County	Utah	NW	Rural	1	0		Left turns	RH merge	N/A	NBLT	Maybe	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Eric Rasband	Utah Department of Transportation	DOT	87	98	Bangerter Hwy (SR154)	Redwood Road (SR68)	NB	40.500653	-111.938688	Salt Lake County	Utah	NW	Rural	1	0		Left turns	RH drop	N/A	EBLT	Maybe	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Eric Rasband	Utah Department of Transportation	DOT	88	99	Bangerter Hwy (SR154)	Redwood Road (SR68)	SB	40.500653	-111.938688	Salt Lake County	Utah	NW	Rural	1	0		Left turns	RH merge	N/A	WBLT	Maybe	Yes	Yes	Yes	Yes	Maybe	Maybe	No
Frank Pearson, PE, Regional Traffic Engineer	New York State DOT, Region 10	DOT	89	100	SR347/Nesconset Port Jefferson hwy	SR111/ Hauppauge	NB	40.83045311	-73.19809556	Suffolk County	New York	NE	Suburban	1	1	55	Th add	RH merge	No RT		Maybe	Maybe	Maybe	Maybe				Yes
Frank Pearson, PE, Regional Traffic Engineer	New York State DOT, Region 10			101	SR347/Nesconset Port Jefferson hwy	SR111/ Hauppauge	SB	40.83045311	-73.19809556	Suffolk County	New York	NE	Suburban	1	1	55	Th add	RH merge	Shared ThRT		Maybe	Maybe	Maybe	Maybe				Yes
Frank Pearson, PE, Regional Traffic Engineer	New York State DOT, Region 10			102	SR347/Nesconset Port Jefferson hwy	SR111/ Hauppauge	EB	40.83045311	-73.19809556	Suffolk County	New York	NE	Suburban	2	2		Th add	LH merge	No RT		Maybe	Maybe	Maybe	Maybe				Yes
Mark Kennedy, PE, Regional Traffic Engineer	New York State DOT, Region 1		90	103	Washington Aave Ext	Springsteen Rd	NB	42.69608631	-73.8497597	Albany County	New York	NE	Urban	2	3		Th basic	RH merge	Shared ThRT + RT pocket		Maybe	Maybe	Maybe	Maybe				Yes
Mark Kennedy, PE, Regional Traffic Engineer	New York State DOT, Region 1		91	104	SR156/ Fuller Rd	Washington Aave Ext	SB	42.69239589	-73.83101106	Albany County	New York	NE	Urban	1	2		Th basic	RH merge	RT pocket/channelized		Maybe	Maybe	Maybe	Maybe	Maybe			Yes
			92	105	US19/ SR7/ Beechurst Ave	University Ave	NB	39.63183696	-79.95660782	Monongolia County	West Virginia	NE	Urban	1	0		Left turns	RH drop	N/A		Maybe	Maybe	Maybe	No	No	No	Maybe	No
Shane Dostal	City of Lincoln	DOT	93	106	SR55W/ S 14th St	SR2/ Nebraska Hwy	WB	40.77215688	-96.70161724	Lancaster County	Nebraska	MW	Suburban	2	3		Th basic	RH merge	RT channelized		Maybe	Maybe	Maybe	Maybe	Maybe		Maybe	No
Shane Dostal	City of Lincoln		94	107	48th Street	US34/ O St	WB	40.81325707	-96.6536808	Lincoln	Nebraska	MW	Urban	2	3		Th basic	RH drop	RT pocket		Maybe	Maybe	Maybe	Maybe	Maybe			No
Shane Dostal	City of Lincoln			108	48th Street	US34/ O St	SB	40.81325707	-96.6536808	Lincoln	Nebraska	MW	Urban	1	2		Th basic	RH drop	RT pocket		Maybe	Maybe	Maybe	Maybe	Maybe			No
Gary Dossey	Caltrans, District 3, Highway Safety	DOT	95	109	Excelsior Rd	Jackson Rd (SR16)	WB	38.518886	-121.297866	Sacramento County	California	SW	Rural	1	1	55	Th add	RH merge	Shared ThRT		Yes	Maybe	Yes	Maybe	Maybe	Yes	Yes	
Gary Dossey	Caltrans, District 3, Highway Safety			110	Excelsior Rd	Jackson Rd (SR16)	EB	38.518886	-121.297866	Sacramento County	California	SW	Rural	1	1	55	Th add	RH merge	Shared ThRT		Yes	Maybe	Yes	Maybe	Maybe	Yes	Yes	
Gary Dossey	Caltrans, District 3, Highway Safety		96	111	SR89	West River Street	NB	39.314928	-120.203801	Sacramento County	California	SW	Rural	1	1		Th add	RH merge	RT pocket		Yes	Maybe	Yes	Maybe	Maybe	Yes	Yes	
Tom Urbanik	The University of Tennessee	University	97	112	Ebenezer Rd	S Northshore Dr (SR332)	EB	35.86521	-84.060819	Knox County	Tenessee	MW	Rural	1	1	40	Th add	RH merge	No RT									No
	FDOT D6	DOT	98	113	US 1	Atlantic Blvd	EB	26.231714	-80.102836	Pompano Beach	Florida	SE	Urban	2	2		Th add	RH drop	RT pocket									
	FDOT D6			114	US 1	Atlantic Blvd	WB	26.231714	-80.102836	Pompano Beach	Florida	SE	Urban	2	2		Th add	RH drop	Shared ThRT									
	City of Raleigh	City	99	115	Oberlin Rd	Clark Ave	WB	35.789153	-78.662983	Raleigh	North Carolina	SE	Urban	1	1		Th add	LH drop	Shared ThRT									
	City of Raleigh		100	116	Oberlin Rd	Wade Ave	NB	35.7999	-78.660439	Raleigh	North Carolina	SE	Urban	1	1		Th add	RH drop	Shared ThRT	Interchange								
	City of Raleigh			117	Oberlin Rd	Wade Ave	SB	35.7999	-78.660439	Raleigh	North Carolina	SE	Urban	1	1		Th add	RH merge	RT pocket/channelized	Interchange								
			101	118	IN 37	Hospital Rd	NB	39.425403	-86.401731	Martinsville	Indiana	MW	Suburban	2	2	50	Th add	RH merge	Shared ThRT									
Thomas Tushner	Washington County, OR	County	102	119	SW Terwilliger Blvd	SW Taylors Ferry Rd	SB	45.463114	-122.684867	Portland	Oregon	NW	Urban	1	1	35	Th add	RH merge	Shared ThRT	No merge arrow striping								
Joseph Auth	ODOT	DOT	103	120	SW Tualatin Valley Hwy (OR 8)	SW Murray Blvd	EB	45.489572	-122.82611	Beaverton	Oregon	NW	Urban	2	2	45	Th add	RH merge	Shared ThRT	No merge arrow striping								
Joseph Auth	ODOT			121	SW Tualatin Valley Hwy (OR 8)	SW Murray Blvd	WB	45.489572	-122.82611	Beaverton	Oregon	NW	Urban	2	2	45	Th add	RH merge	Shared ThRT	No merge arrow striping								
Thomas Tushner	Washington County, OR	County	104	122	SW MacAdam Ave	SW Taylors Ferry Rd	SB	45.471283	-122.671618	Portland	Oregon	NW	Urban	2	2		Th add	RH merge	RT channelized	No merge arrow striping								
Thomas Tushner	Washington County, OR		105	123	SW 158th Ave	SW Walker Rd	WB	45.516761	-122.839551	Beaverton	Oregon	NW	Urban	2	2		Th add	RH drop	Shared ThRT	Auxiliary lane is 3,000'								
Thomas Tushner	Washington County, OR		106	124	NW Evergreen Pkwy	NW Cornell Rd	SB	45.536203	-122.867511	Beaverton	Oregon	NW	Urban	2	3		Th basic	RH drop	Shared ThRT	Bike lane stripes on drop								
Thomas Tushner	Washington County, OR		107	125	NW 185th Ave	NW Walker Rd	EB	45.527719	-122.867472	Beaverton	Oregon	NW	Urban	1	1		Th add	RH merge	RT pocket									
Thomas Tushner	Washington County, OR			126	NW 185th Ave	NW Walker Rd	WB	45.527719	-122.867472	Beaverton	Oregon	NW	Urban	1	1	45	Th add	RH merge	Shared ThRT									
Thomas Tushner	Washington County, OR		108	127	SW Meadow Dr	SW Walker Rd	WB	45.513618	-122.828377	Beaverton	Oregon	NW	Urban	1	1	45	Th add	RH merge	Shared ThRT									
Thomas Tushner	Washington County, OR		109	128	SW Murray Blvd	SW Walker Rd	EB	45.510558	-122.822142	Beaverton	Oregon	NW	Urban	1	1	35	Th add	RH merge	Shared ThRT									
Thomas Tushner	Washington County, OR			129	SW Murray Blvd	SW Walker Rd	WB	45.510558	-122.822142	Beaverton	Oregon	NW	Urban	1	1	35	Th add	RH merge	Shared ThRT									

