University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

Developing a model for the utilisation of auxiliary through lanes at signalised intersections

A dissertation submitted by

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Abstract

Additional through lanes are quite often added at signalised intersections to increase their capacity, these are known as auxiliary through lanes (ATLs). The amount of traffic that uses these lanes relative to the adjacent continuous through lanes (the lane utilisation) is often quite low. This research project aimed to develop a model for predicting the utilisation of these lanes.

The capacity of intersections is often the limiting factor in determining the capacity of a road. Being able to predict this capacity at the planning stage accurately is critical in determining the success or longevity of a potential project. At intersections with ATLs installed, the utilisation of the ATL is a crucial factor in determining the capacity of that intersection. Previous research has indicated that the length of these lanes may be used to assess their utilisation, but to date, determination of a relationship between these two values has not occurred. This project intended to build on this research to develop a relationship between ATL length and utilisation, as well as including other variables that may improve the model accuracy.

This research selected 57 intersection approaches across Australia for inclusion as case study sites. The ATL utilisation for each site was determined, and critical variables were collected, including ATL length, degree of saturation and traffic signal timing parameters. Multiple variable correlation and linear regression methods were implemented to determine the relationship between different combinations of these variables. The strongest of these were then used to compare the model to other methods of ATL prediction identified in the literature review.

The study found that the relationship between single variables and the ATL utilisation were quite weak. However, the results show a moderately strong relationship when variables were combined using multiple regression analysis. The combination of variables that gave the most robust relationship were the number of lanes, cycle time, degree of saturation, ATL departure and approach lengths and the speed limit in that order of significance.

The model developed by this study compares well against other ATL utilisation prediction models in the limited testing undertaken within this study.

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Certification

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Nomenclature

c	=	Traffic signal cycle time (secs)
D	=	Auxiliary lane total length (m) ($d_1 + d_2 + d_3$)
d_1	=	Auxiliary lane approach length (m)
d_2	=	Auxiliary lane intersection length (m)
d_3	=	Auxiliary lane departure length (m)
d_L	=	Auxiliary lane length downstream from stop line (m), i.e. $d_2 + d_3$
E_{HV}	=	Heavy vehicle passenger car equivalent (veh)
f_{et}	=	Factor relating the proportion of turning vehicles to the ease of turning
f_a	=	Saturation flow adjustment factor for area type
f_{bb}	=	Saturation flow adjustment factor for bus blocking effect
f_g	=	Saturation flow adjustment factor for approach grade
f_{HV}	=	Saturation flow adjustment factor for heavy vehicle composition
f_{Lpb}	=	Saturation flow adjustment factor for pedestrian/bicycle effects on left turns
f_{LT}	=	Saturation flow adjustment factor for left-turn vehicle presence
f_{LU}	=	Saturation flow adjustment factor for lane utilisation
f_p	=	Saturation flow adjustment factor for adjacent parking activity
f_{Rpb}	=	Saturation flow adjustment factor for pedestrian/bicycle effect on right turns
f_{RT}	=	Saturation flow adjustment factor for right turn vehicle presence
f_w	=	Saturation flow adjustment factor for width
G	=	Traffic signal displayed green time (secs)
g	=	Traffic signal effective green time (secs)
j	=	Average queue space taken up by a vehicle (m)
$d_{full,a}$	=	Minimum approach auxiliary lane length for full utilisation (m)
$d_{full,d}$	=	Minimum departure auxiliary lane length for full utilisation (m)
d_{min}	=	Minimum downstream auxiliary lane length (m)
N	=	Model calibration factor
N_e	=	Number of exclusive lanes in the movement group
d_p	=	Proportion of d_1 to $d_{full,a}$
P_{HV}	=	Proportion of heavy vehicles in the traffic stream
p	=	Lane utilisation ratio