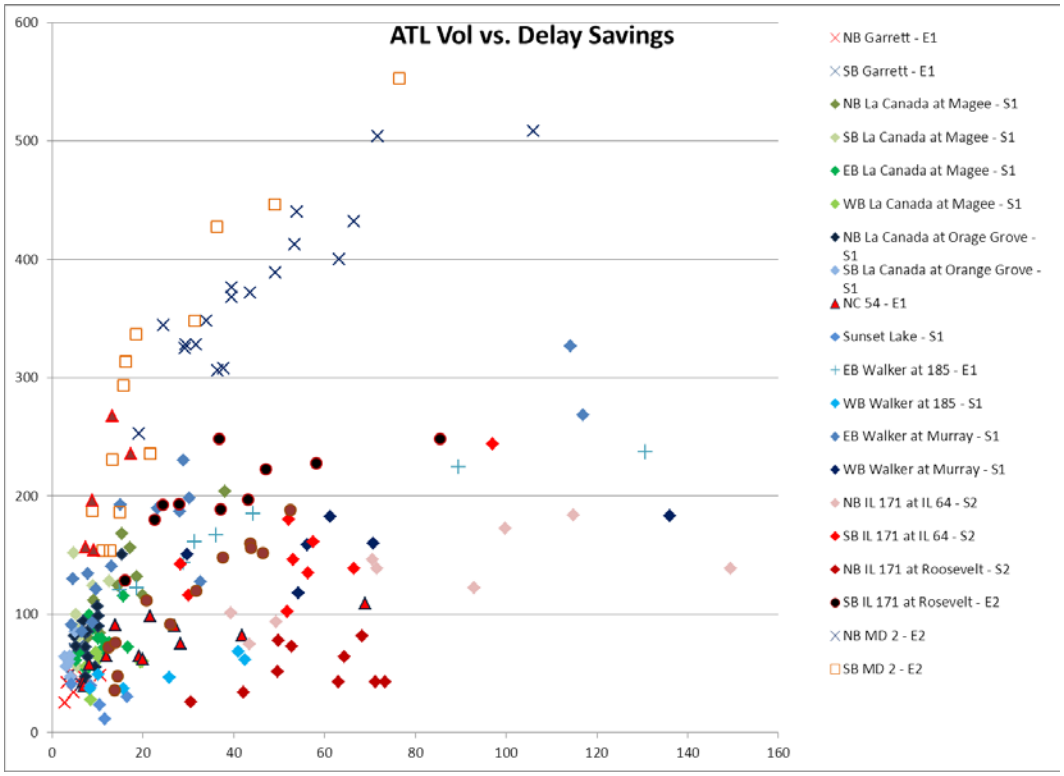
Contact 1			Location of Intersection											Type of ATL							Data available							
Name	**Organization**	**Agency Type**	**Intersection ID**	**Approach ID**	**N/S Roadway**	**E/W Roadway**	**Approach**	**Latitude**	**Longitude**	**City**	**State**	**Region of Country**	**Context**	**Nb of Basic Downstream Through Lanes**	**Nb of Basic Upstream Through Lanes**	**Posted Speed**	**Upstream transition**	**Downstream transition**	**Upstream right turn type**	**Comment**	**Trafic counts**	**Crash Data**	**Signal Timing Data**	**Speed Information**	**Previous Study**	**Anecdotal Evidence**	**Other Info**	**ATLs Guidelines in agency?**
Joseph Auth	ODOT	DOT	110	130	OR 43	Marylbrook Dr	NB	45.397419	-122.6526	Lake Oswego	Oregon	NW	Suburban	1	1		Th add	RH merge	RT pocket	Auxiliary lane is 1,800'								
Joseph Auth	ODOT			131	OR 43	Marylbrook Dr	SB	45.397419	-122.6526	Lake Oswego	Oregon	NW	Suburban	1	1		Th add	RH merge	RT pocket	Auxiliary lane is 1,800'								
Joseph Auth	ODOT		111	132	Pac Highway (OR 99W)	SW Edy Rd	NB	45.366467	-122.848347	Tualatin North	Oregon	NW	Urban	2	2		Th add	RH merge	RT pocket	Auxiliary lane is 3,000'								
Joseph Auth	ODOT			133	Pac Highway (OR 99W)	SW Edy Rd	SB	45.366467	-122.848347	Tualatin North	Oregon	NW	Urban	2	2	45	Th add	RH merge	Shared ThRT	Auxiliary lane is 3,000'								
James H. Dunlop	Transportation Mobility and Safety Division North Carolina Department of Transportation	DOT	112	134	CR571/ Ridgeway Rd	NJ 70	EB	40.023462	-74.269557	Ocean County	New Jersey	NE	Suburban	1	1	-	Th add	RH merge	Shared ThRT	Right-side space for driveway access								
James H. Dunlop	North Carolina Department of Transportation			135	CR571/ Ridgeway Rd	NJ 70	WB	40.023462	-74.269557	Ocean County	New Jersey	NE	Suburban	1	1	-	Th add	RH merge	Shared ThRT	Right-side space for driveway access								
James H. Dunlop	North Carolina Department of Transportation		113	136	US9/ Lakewood Rd	CR571/ Indian Head Rd	EB	39.992803	-74.211368	Ocean County	New Jersey	NE	Suburban	1	2		Th basic	RH merge	Shared ThRT	This intersection is very congested during peak hours.								
James H. Dunlop	North Carolina Department of Transportation		114	137	Whitesville Road	NJ70	WB	40.037395	-74.242668	Ocean County	New Jersey	NE	Suburban	1	1	45	Th add	RH merge	Shared ThRT	Right-side space for driveway access								
James H. Dunlop	North Carolina Department of Transportation		115	138	Old Marlton Pike	NJ70/ Marlton pike	EB	39.898988	-74.852525	Burlington County	New Jersey	NE	Rural	1	1	45	Th add	RH merge	Shared ThRT	Right-side space for driveway access								
James H. Dunlop	North Carolina Department of Transportation			139	Old Marlton Pike	NJ70/ Marlton pike	EB	39.898988	-74.852525	Burlington County	New Jersey	NE	Rural	1	1	45	Th add	RH merge	Shared ThRT	Right-side space for driveway access								

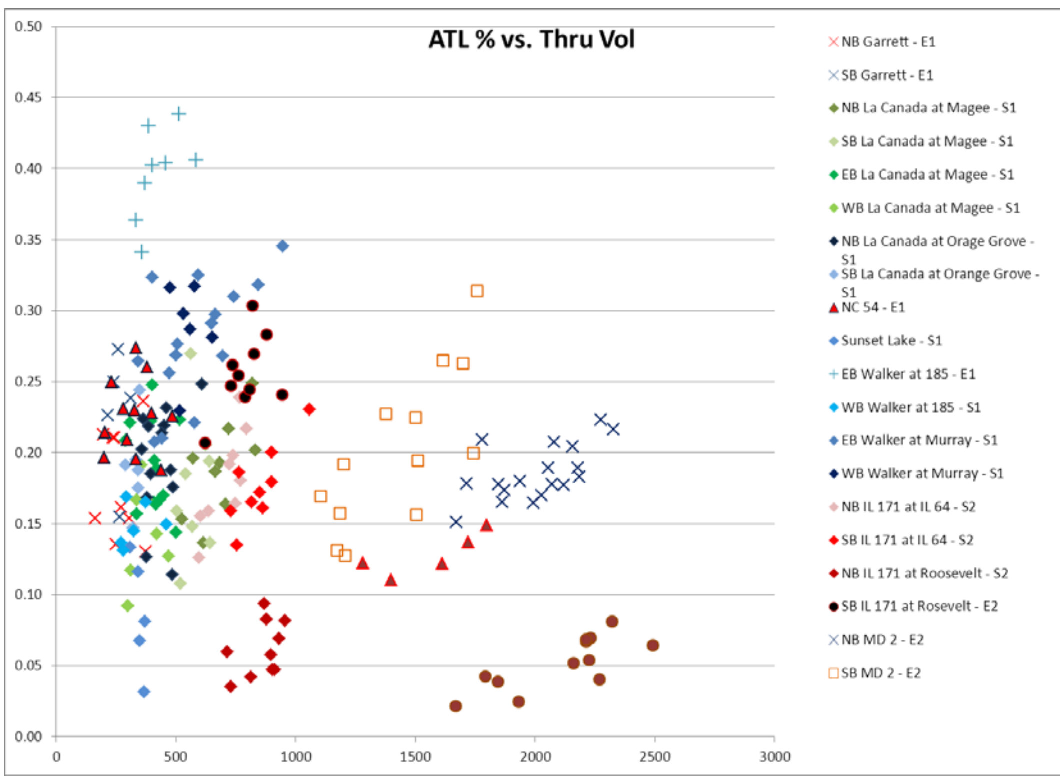
APPENDIX B: FIELD DATA COLLECTION RESULTS