Q	=	Theoretical lane capacity (veh/hr)
q	=	Demand vehicle flow rate (veh/hr)
q_{TL}	=	Left through lane demand flow rate (veh/hr)
q_{TR}	=	Right through lane demand flow rate (veh/hr)
q_{TC}	=	Centre through lane demand flow rate (veh/hr)
q_{ATL}	=	demand flow rate for auxiliary through lane (veh/hr)
p_u	=	Lane utilisation ratio (upstream) (%)
$p_{d,min}$	=	Minimum lane utilisation ratio (downstream) (%)
s	=	Adjusted saturation flow rate (veh/hr)
s_0	=	Base saturation flow rate (veh/hr)
q_g	=	Demand flow rate for the movement group (veh/hr)
q_c	=	Demand flow rate for critical lane (veh/hr)
q_T	=	Through movement demand flow rate (veh/hr)
X	=	Degree of saturation
X_T	=	Through movement degree of saturation
y	=	ratio of demand flow rate (q) to adjusted saturation flow rate (s)
R^2	=	R-square, Coefficient of determination

Glossary of Terms

CTL = Continuous Through Lane

ATL = Auxiliary Through Lane

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1. Introduction

1.1 Background

The concept of lane underutilisation refers to the phenomenon where drivers choose to use traffic lanes disproportionately, such that a lane has a smaller share of its capacity used than adjacent lanes. For example, one can observe this where a lane is frequently used by turning vehicles; through vehicles tend to avoid using the lane to reduce the chances of having to slow or stop. There are many other causes of lane underutilisation; these are discussed later in the literature review, along with many of the concepts presented here.

Signalised intersections, as an unfortunate side effect of their operation, restrict the capacity of a road by limiting the amount of time each traffic movement is permitted to flow. Road designers quite often place an additional lane locally at the intersection to increase the capacity during the green period that flow is allowed to counteract this side effect, such lanes are termed auxiliary through lanes.

Ordinarily, drivers underutilise auxiliary through lanes at traffic signals due to drivers disliking the need to diverge and merge upstream and downstream of the traffic signals respectively. Drivers perceive this act of merging as a challenge, so the benefits of using the auxiliary through lane must be seen to outweigh this cost. There has been research and efforts put into the design of auxiliary through lanes to try to reduce these effects by making the use of the lane more attractive and making the merging process easier.

The field has known methods to measure the lane utilisation at existing intersections for some time; these can be used to demonstrate the different utilisation rates of lanes and identify if underutilisation is occurring. The development of methods to estimate the utilisation of auxiliary through lanes has also occurred, though most of these are not based on any empirical data and do not perform well when tested against real-world observations

Previous models use auxiliary through lane length as a variable in determining utilisation, but these models do not seem to be based on any data or make assumptions with no apparent justification. These models tend to reflect actual utilisation poorly. However, research does indicate that there appears to be a positive relationship between the length of auxiliary through lanes and their utilisation.

Over the years, research into many other variables that have a good relationship with auxiliary through lane utilisation has occurred. Analysed individually, some of these variables (particularly degree of saturation) have produced strong relationships. It is possible that combining some of these variables with the length variable will deliver even stronger results.

The ability to accurately estimate the capacity of an intersection is critical. If the traffic modelling overestimates lane utilisation of auxiliary through lanes or even neglects the effects of lane underutilisation altogether, the designer can inadvertently exaggerate the capacity of the intersection. In such cases, there is a risk that new intersections will be under-designed, potentially leading to network performance issues, or expensive future retrofit works to enable the network to handle the required flow rates.

It follows then that the ability to accurately forecast the utilisation of an auxiliary through lane at the design stage is a vital component of the overall traffic model.

1.2 Project aim

The overarching aim of this research was to improve the accuracy of analytical & microsimulation models to determine the capacity of signalised intersections with auxiliary through lanes. The project aimed to confirm previous research that a relationship exists between the utilisation of auxiliary through lanes and their length and develop a model for predicting utilisation based on this variable. The project also considered other variables that may explain variations in lane utilisation for inclusion in the model.