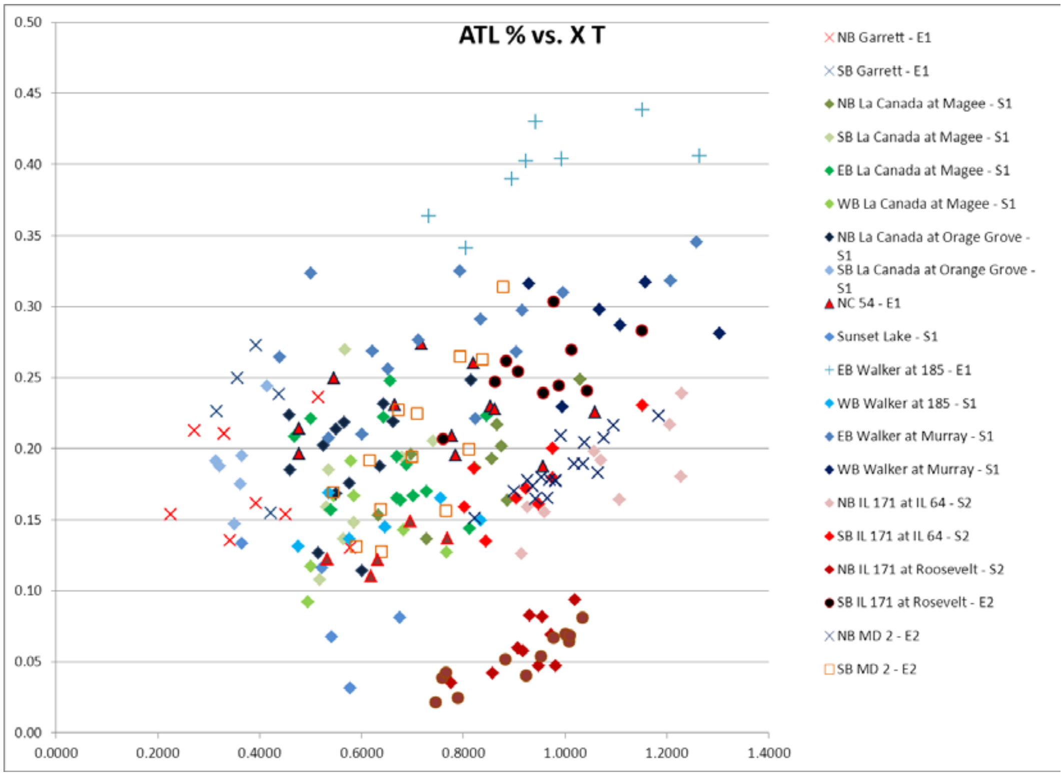
This appendix includes the raw data plots from the 15-minute observations at each of the study ATL approaches.