1.3 Project objectives

The objectives of this project were to:

1. Examine current methods of determining lane utilisation for existing signalised intersections and the prediction models currently available.
2. Undertake case studies to determine the utilisation of auxiliary through lanes at existing intersections.
3. Determine the key variables contributing to the utilisation of auxiliary through lanes at signalised intersections.
4. Analyse the relationship between the key variables identified.
5. Develop a mathematical relationship between the selected variables and the utilisation of auxiliary through lanes.
6. Estimate lane utilisation of auxiliary through lanes using the developed model and compare the results with other prediction models and case study results.

1.4 Project outcomes & consequential effects

The success of this research will provide traffic engineers and modellers with an accurate method of predicting the utilisation of auxiliary through lanes where they are proposed at new or upgraded signalised intersections, allowing a more realistic calculation of the intersection's capacity and subsequent determination of its design life.

Such improvements in capacity estimation could reduce the likelihood of road authorities designing under capacity intersections, saving funds that might be needed to rectify the issue or upgrade the intersection sooner, due to a design life that was shorter than expected.

1.5 Scope and limitations

- The project did not look to determine the causes of lane underutilisation at intersections, only to determine the effect that a selection of key variables has on utilisation.
- The project only measured lane utilisation during the morning and afternoon peak hour for each intersection.

2. Literature Review

2.1 Overview

The concept of uneven lane utilisation at traffic signals, and indeed other facilities, has been known for some time. As presented below, previous research has developed clear definitions of lane utilisation and determined methods of quantifying it at existing facilities. The literature also identifies apparent relationships between key variables and the utilisation of auxiliary through lanes (ATLs). However, attempts to produce a method of estimating lane utilisation ratios for new or modified signalised intersections have been inaccurate thus far.

2.2 Fundamentals of traffic signal operations

Traffic signals control the flow of traffic through an intersection by allowing movements to flow for a certain period in sequence while prohibiting other (usually conflicting) movements from coinciding.

Ogden and Taylor (1999) present that a phase is a predetermined setting within the traffic signal controller that gives a green signal to some movements and displays a red signal to others, and that a cycle is the complete set of phases run in sequence.

Ogden and Taylor (1999) go on to say that, in principle, a traffic signal site will continuously loop through cycle’s indefinitely, going through each phase in sequence every cycle. However, modern traffic signal controllers use vehicle detection to determine if a phase should be run or not depending on vehicle demand. These systems calculate the demand for a particular movement and modify the phase time to ensure it allocates a sufficient length of time to service that movement. The sum of the lengths of all phase times is the cycle time (c).

Ogden and Taylor (1999) define the period of a phase that displays a green signal as the green time (G). However, the paper explains that not all of this time is useable for full traffic flow as drivers must react to the signal and vehicles do not accelerate instantaneously, terming this as start loss. Conversely, the flow does not stop the instant the green signal disappears, so there is time gained at the end of the phase known as end gain. Ogden and Taylor (1999) and Mannering et al. (2009) introduce the term effective green time (g), which is equal to the actual time that full vehicle flow occurs. Calculating this:

$$g = G - \text{start loss} + \text{end gain} \tag{2.1}$$

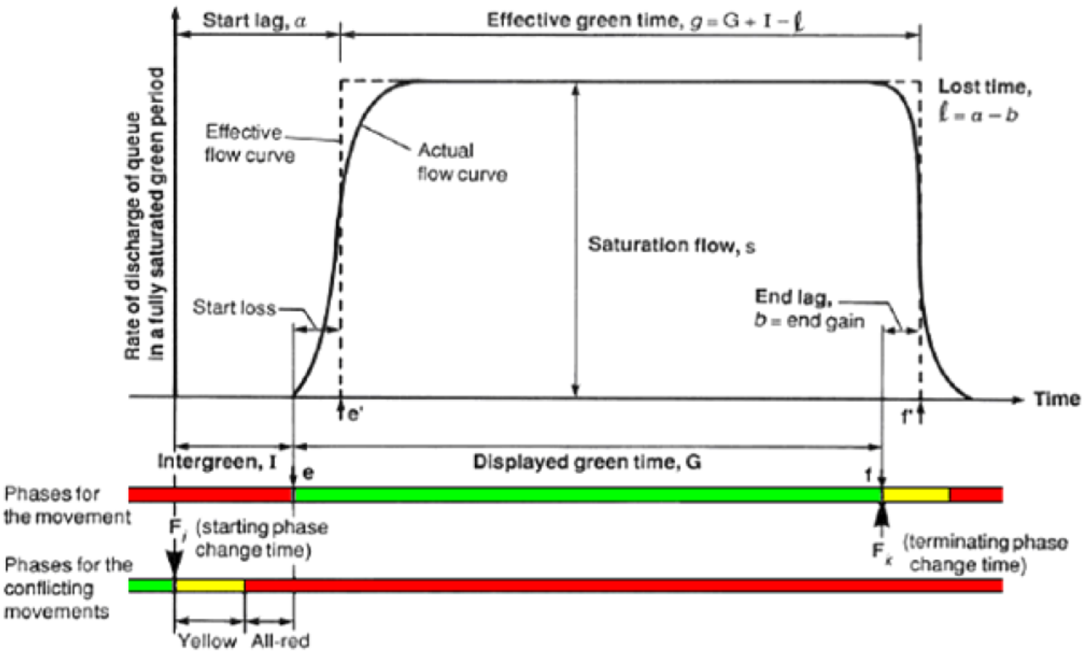


Figure 2-1 Signal phase transition (Akcelik 1981)

Transportation Research Board of the National Academies (2010) gives that without further data, the value of the start loss can be assumed to be equal to the end gain, Akcelik (1981) supports this notion. Assuming this equality results in the effective green time being equal in magnitude to the displayed green time.

The explanation of these traffic signal fundamentals presented by Ogden and Taylor (1999) align with the concepts discussed by Austroads (2013), Austroads (2016) and Akcelik (1981), with the text being a reputable source of information quite frequently referenced in Australian guidelines. The concepts presented are considered credible and are adopted for this report.

Traffic signals can operate as pre-timed, semi-actuated or fully actuated systems. As described by Mannering et al. (2009), a pre-timed system has cycle and phase length settings inbuilt; while the ability to set multiple plans for different times of the day and different days of the week exists, they cannot respond to the prevailing traffic conditions. The current traffic conditions entirely influence a fully actuated system, while the system receives some initial settings, the control is free to manipulate these as required within constraints. A semi-actuated system has approaches operating on both systems.

2.3 The capacity of traffic signals

Austroads (2015) defines the concept of saturation flow (*s*) as the maximum flow rate of vehicles past a point; Ogden and Taylor (1999) support this. Transportation Research Board of the National Academies (2010) calculates flow rate as:

$$q \quad = \quad 3{,}600 \; / \; h \qquad (2.2)$$

Where headway (*h*) is the time in seconds between successive vehicles as they pass a point, measured from the same point on the vehicles. Accordingly, the calculation of the saturation flow (s_0) is then:

$$S_0 \quad = \quad 3{,}600 \; / \; h \qquad (2.3)$$

Headway in this instance refers to time headway, which is the time between the front of one vehicle passing a point on the road, and the front of the immediately trailing vehicle passing the same point (Garber & Hoel 2010). Mannering et al. (2009) described this term and the calculation of the saturation flow rate similarly. Garber and Hoel (2010) define flow as the equivalent hourly rate at which vehicles pass a point on the road during a unit of time.

It is suggested by Transportation Research Board of the National Academies (2010) that an assumed base saturation flow rate of 1,750 veh/hr/lane is appropriate in the absence of more detailed information, subject to local road factors, while Mannering et al. (2009) assumes 1900 veh/hr/lane. Austroads (2017) suggests that a base saturation flow rate of 1,800 veh/hr for single-lane flow and 2,400 veh/hr/lane for multi-lane flow; this is equivalent to headways of 2.0 seconds and 1.5 seconds respectively.