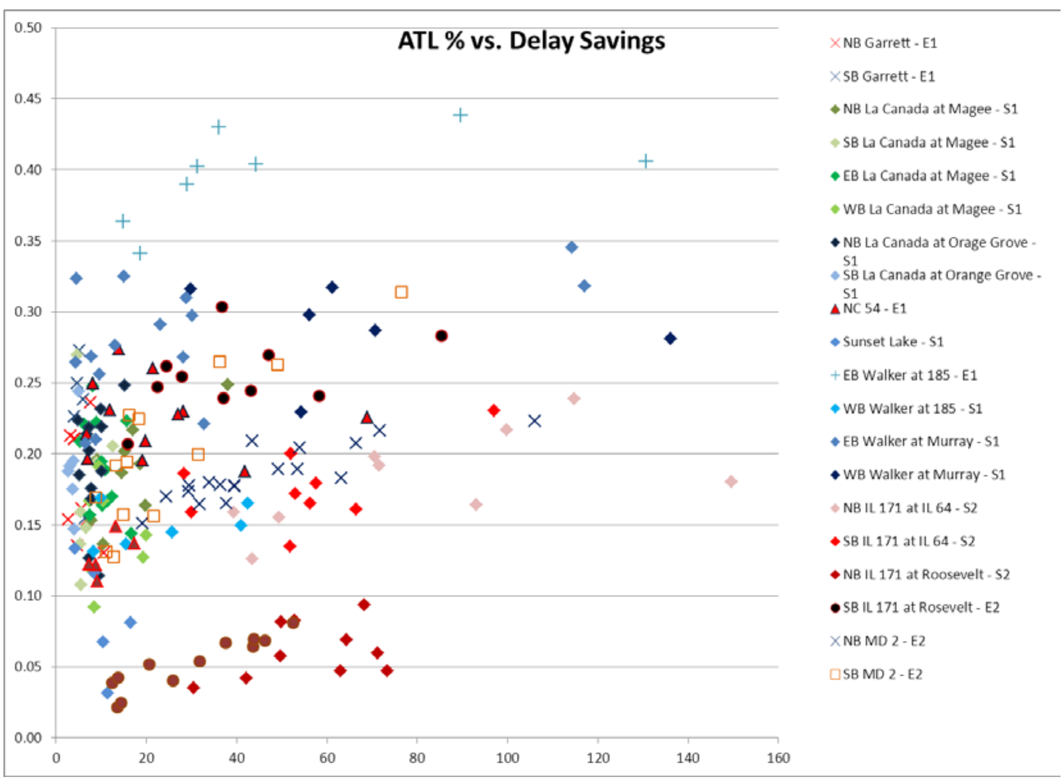


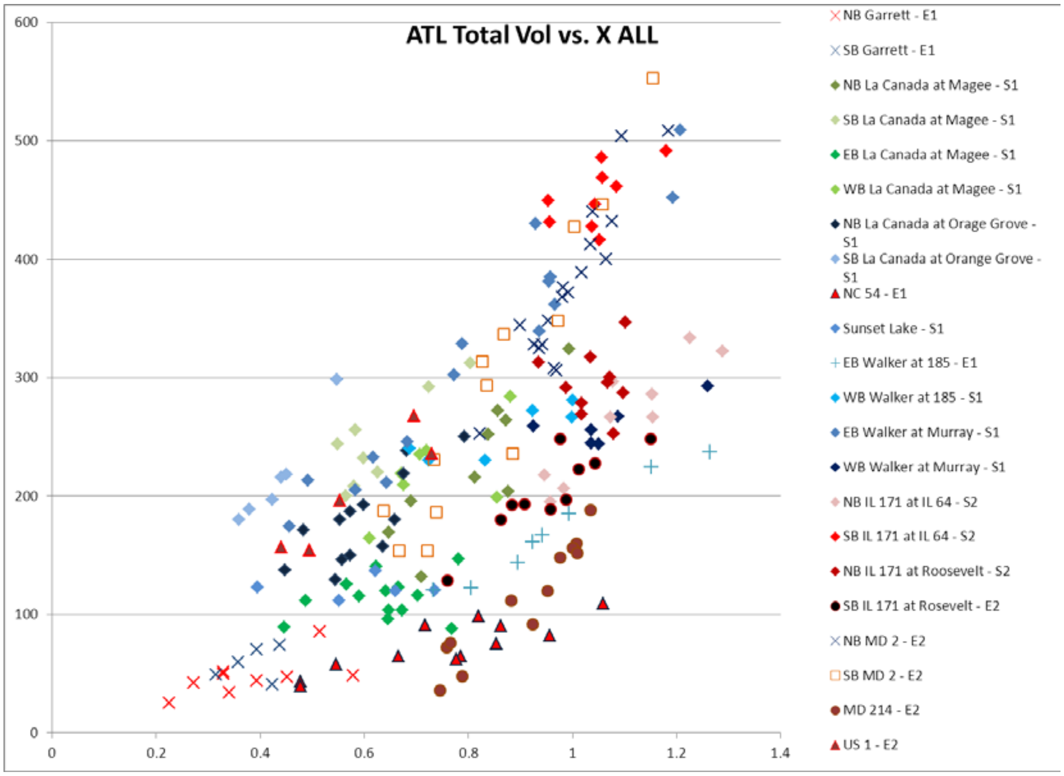


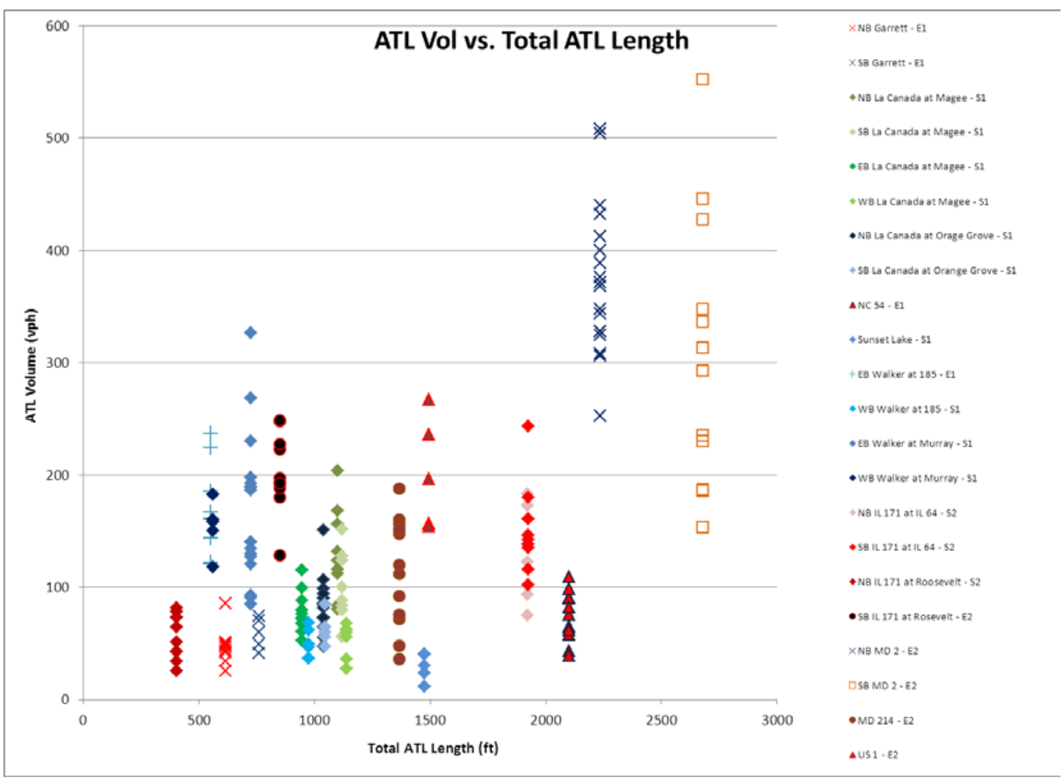

ATL Vol vs. Delay Savings
600
500
400
300
200
100
0
0
20
40
60
80
100
120
140
160
NB Garrett - E1
SB Garrett - E1
NB La Canada at Magee - S1
SB La Canada at Magee - S1
EB La Canada at Magee - S1
WB La Canada at Magee - S1
NB La Canada at Orage Grove - S1
SB La Canada at Orange Grove - S1
NC 54 - E1
Sunset Lake - S1
EB Walker at 185 - E1
WB Walker at 185 - S1
EB Walker at Murray - S1
WB Walker at Murray - S1
NB IL 171 at IL 64 - S2
SB IL 171 at IL 64 - S2
NB IL 171 at Roosevelt - S2
SB IL 171 at Rosevelt - E2
NB MD 2 - E2
SB MD 2 - E2

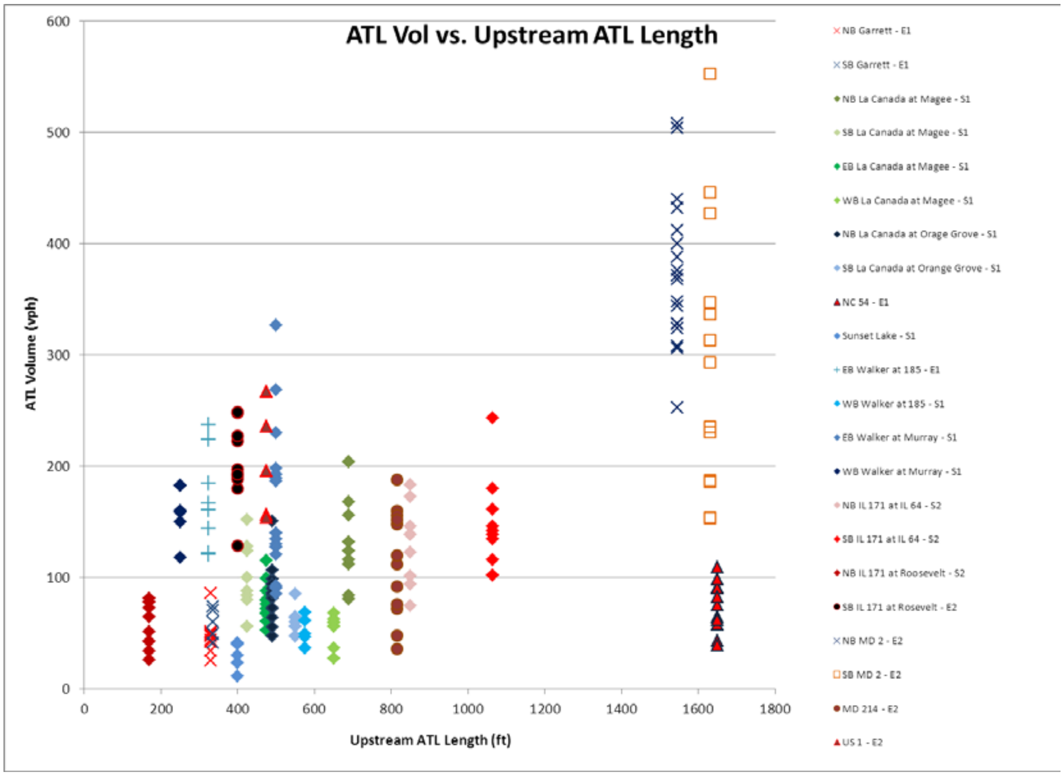

ATL % vs. Thru Vol
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
500
1000
1500
2000
2500
3000
NB Garrett - E1
SB Garrett - E1
NB La Canada at Magee - S1
SB La Canada at Magee - S1
EB La Canada at Magee - S1
WB La Canada at Magee - S1
NB La Canada at Orage Grove - S1
SB La Canada at Orange Grove - S1
NC 54 - E1
Sunset Lake - S1
EB Walker at 185 - E1
WB Walker at 185 - S1
EB Walker at Murray - S1
WB Walker at Murray - S1
NB IL 171 at IL 64 - S2
SB IL 171 at IL 64 - S2
NB IL 171 at Roosevelt - S2
SB IL 171 at Rosevelt - E2
NB MD 2 - E2
SB MD 2 - E2

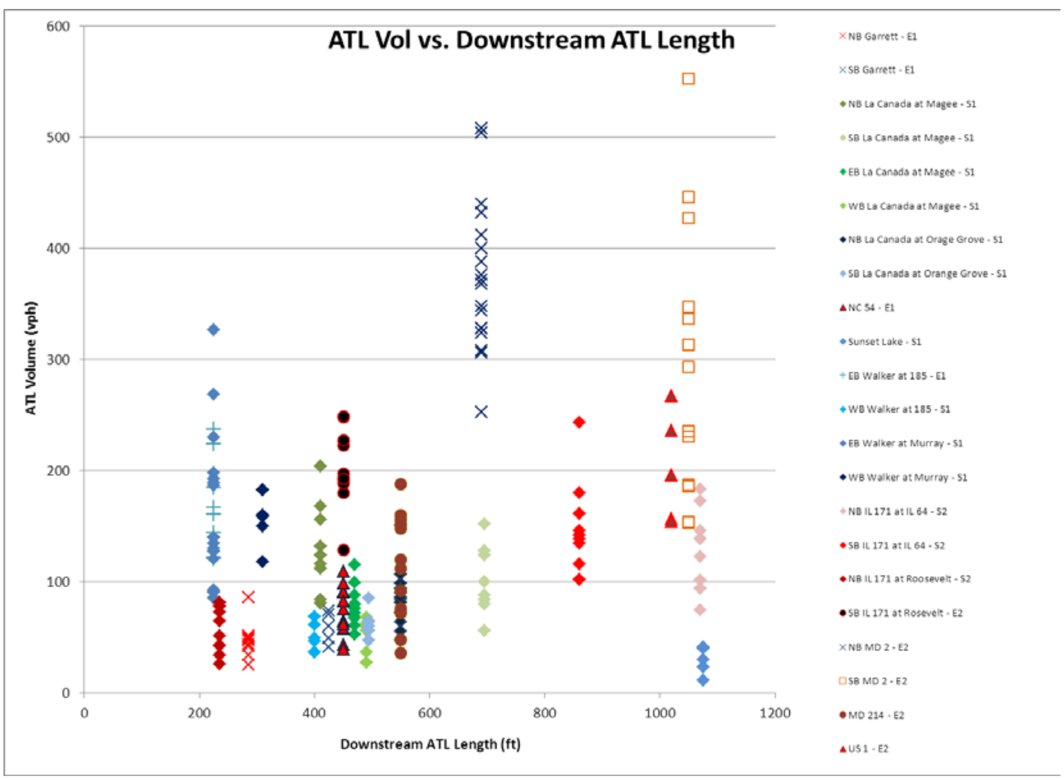

ATL % vs. X T
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0.0000
0.2000
0.4000
0.6000
0.8000
1.0000
1.2000
1.4000
NB Garrett - E1
SB Garrett - E1
NB La Canada at Magee - S1
SB La Canada at Magee - S1
EB La Canada at Magee - S1
WB La Canada at Magee - S1
NB La Canada at Orage Grove - S1
SB La Canada at Orange Grove - S1
NC 54 - E1
Sunset Lake - S1
EB Walker at 185 - E1
WB Walker at 185 - S1
EB Walker at Murray - S1
WB Walker at Murray - S1
NB IL 171 at IL 64 - S2
SB IL 171 at IL 64 - S2
NB IL 171 at Roosevelt - S2
SB IL 171 at Rosevelt - E2
NB MD 2 - E2
SB MD 2 - E2


ATL % vs. Delay Savings
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
0
20
40
60
80
100
120
140
160
NB Garrett - E1
SB Garrett - E1
NB La Canada at Magee - S1
SB La Canada at Magee - S1
EB La Canada at Magee - S1
WB La Canada at Magee - S1
NB La Canada at Orage Grove - S1
SB La Canada at Orange Grove - S1
NC 54 - E1
Sunset Lake - S1
EB Walker at 185 - E1
WB Walker at 185 - S1
EB Walker at Murray - S1
WB Walker at Murray - S1
NB IL 171 at IL 64 - S2
SB IL 171 at IL 64 - S2
NB IL 171 at Roosevelt - S2
SB IL 171 at Rosevelt - E2
NB MD 2 - E2
SB MD 2 - E2


ATL Total Vol vs. X ALL
0
100
200
300
400
500
600
0
0.2
0.4
0.6
0.8
1
1.2
1.4
NB Garrett - E1
SB Garrett - E1
NB La Canada at Magee - S1
SB La Canada at Magee - S1
EB La Canada at Magee - S1
WB La Canada at Magee - S1
NB La Canada at Orage Grove - S1
SB La Canada at Orange Grove - S1
NC 54 - E1
Sunset Lake - S1
EB Walker at 185 - E1
WB Walker at 185 - S1
EB Walker at Murray - S1
WB Walker at Murray - S1
NB IL 171 at IL 64 - S2
SB IL 171 at IL 64 - S2
NB IL 171 at Roosevelt - S2
SB IL 171 at Rosevelt - E2
NB MD 2 - E2
SB MD 2 - E2
MD 214 - E2
US 1 - E2


ATL Vol vs. Total ATL Length
ATL Volume (vph)
Total ATL Length (ft)
0
100
200
300
400
500
600
0
500
1000
1500
2000
2500
3000
NB Garrett - E1
SB Garrett - E1
NB La Canada at Magee - S1
SB La Canada at Magee - S1
EB La Canada at Magee - S1
WB La Canada at Magee - S1
NB La Canada at Orage Grove - S1
SB La Canada at Orange Grove - S1
NC 54 - E1
Sunset Lake - S1
EB Walker at 185 - E1
WB Walker at 185 - S1
EB Walker at Murray - S1
WB Walker at Murray - S1
NB IL 171 at IL 64 - S2
SB IL 171 at IL 64 - S2
NB IL 171 at Roosevelt - S2
SB IL 171 at Rosevelt - E2
NB MD 2 - E2
SB MD 2 - E2
MD 214 - E2
US 1 - E2


ATL Vol vs. Upstream ATL Length
ATL Volume (vph)
Upstream ATL Length (ft)
0
100
200
300
400
500
600
0
200
400
600
800
1000
1200
1400
1600
1800
NB Garrett - E1
SB Garrett - E1
NB La Canada at Magee - S1
SB La Canada at Magee - S1
EB La Canada at Magee - S1
WB La Canada at Magee - S1
NB La Canada at Orage Grove - S1
SB La Canada at Orange Grove - S1
NC 54 - E1
Sunset Lake - S1
EB Walker at 185 - E1
WB Walker at 185 - S1
EB Walker at Murray - S1
WB Walker at Murray - S1
NB IL 171 at IL 64 - S2
SB IL 171 at IL 64 - S2
NB IL 171 at Roosevelt - S2
SB IL 171 at Rosevelt - E2
NB MD 2 - E2
SB MD 2 - E2
MD 214 - E2
US 1 - E2


ATL Vol vs. Downstream ATL Length
ATL Volume (vph)
Downstream ATL Length (ft)
0
100
200
300
400
500
600
0
200
400
600
800
1000
1200
NB Garrett - E1
SB Garrett - E1
NB La Canada at Magee - S1
SB La Canada at Magee - S1
EB La Canada at Magee - S1
WB La Canada at Magee - S1
NB La Canada at Orage Grove - S1
SB La Canada at Orange Grove - S1
NC 54 - E1
Sunset Lake - S1
EB Walker at 185 - E1
WB Walker at 185 - S1
EB Walker at Murray - S1
WB Walker at Murray - S1
NB IL 171 at IL 64 - S2
SB IL 171 at IL 64 - S2
NB IL 171 at Roosevelt - S2
SB IL 171 at Rosevelt - E2
NB MD 2 - E2
SB MD 2 - E2
MD 214 - E2
US 1 - E2

APPENDIX C: ATL LENGTH ESTIMATION PROCEDURE

This procedure is built around the ATL flow rate estimation models. Since there are separate models for 1-CTL and 2-CTL cases, the same reasoning applies to the ATL length estimation process. The procedure is implemented in two Excel spreadsheets that estimate minimum ATL length and provide other important performance measures as outputs. The starting point of the analysis has no ATL presence. For the 1-CTL case, the procedure considers an approach with a single shared through-right continuous lane, while the 2-CTL case assumes an exclusive through-movement lane and a shared through-right continuous lane. In all cases, left turns are assumed to operate from an exclusive lane or pocket and therefore are not part of the analysis.

An outline of the procedure as it relates to the ATL upstream length determination is explained in the following steps:

1. Identify whether the 1-CTL or 2-CTL case applies.
2. Supply the data required for ATL flow-rate estimation including:
    a. Total approach through and right turn flow rates,
    b. Cycle length and effective green time for the subject approach, and
    c. Saturation flow rate for both through and right-turn movements.
3. Estimate the ATL flow rate based on the 1-CTL or 2-CTL model in Section 3.4.
4. Calculate the ATL through flow rate assuming equal lane v/s based on the HCM 2010 shared or exclusive lane group volume distribution.
5. Take the predicted ATL flow rate as the lower estimate from steps (3) and (4) above.
6. Calculate the ATL and CTL lane volumes, capacity, control delay, and back of queue using the HCM 2010 signalized intersection procedures. For shared ATLs, include the right-turn flow rate in the lane flow computations.
7. Estimate the 95th percentile queues in both the ATL and CTL (for one CTL lane in the case of two CTLs) using HCM procedures.
8. Select a storage length based on the greater of the 95th percentile queues in the ATL and CTL. Queue storage or access distance is calculated based on an estimate of average vehicle spacing in a stopped queue.

The determination of the requisite downstream length requires a further set of input parameters, some of which may be defaulted as shown in parentheses, namely:

- Approach free flow speed or speed limit,
- Average acceleration rate from a stop on the ATL (10 ft/sec2),
- Intersection width measured from the stop line to the far curb (40 ft),
- Minimum acceptable headway in CTL traffic stream (6 sec), and
- Driver reaction time (1 sec).

The downstream length estimation based on storage of vehicles at the desired spacing in the downstream length (DSL1) proceeds as follows.

Estimate the average uniform and random and oversaturation back of queue (BOQ) for ATL through traffic only (Q1 + Q2 in HCM terminology). This approach incorporates two opposing and simplifying assumptions. The first is that the required length will be based on the average BOQ as opposed to the 95th percentile value as was done in the upstream case. This is offset by another assumption where all through-movement vehicles in the ATL are assumed to be contiguous in the queue and not separated intermittently by right-turning vehicles in a shared lane, which would result in a larger separation between through-movement vehicles. This procedure assumes that the effects of the two assumptions will balance.

The downstream storage criterion is based on providing sufficient spacing between ATL vehicles at the free flow speed or speed limit. Since vehicles accelerate from the stop line position, the downstream distance measured from the far curb can be shown to be:

$$\boxed{DSL_1 = \frac{V^2}{2a} + (L + TV)(BOQ - 1) - INTW}$$

Where:

V = free flow speed or speed limit (in feet/second),

a = acceleration rate from stop-line (in feet/second2),

L = spacing between vehicles at stop (in feet),

T = driver reaction time (in seconds), and

INTW = intersection width measured from the stop bar to the far curb (in feet).

The second criterion for estimating required downstream length is based on gap availability and acceptance under uninterrupted flow conditions, especially on high-speed approaches. The concept is that, after traveling a reaction distance past the intersection, an ATL driver must find an acceptable merge gap in the neighboring CTL within the confines of the downstream ATL length. Using assumptions on the headway distribution in the CTL and a minimum acceptable merge headway value, the distance measured from the far curb is shown to be:

$$\boxed{DSL_2 = \quad V(T + NUM \times G_r)}$$

Where:

NUM = the number of rejected gaps in the CTL. This could be either the mean value of rejected or a pre-specified percentile number of rejected gaps, as explained below.

Gr = expected or average size of a rejected headway in the CTL (in seconds).

This model used to calculate DSL_2 is based upon a gap acceptance procedure with the following assumptions:

- Drivers begin searching for gaps as soon as they pass the stop bar,
- Drivers have reached the operating speed of the arterial,
- Drivers are homogeneous with regard to a critical headway tc, and
- Traffic in the adjacent CTL follows an exponential headway distribution.