As discussed in section 2.2, traffic signals operate by allocating an amount of time to each movement, with the effective green time equating to that time that vehicles flow. It is logical then that the capacity (*Q*) of the traffic signals to service a given movement per unit of time is a function of the effective green time allocated to that movement and the saturation flow rate for that movement; this is supported by Mannering et al. (2009) and Ogden and Taylor (1999), with their texts calculating the capacity of a lane as:

$$Q \quad = \quad \mathrm{s}\,g \; / \; c \qquad (2.4)$$

Where *s* is the adjusted saturation headway. This equation is supported by the Transportation Research Board of the National Academies (2010), with their text then multiplying this value by the number of lanes to find the total capacity where multiple lanes are present.

Akcelik (1981) defines the ratio of the arrival flow rate (q) to the saturation flow (s) as the flow ratio (y):

$$y = q / s \tag{2.5}$$

and the ratio of arrival flow rate (q) to the capacity (Q) as the degree of saturation (X):

$$X = q / Q = q c / s g \tag{2.6}$$

Transportation Research Board of the National Academies (2010) describes a similar equation, but terms it the 'volume-to-capacity ratio'.

It follows that for a movement to be fully serviced:

$$Q > q \tag{2.7}$$

$$\text{i.e. } s g > q c \tag{2.8}$$

and

$$X < 1 \tag{2.9}$$

Akcelik (1981) and Ogden and Taylor (1999) both contend that failing to meet the above criteria vehicle queues will continue to grow. Transportation Research Board of the National Academies (2010) contends that a volume-to-capacity ratio (or degree of saturation) greater than 0.95 can result in flow breakdown, given the inherent variability in traffic flow.

The Australian Road Research Board commissioned the Akcelik (1981) and is considered a reliable source; the concepts and equations it presents also pass the logic test when considered from first principles. This research adopts these concepts, as well as those offered by Ogden and Taylor (1999) and deems them reliable.

Transportation Research Board of the National Academies (2010) and Austroads (2015) are both reference material used internationally and domestically by road authorities; as such, the information is considered highly credible.

2.4 Reasons for installing auxiliary through lanes

Transportation Research Board of the National Academies (2011) contends that signalised intersections form capacity 'choke points' on a road, as by their very nature they limit the amount of time available for traffic along a road to flow. The report adds that auxiliary through lanes can be added to increase the stop line capacity of a signalised intersection by increasing the number of lanes available for flow to occur. The report suggests that they are added in place of continuous through lanes when it is not feasible to install a continuous lane or the auxiliary lane is enough in terms of capacity.

2.5 Current methods of estimating traffic flow and capacities

For a multitude of reasons, it is important to be able to accurately estimate the capacity of traffic signals to service each movement. As discussed, capacity is a function of the saturation flow and the effective green time. It is simple enough to calculate the effective green time, and there is a consensus on the method explained in section 2.2 to do this. There is also general agreement on how to calculate the theoretical saturation flow for a single lane of traffic. Akcelik (1981) presents the method as taking the maximum theoretical flow rate and reducing it based on factors such as lane widths, gradient, heavy vehicle composition and operating environment (i.e. side friction from driveways and parking). Transportation Research Board of the National Academies (2010) presents a similar method. It suggests that, without further detailed information, an assumed base saturation flow rate (s_0) of 1,900 applies for metropolitan areas greater than 250,000 in population, otherwise 1,750 pcu/hr/lane. The adjusted saturation flow rate is then calculated using a series of factors according to the following equation:

$$s = s_0 f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb} \quad (2.10)$$

where:

f_w = adjustment factor for width

f_{HV} = adjustment factor for heavy vehicle composition

f_g = adjustment factor for approach grade

f_p = adjustment factor for adjacent parking activity

f_{bb} = adjustment factor for bus blocking effect

f_a = adjustment factor for area type

f_{LU} = adjustment factor for lane utilisation

f_{LT} = adjustment factor for left-turn vehicle presence (note: right drive rule)

f_{RT} = adjustment factor for right turn vehicle presence (note right drive rule)

f_{Lpb} = adjustment factor for pedestrian/bicycle effects for left turn groups

f_{Rpb} = adjustment factor for pedestrian/bicycle effects for right turn groups

The maximum theoretical flow rate is presented in Austroads (2015) as time divided by vehicle headway, a concept widely accepted in the traffic engineering field. Austroads (2017) takes the HCM method defined by the Transportation Research Board of the National Academies (2010) and simplifies it to use the key variables:

$$Q = 2400\, f_w\, f_{HV} \tag{2.11}$$

where:

Q = capacity (pcu/hr)

f_w = adjustment factor for width (Table 2-1)

f_{HV} = adjustment factor for heavy vehicles

$$= 1 \,/\, [\, 1 + P_{HV}\ (\, E_{HV} - 1\,)] \tag{2.12}$$

P_{HV} = proportion of heavy vehicles in the traffic stream

E_{HV} = average passenger car equivalents for heavy vehicles (Table 2-2)

Table 2-1 Width factors (Austroads 2017)

Lateral Clearance Each Side (m)	**Lane Width (m)**		
	3.7	**3.2**	**2.7**
2	1.00	0.90	0.70
1	0.90	0.80	0.63
0	0.65	0.60	0.50

Table 2-2 Heavy vehicle passenger car equivalents (Austroads 2017)

Grade	**Passenger Car Equivalents**
Level	2.0
Moderate	4.0
Long Sustained	8.0

When it comes to multiple lane approaches to traffic signals there are a few different methods used in practice to predict the spread of flow rates across the approach lanes, and thus the total flow capacity for that group of lanes. Akcelik (1989a) describes the following three methods:

- The ‘Equal Degree of Saturation’ method assumes that each lane has the same degree of saturation. In this method, if lane capacities differ, then the flows will be unequal due to the requirement to maintain equal degrees of saturation. This method assumes that drivers will choose the lane with the least congestion (lowest degree of saturation) to clear the intersection in the minimum number of cycles, and thus the degree of saturation across lanes will equalise. The popular microsimulation software package SIDRA (Akcelik & Associates Pty Ltd 2019) adopts this method.
- The ‘Equal Average Delay’ method assumes that drivers will choose the lane that has the least delay, regardless of congestion or queue length.
- The ‘Equal Queue Length’ method assumes that drivers will choose the lane with the shortest queue regardless of delay and congestion. In practice, the Equal Degree of Saturation method produces results somewhere in between the other two methods mentioned.

Chen et al. (2012) also cites the Equal Degree of Saturation method and discusses the following two methods as well:

- The ‘Equal Flow Ratio’ method is a similar method to the Equal Degree of Saturation Method but disregards any differences in effective green times present across lanes.
- The ‘Equal Lane Volume’ principle assumes that all lanes within a group will have equal volumes.

Akcelik (1989a) introduces the term lane utilisation ratio (p_i), describing it as the ratio of the degree of saturation of the subject lane (X_i) to that of the critical (largest) degree of saturation lane on the approach (X):

$$p_i = X_i \ / \ X \tag{2.13}$$

Akcelik (1989a) explains that for the equal degree of saturation method:

$$X_1 = X_2 = \ldots = X_n$$

therefore:

$$p_1 = p_2 = \ldots = p_n = 1$$

where n is the number of lanes.

2.6 Auxiliary through lane flows

The lane flow distribution methods discussed in section 2.5 may be appropriate for continuous through lanes (CTLs) at an intersection, however, when one observes an intersection with an auxiliary through lane (ATL), they will soon notice that the flow volume in the ATL is much less than in CTLs. These observations have been documented in many studies, including those by Bugg et al. (2013), Karma et al. (2010) and Royce et al. (2006); this is a specific example of lane underutilisation, Akcelik (1989a) terming this phenomenon the 'short lane effect'.

Lane underutilisation can occur for many reasons, with some of them discussed in section 2.7.

2.7 Causes of lane underutilisation

For freeway segments, Lee and Park (2012) determined that drivers choose their lane based on their desired speed, vehicle performance, origin-destination of trips and the downstream traffic conditions. Okura and Somasundaraswaran (1996) concluded that the volume of traffic flow, heavy vehicle composition and average speed were significant contributors to lane selection. The research determined that upstream flow conditions affected lane selection, in addition to the downstream factors identified by Lee and Park (2012).