The following steps describe the model development:

Step 1. Determine the number of rejected gaps encountered until an acceptable gap is found. Let p be the probability of rejecting a gap in the CTL and tc the size of the critical gap. Then

$$p = P(h \leq t_c) = 1 - e^{-\lambda t_c}$$

where λ is the flow rate in the CTL (in vehicles per hour)

Then the probability of rejecting exactly i gaps is p^i (1-p) and the expected number of rejected gaps is:

$$N_r = \sum_{i=0}^{\infty} ip^i(1-p) = p(1-p)\sum_{i=0}^{\infty}\frac{d}{dp}p^i = p(1-p)\frac{d}{dp}\left(\frac{1}{1-p}\right) = \frac{p(1-p)}{(1-p)^2} = \frac{p}{1-p}$$

An alternative approach to using N_r is to design the downstream length to accommodate the 95th number of rejected gaps, as opposed to the mean value. In this case, we would like to determine the number of rejected gaps that would only be exceeded at most (1- α) percent of the time. In other words, find "I" such that the number of rejected gaps X is such that

$$P(X > I) \quad < 1-\alpha$$

or conversely $$P(X \leq I) \quad \geq \alpha$$

which can be then expressed as

$$\sum_{i=0}^{I} p^i(1-p) \geq \alpha$$

Solving for I gives the condition for the percentile rejected gap:

$$I(\alpha, p) \geq \frac{\ln(1-\alpha)}{\ln(p)} - 1$$

For example, if the probability of a rejected gap p=0.50 and a 95th percentile confidence level on the number of rejected gaps is desired, then

$$I\,(0.95{,}0.50) \geq \frac{\ln 0.05}{\ln 0.50} - 1 = 3.32$$

This compare with a mean number of rejected gaps of

$$N_r = 0.50 \;/\; (1\text{-}0.50) = 1.0$$

In the remaining steps, the user may chose to apply either the percentile or mean value of rejected gaps.

Step 2. Determine the expected size of a rejected gap, E(t|t<t_c):

$$G_r = \frac{E(t)}{P(t < t_c)}$$

Where

$$E(t) = \int_0^{t_c} t \times f(t)dt = \int_0^{t_c} \lambda t\, e^{\lambda t}\, dt = \left[-te^{-\lambda t}\right]_0^{t_c} - \int_0^{t_c} e^{-\lambda t}\, dt$$

using integration by parts, and after simplifying gives:

$$E(t) = \frac{1}{\lambda}\left(1 - e^{-\lambda t_c}\right) - t_c e^{-\lambda t_c}$$

Since $P(t < t_c) = 1 - e^{-\lambda t_c}$

$$E(t|t < t_c) = \frac{1}{\lambda} - \frac{t_c e^{-\lambda t_c}}{1 - e^{-\lambda t_c}}$$

Step 3. Calculate the expected or percentile waiting time for an acceptable gap. In the first case, that value is equal to the product of the expected number of rejected gaps and the expected size of a rejected gap:

$$\frac{p}{1-p} \times E(t|t < t_c) = \frac{1 - e^{-\lambda t_c}}{e^{-\lambda t_c}} \times \left(\frac{1}{\lambda} - \frac{t_c e^{-\lambda t_c}}{1 - e^{-\lambda t_c}}\right) = \frac{1 - e^{-\lambda t_c}}{\lambda e^{-\lambda t_c}} - t_c$$

Optionally, if one selected the percentile gap approach, then the waiting time for the (alpha) percentile rejected gap would be

$$= I(\alpha, p)\{\frac{1}{\lambda} - \frac{t_c e^{-\lambda t_c}}{1 - e^{-\lambda t_c}}\}$$

Step 4. Calculate the distance traveled before an acceptable gap is found:

$$D = V\left(\frac{1 - e^{-\lambda t_c}}{\lambda e^{-\lambda t_c}} - t_c\right)$$

Or in the case of the percentile gap:

$$D = V\ I(\alpha, p)\{\frac{1}{\lambda} - \frac{t_c e^{-\lambda t_c}}{1 - e^{-\lambda t_c}}\}$$

Where: V is the operating speed in feet per second.

Incorporating the reaction time T, the total distance traveled (in feet) is given by

$$DSL_2 = V\left(T + \frac{1 - e^{-\lambda t_c}}{\lambda e^{-\lambda t_c}} - t_c\right) = V(T + N_r G_r)$$

Or in the case of the percentile gap,

$$DSL_2 = V[T + I(\alpha, p)\ G_r]$$

APPENDIX D: HCM IMPLEMENTATION CASE STUDY

The following example is a reworking of “Example Problem 1: Automobile LOS” in the 2010 Highway Capacity Manual (HCM) (*1*). In this reworking, the rightmost lane of the Eastbound approach is considered to be an auxiliary through lane (ATL) shared with right turns, instead of a continuous lane. An additional case examines the operations of adding an ATL to the northbound approach. This study makes use of a computational engine developed in the 2010 Highway Capacity Manual to analyze actuated control, as well as a computational engine developed as part of NCHRP 3-98 to analyze ATL facilities.

PURPOSE

The purpose of this reworking is to examine the practicality of the models developed as part of NCHRP 03-98. Although the models were calibrated and verified using over 200 15-minute data points selected from over 20 sites across the United States, there has been little research on how the models would eventually be implemented in the HCM. Specifically, the models should be able to:

- Predict reasonable values for ATL volumes (particularly, the predicted v/c for the ATL from the 3-98 models should not exceed the v/c for any CTL on the same approach)
- Allow for easy computation by consisting of lucid, sensible variables
- Result in guidelines which can be used to recommend design elements of the approach to the intersection (e.g., storage and downstream taper length)

This paper presents the use of a practical example already been approved for the HCM 2010.

MODELS

The first case in the example makes use of the following model developed for ATL approaches with one CTL (Model 1):

ATL Through-movement Volume (vph) = $20.226 + 81.791X_T^2 + \frac{1.65\,(Thru\,Vol)^2}{10{,}000}$

where $X_T = \frac{Through\ Volume}{s \times \frac{g}{C}}$ (X_T is the v/c ratio for through traffic <u>assuming no ATL</u>)

The second part of the example uses the following model developed for ATL approaches with two CTLs (Model 2B):

ATL Through-movement Volume (vph) = $29.24 + \frac{17.3 \times Thru\ Vol}{100} - 90.291X_R$

where X_R is similar to the XT term but uses right-turn volume to capacity for a shared lane.

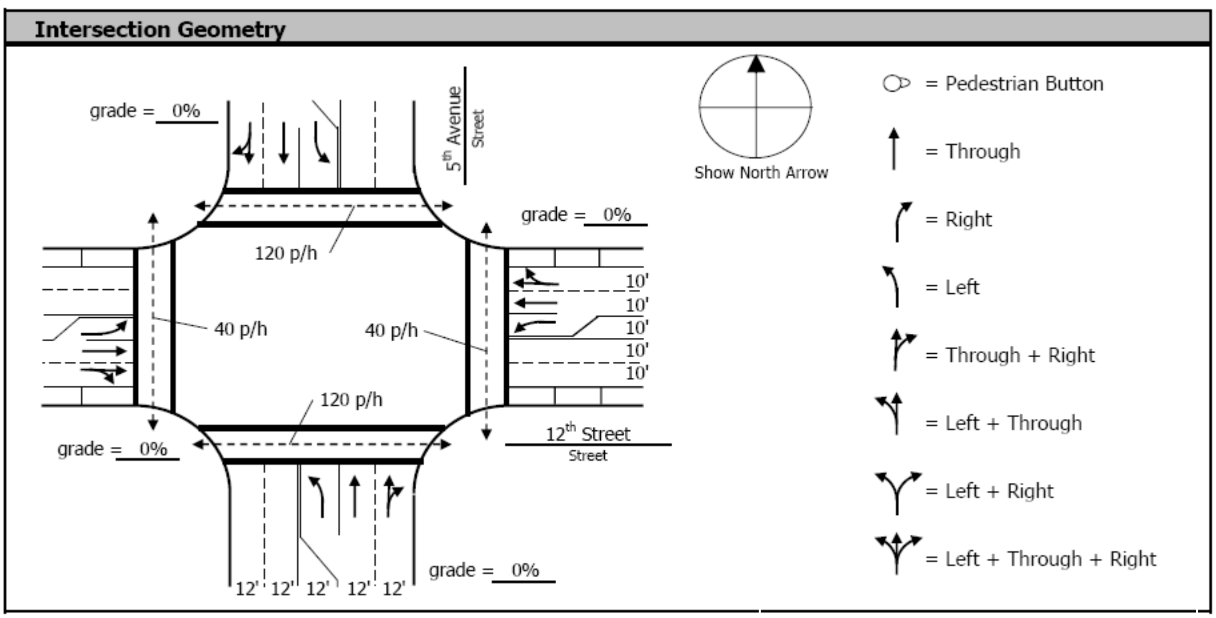
METHODOLOGY

This application uses an iterative approach to account for the volume shifted from the CTL(s) to the ATL and how this affects the actuated signal timing. The following steps are involved:

1. Run the HCM 2010 computational engine *without* the ATL(s) and record g/C & X_T for the ATL approach(es).
2. Apply the ATL computational engine and record the predicted ATL through volume for all ATL approaches, considering the minimum value determined by the equal-degree-of-saturation upper bound.
3. Subtract the ATL through volume from the total through volume and re-run the HCM 2010 computational engine. Record the new values of g/C and X_T for the ATL approach(es).
4. Reapply Step (2) using the new signal timing parameters determined in Step (3).
5. Check to see if the new ATL through volume prediction converges to within 10 vph of the previous ATL through volume prediction. If so, proceed to calculating and reporting performance measures commensurate with HCM 2010 procedures. If not, repeat process starting with Step (2).