For lanes at intersections, Royce et al. (2006) identified the following variables that effected lane choice, and thus lane utilisation:

- Short approach lanes
- Short adjacent left-turn lanes
- High volumes of turning traffic in shared lanes
- High volumes of pedestrians conflicting with turning traffic in a shared lane
- Origin of motorists (they may be forced to enter a nearside lane upstream of signals)
- Destination of motorists (they may select their lane based on their downstream destination)
- Number of lanes at the intersection
- Requirement and difficulty of merge downstream of signals
- Length of phase green time
- Congestion levels/ degree of saturation
- A high percentage of heavy vehicles in the lane
- Buses and bus stops upstream/ downstream of signals
- Parking/ loading upstream/ downstream of signals

Akcelik & Associates Pty Ltd (2019) reflects these conditions.

Karma et al. (2010) identified that from the various causes, the most significant contributors to lane utilisation are:

- Short auxiliary lanes (upstream and downstream)
- Shared Lanes
- The occurrence of lane blockage
- Side friction from driveway and parking.

2.8 Reasons for auxiliary through lane underutilisation

This report focusses on the short lane effect for auxiliary through lanes (ATLs), as termed by Akcelik (1989a) and discussed by Karma et al. (2010).

Bugg et al. (2013) suggest that approaching drivers will naturally tend to select the lane that they believe will minimise their travel time through the intersection while upholding a certain level of comfort and courtesy to other drivers. Drivers will balance these two elements between the desire to use a short auxiliary through lane to save time, and the need to diverge and merge – causing a level of discomfort to the driver and discourtesy to other motorists. These behaviours were hypothesised based on the following observations:

- Drivers that arrived during a red signal generally remained in the CTL, unless the queue in this lane became too long.
- Drivers that arrived early during a green signal followed the same principle, probably perceiving that the green time would be sufficient for them to clear the intersection.
- Drivers arriving late during the green signal typically only used the ATL to overtake a slow vehicle in the CTL, possibly perceiving that the green signal would soon end.
- Occasionally, drivers that initially joined the CTL queue sometimes switched to the ATL after its queue had discharged, realising that they may not clear the intersection in the CTL.

Akcelik (1981) made similar conclusions, stating that the ATL will experience equivalent saturation flow as other lanes for the length of time it takes to clear vehicles queued in this lane after a red signal. However, once traffic begins to flow, vehicles are unlikely to enter the ATL, and thus it will have a reduced flow rate.

Bugg et al. (2013) concluded their analysis by suggesting that ATL use is a function of the arrival phase and the queue lengths in each lane. The research theorised that this intuitively aligns with the decision point of the driver; with drivers arriving on a red phase choosing their lane based on queue lengths and are twice as likely to join the queue in the CTL as the ATL.

When arriving in a green phase, drivers were found to tend to remain in the CTL if they perceived they could pass through the intersection before the end of the green phase; otherwise they may shift to the ATL if the queue is shorter. Tarawneh and Tarawneh (2002) expanded on this by stating that the upstream length of the ATL must be long enough to extend beyond the back of the CTL queue to allow vehicles to enter it. Mannering et al. (2009) calculated the number of vehicles in a queue by multiplying the arrival rate by the length of the red signal, the same method also proposed by Garber and Hoel (2010). Calculating this length is done by multiplying the number of vehicles by the length of the space that they occupy (including leading and trailing gaps), *j*.

Aligning with Bugg et al. (2013)'s conclusion and observations, McCoy and Tobin (1982) found that a longer green time leads to lower utilisation of the ATL, due to the increased likelihood of arriving during the green phase. The report also found that ATL use increases with the total length of the lane, but taper length had negligible effect.

Tarawneh and Tarawneh (2002) also found that the downstream length of ATLs and downstream turning volumes (either for driveways or side roads) significantly affected the utilisation of ATLs. The report found that the utilisation factors suggested by the Transportation Research Board of the National Academies (2010) were much higher than those calculated from their trial sites.