EXAMPLE PROBLEM

The study intersection (HCM 2010 Example Problem 1) is shown in Figure D-1.



Source: Highway Capacity Manual 2010

Figure D-1. Input data for HCM 2010 Example Problem 1: Automobile LOS as shown in Exhibit 18-37.

For the purposes of this application the ATL operational procedures were applied to two cases:

Case 1 - Assumes that the east- and westbound approaches contain one CTL and one shared ATL.

Case 2 - Considers an additional shared ATL on the northbound approach.

The signal timing information for the example problem is depicted from the screen capture of the computation engine in Figure D-2.

Controller Data Worksheet

General Information

Analyst:	BR	Intersection:	5th Avenue/12th Street
Agency or Company:		Area Type:	CBD
Date Performed:	2/11/2010	Phase 2:	EB
Analysis Time Period:	5:30 pm to 5:45 pm	Analysis Year:	2010
Filename:	C:\Documents and Settings\TexasEX3		

Phase Sequence and Left-Turn Mode

WB left (1) with WB thru (6)	EB left (5) with EB thru (2)	NB left (3) before SB thru (4)	SB left (7) before NB thru (8)
WB left permitted	EB left permitted	NB left (3) prot-perm	SB left (7) prot-perm

Phase Settings

Approach	Eastbound		Westbound		Northbound		Southbound	
Phase number		2		6	3	8	7	4
Movement		L+T+R		L+T+R	L	T+R	L	T+R
Lead/lag left-turn phase		--		--	Lead	--	Lead	--
Left-turn mode		Perm.		Perm.	Pr/Pm	--	Pr/Pm	--
Passage time, s		2.0		2.0	2.0	2.0	2.0	2.0
Maximum green, s		30		30	25	50	25	50
Minimum green, s		5		5	5	5	5	5
Yellow change, s		4.0		4.0	4.0	4.0	4.0	4.0
Red clearance, s		0.0		0.0	0.0	0.0	0.0	0.0
Walk+ ped. clear, s		19		19		21		21
Recall?:	No	No	No	No	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes

Enable Simultaneous Gap-Out (check = Yes)?
Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑

Enable Dallas Left-Turn Phasing?
Phases 1,2,5,6: ☐ Phases 3,4,7,8: ☐

Protected right-turn with left-turn phase?	n.a.		n.a.		Eastbd. right		Westbd. right	
	No		No		No		No	

Phase number assignment to timers (by ring):					Controller timer ring structure:				
Ring 1:	0	2	3	4	Ring 1:	Timer 1	Timer 2	Timer 3	Timer 4
Ring 2:	0	6	7	8	Ring 2:	Timer 5	Timer 6	Timer 7	Timer 8

Figure D-2. Signal Timing Data as shown from HCM Computation Engine for Example Problem 1.

The specific intersection and saturation flow rate data are shown in Figure D-3. Note that this example assumes fully-actuated control with random arrivals.

Movement-Specific Intersection Data Worksheet												
Approach	Eastbound			Westbound			Northbound			Southbound		
Movement	L	T	R	L	T	R	L	T	R	L	T	R
Movement number	5	2	12	1	6	16	3	8	18	7	4	14
Traffic Characteristics *(Enter the volume data in all columns. For all other blue cells, enter values only if there are one or more lanes.)*												
Volume, veh/h	71	318	106	118	600	24	133	1,644	111	194	933	111
Right-turn-on-red volume, veh/h			0			0			22			33
Percent heavy vehicles, %	5	5		5	5		2	2		2	2	
Lane utilization adjustment factor	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Peak hour factor 1.00	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Start-up lost time, s	2.0	2.0		2.0	2.0		2.0	2.0		2.0	2.0	
Extension of eff. green time, s	2.0	2.0		2.0	2.0		2.0	2.0		2.0	2.0	
Platoon ratio	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Upstream filtering factor	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Pedestrian volume, p/h		120			120			40			40	
Bicycle volume, bicycles/h		0			0			0			0	
(future use)												
Initial queue, veh	0	0		0	0		0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
(future use)												
Multiple-Period Analysis Counts *(if all cell values = 0, then values in the 'Volume' row above will be used for a single-period analysis)*												
Period **1** traffic count, veh												
Period **2** traffic count, veh												
Period **3** traffic count, veh												
Period **4** traffic count, veh												
Intersection Approach Characteristics *(Enter the number of lanes. For all blue cells, enter values only if there are one or more lanes.)*												
Number of lanes	1	2	0	1	2	0	1	2	0	1	2	0
Lane assignment	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.
Average lane width, ft	10.0	10.0		10.0	10.0		12.0	12.0		12.0	12.0	
Number of receiving lanes		2			2			2			2	
Turn bay or segment length, ft	200	999		200	999		200	999		200	999	
Approach Data	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*
Parking present?	No		Yes	No		Yes	No		No	No		No
Parking maneuvers, maneuvers/h	0		5	0		5	0		0	0		0
Bus stopping rate, buses/h			0			0			0			0
Approach grade, %	0	0	0	0	0	0	0	0	0	0	0	0
Detection Data *(Enter values only if there are one or more lanes.)*												
Stop line detector length, ft	40	40	n.a.	40	40	n.a.	40	40	n.a.	40	40	n.a.
(future use)												

Figure D-3. Detailed Intersection Input Data from HCM Computational Engine for Example Problem 1.

RESULTS

Case 1 – Eastbound ATL

Table 1 displays a summary of the analysis results for each iteration applied for the eastbound approach (Iteration 0 encompasses steps 1 through 3). The last row indicates whether the 3-98 model would control the ATL volume calculation, rather than the upper bound calculated based on equal v/s in each lane.

Table D-1. Case 1 analysis results for eastbound results.

Iteration	0	1	2
Thru Volume (vph)	318	240	221
Queue Service Time (effective green) (s)	28	21	20
Equilibrium Cycle Length (s)	102	102	102
g/C	0.27	0.21	0.20
Xt	0.71	0.95	1
ATL vol (vph)	78	97	97
3-98 model governs?	yes	no	no

As shown in Table D-1, the ATL volume prediction converged after two iterations but was ultimately governed by the equal volume-to-saturation flow rate upper bound from the HCM 2010 methodology.

Case 2 – Add Northbound ATL

Table D-2 displays a summary of the process used to determine the ATL volume for the northbound approach (Iteration 0 encompasses steps 1 through 3):

Table D-2. Case 2 analysis results.

Iteration	0	1	2	3
Thru Volume (vph)	1644	1211	1143	1126
Queue Service Time (effective green) (s)	38	20	18	18
Equilibrium Cycle Length (s)	89	66	63	62
g/C	0.43	0.30	0.29	0.29
Xt	1.01	1.43	1.51	1.53
ATL vol (vph)	433	501	518	521
3-98 model governs?	Yes	Yes	yes	yes

As shown in the last row of the table, the 3-98 model controls the ATL volume calculation, rather than the upper bound calculated based on equal v/s in each lane.

APPENDIX E: ATL SIMULATION PROCEDURES

Principles of Lane Change Algorithms

Microsimulation tools explicitly model the movement of individual vehicles using a series of behavioral rules known as algorithms. Among these, lane changing algorithms are most critical for accurately describing ATL behavior. Most simulation models distinguish between "voluntary" and "mandatory" lane changes. *Voluntary lane changes* apply when a driver has multiple lanes available on the desired route, and switches lanes to—for example—pass a slower vehicle. The key point here is that the subject vehicle would have arrived at its desired destination regardless of whether it changed lanes. *Mandatory lane changes* on the other hand, are those that are necessary for performing a turning maneuver, or for prepositioning in anticipation of a downstream lane drop. In other words, a mandatory lane change has to take place if a vehicle is to continue on its desired path. In the application to ATLs, the driver's decision to enter the ATL is generally a consequence of a voluntary lane change (e.g. to pass a queue of vehicles in the CTL). To be precise, the desire for a voluntary lane change is initially triggered by the car-following algorithm if the target vehicle's desired speed exceeds that of a vehicle ahead of it in the same lane. The voluntary lane change then describes the process of searching for suitable gaps in the adjacent lane (in this case the ATL), and then ultimately switching lanes. On the other hand returning from the ATL to the CTL represents a mandatory lane change. Most simulation tools have different parameter sets in their voluntary and mandatory lane change algorithms and the analyst needs to understand the associated settings to accurately model the lane changing behavior.

In the case of mandatory lane changes, a common parameter used in the algorithm is the upstream decision distance. This distance is typically measured relative to the ATL lane drop and refers to the point at which drivers begin to be concerned with the lane drop. E-1 illustrates this concept.

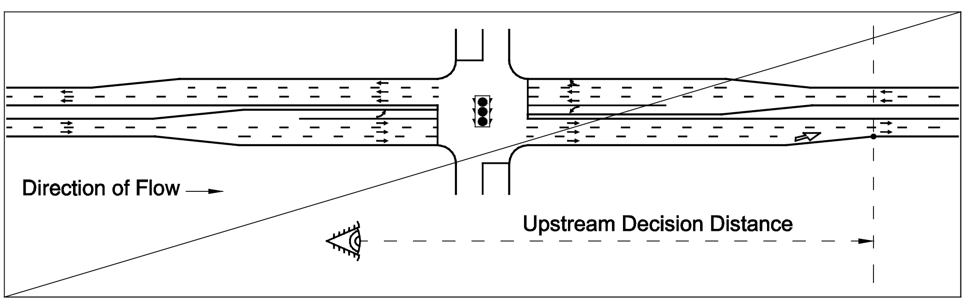


Figure E-1. Illustration of upstream decision distance in simulation.

The upstream decision distance describes the point at which the mandatory lane change algorithm becomes active. In most simulation tools, drivers will begin to try and merge at this decision point if gaps are available, and will become increasingly aggressive about their lane changing behavior as the distance to the downstream drop decreases. Further, in most cases, the mandatory lane change algorithm will override any voluntary lane changes. As a result, no voluntary lane changes will take place past the upstream ATL decision point, and consequently no CTL-to-ATL maneuvers will take place past that point. In this context it is important to emphasize that a coded upstream decision distance that is greater than the total ATL length *will prevent any voluntary lane changes into the ATL and will therefore result in zero through flow on the ATL.*

In this research effort, upstream decision distance (described in VISSIM as the lane change distance, LCD) proved to be the single best predictor of ATL lane utilization and a critical calibration factor to replicate field-observed ATL utilization in VISSIM.

Calibration of Simulation Models

Consistent with any simulation analysis, the parameter set used in the simulation model needs to be calibrated to match field conditions or known relationships in traffic flow theory. Calibration can include various changes to built-in simulation algorithms, including speed distributions, car-following logic, or lane-changing parameters. Significant research is available on the topic of simulation calibration, including material compiled by FHWA in the Traffic Analysis Toolbox. For this discussion, the topic of calibration is condensed to the specific application to ATLs.

The foremost goal in the calibration of a simulated CTL-ATL system is to match the field-observed ATL utilization, or in the absence of field data, the ATL volume predicted from the models presented in these guidelines. By varying the LCD parameter in VISSIM, the research team was able to successfully calibrate 19 of the 22 ATL studied ATL approaches. The remaining three approaches exhibited very low utilization (less than 10%). These low-utilization percentages could not be replicated without also making significant adjustments to the car-following logic, which in turn resulted in more simulated "crashes."

Figure E-2 shows the resulting relationship between field-observed and simulated ATL percentages for the 19 approaches (black) and a best-fit line. The three low-utilization approaches (gray) are treated as outliers.

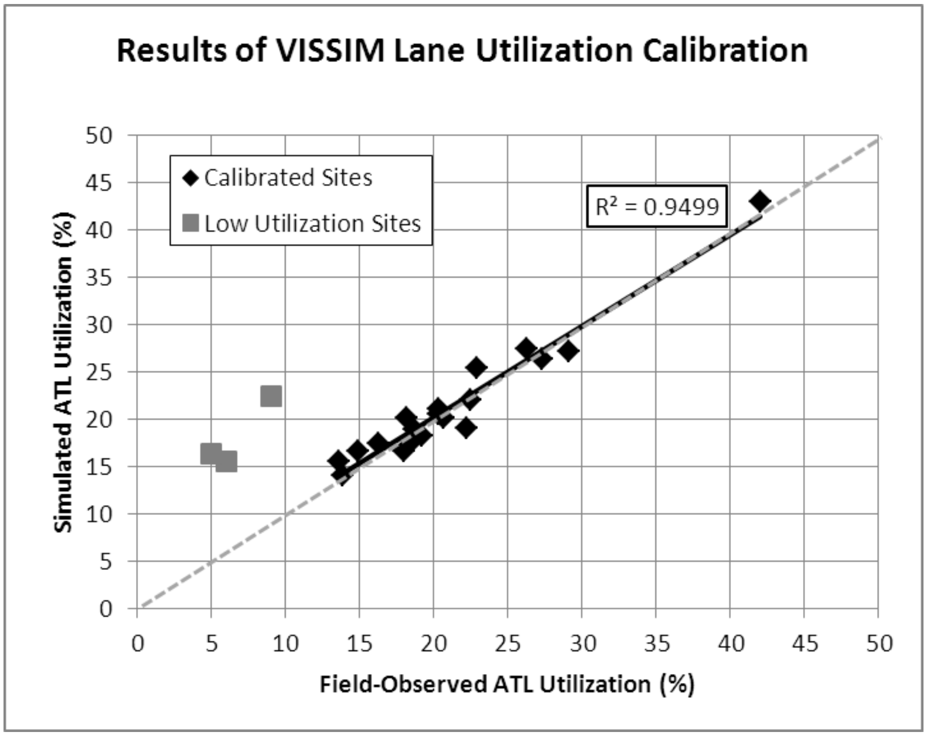


Figure E-2. Calibration result showing field-observed versus simulated ATL utilization.

In interpreting Figure E-2 it should be emphasized that the simulated ATL utilization percentages were the result of free lane selection by drivers on the intersection approach, subject to the algorithms of car-following, lane changing, etc. The ATL utilizations were not "forced" in the sense that a fixed percentage of through traffic was routed through the ATL. In this sense, the resulting R^2 of 0.95 shows a high rate of success in calibrating ATL utilization through the LCD parameter in VISSIM.

Other calibration efforts may include accurate coding of turning-movement flows, speed distributions, signal-timing parameters, etc. The analyst should further validate some of the outputs from the simulation model to field data if available. These outputs may include approach delays, total through travel time, or vehicle queues.

ATL Utilization Prediction Model

Given the sensitivity of the LCD parameter on ATL utilization, an effort was made to predict the correct LCD setting for (future) ATL sites, where the true utilization is unknown. The dependent variable LCD was expressed in the following way:

LCD % Total: the LCD expressed as a percentage of the total ATL length, computed as

$$LCD\%TOTAL = \frac{LCD}{Total\ ATL\ Length} \times 100\%$$

Other forms of the dependent variable were explored, but the one quoted above emerged as the preferred definition. Several explanatory variables were hypothesized to affect the LCD used to calibrate VISSIM, including traffic volumes, approach speeds, upstream and downstream length, and a distinction between single and dual CTLs. Ultimately, the following two explanatory variables were used in the LCD prediction model:

Volume: through traffic flow rate expressed in vehicles per hour (vph)

Upstream: the length of the ATL segment upstream of the stop bar, in feet

The resulting model predicting LCD%TOTAL as a function of these two variables is given below:

$$LCD\%Total = 89.696 - .01125(Upstream) - .0172(Volume)$$

R^2=0.622

The R^2 value suggests that 62.2% of the variability in the LCD variable that provided the best match to the field data is explained by the variables in the model for the regression data set. This suggests that the model can be used to arrive at a reasonable initial estimate for the LCD parameter if VISSIM is used to model the ATL. For other simulation tools, this model may similarly guide an initial parameter estimate, but the model has not been calibrated for such applications.

The model suggests that the LCD begins at 89.696% of the total ATL length, which coincides approximately with the highest LCD value observed in previous VISSIM calibration. That term is then discounted with increasing upstream length and through volume. This implies that ATL utilization *increases* with increasing upstream length and through volume. This is consistent with field observation. A closer exploration of the two explanatory variables also suggests a good model fit as shown in Figure E-3 below.

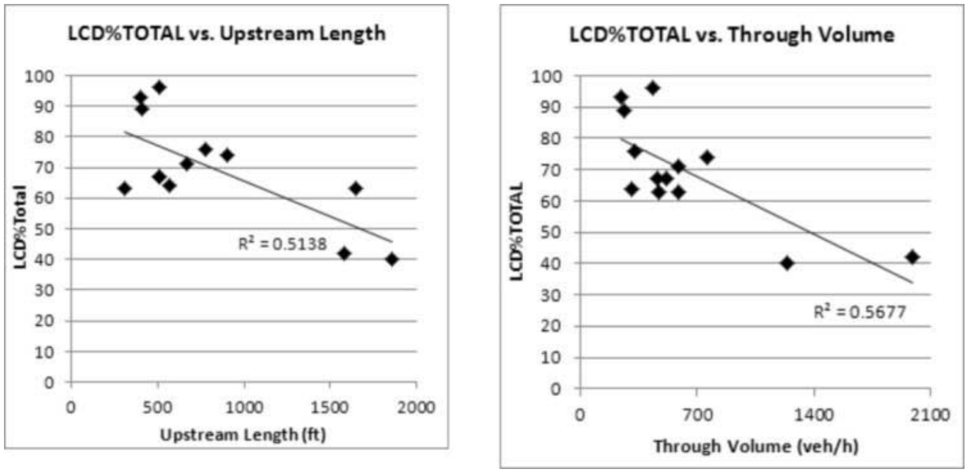


Figure E-3. Sensitivity of LCD%TOTAL versus upstream length and through volume.

Figure E-3 shows that LCD%Total is approximately linear with respect to the upstream length of the ATL and the combined through volume.