Karma et al. (2010) suggest that the utilisation of ATLs varies with the level of traffic flow throughout the day, aligning with the conclusions made above. Based on the observations made in the study, it suggests that the main factor contributing to the use of an ATL was the length of these lanes.

Royce et al. (2006) found a direct correlation between the length of an ATL and its use. However, due to the limited sample size of this study, a mathematical relationship was unable to be produced.

Transportation Research Board of the National Academies (2011) states that, based on empirical evidence, they have found that the two critical variables in determining ATL utilisation are the through movement degree of saturation on the subject approach and the approach geometry. However, if downstream lengths are too short, they may discourage drivers from choosing the lane due to the difficulty merging back into the adjacent continuous lane.

Yang et al. (2018) found that lane changing frequency increased with traffic density in a freeway situation. Density increases with the degree of saturation, so this would support the above research.

2.9 Effect of travel speed on lane changing

Golbabaei et al. (2014) undertook their research using a calibrated microscopic traffic simulator; it showed that reducing the speed deviation amongst vehicles decreased the amount of lane changing significantly. Archer et al. (n.d.) hypothesised that given lower speed limits would produce a smoother traffic stream; this would result in reduced lane-changing friction and travel speed variance amongst vehicles. Brubacher et al. (2018) found that speed limit changes might have a negligible effect on speed variance, though if there were any minor effect, increasing speeds would increase the variation slightly. This research would tend to indicate that, while it may only be marginal, lower speed limits would result in less lane-changing manoeuvres.

However, in conflict to this, Soriguera et al. (2017) found that for moderate demand levels, lower speed limits resulted in an increased probability of lane changing.

All this research analysed uninterrupted flow environments such as freeways, so their applicability to lane changing at intersections is unknown.

2.10 Determining lane utilisation for existing intersections

Akcelik (1981) states that the lane utilisation for each lane can be calculated by direct measurement of the respective degrees of saturation (see section 2.5). As mentioned in section 2.3 and equation 2.6, the degree of saturation is the flow rate (for the period analysed) divided by the saturation flow rate.

As mentioned in section 2.5, Transportation Research Board of the National Academies (2010) states that a typical base saturation flow rate can be assumed, with individual lane saturation flows calculated from a set of adjustment factors applied to the base rate in line with equation 2.10. It follows then that two lanes with similar conditions would have the same adjustment factors and therefore similar saturation flow rates.

If equation 2.6 is substituted into equation 2.13:

$$p_i \quad = \quad X_i \ / \ X \quad = \quad (q_1\, c_1 \ / \ s_1\, g_1) \ / \ (q\, c \ / \ s\, g)$$

From equation 2.10, for two lanes with the same prevailing conditions:

$$s_1 \quad = \quad s_2 \quad = \quad s_3 \quad = \quad \ldots \quad = \quad s$$

and given the lanes are on the same approach under the same signal phase:

$$g_1 \quad = \quad g_2 \quad = \quad g_3 \quad = \quad \ldots \quad = \quad g$$

and:

$$c_1 = c_2 = c_3 = \ldots = c$$

Substituting:

$$p_i = q_1 / q \tag{2.14}$$

From this, lane utilisation can be calculated by directly comparing the flow rate of the subject lane to that of the critical (highest flow) lane, provided the above conditions are satisfied. Akcelik (1981) proposed a similar methodology, showing that:

$$p_i = y_i / y$$

Where y is as defined in section 2.3.

2.11 Current methods of estimating lane utilisation

For new signalised intersections, or where changing an existing signalised intersections, it is not possible to directly determine the lane utilisation before construction. The designer must estimate the lane utilisation rates in these cases.

(Bugg et al. 2013) had developed a lane choice model based on their observations (discussed in 2.8) and while it was able to determine relative probabilities, the report concluded that it was not accurate at predicting actual vehicle usage of each lane.

2.11.1 Australian Road Capacity Guide method

Early research presented by Australian Road Research Board (1968) considered the utilisation of short upstream auxiliary through lanes as a factor of its length.

Where D in Figure 2-2 equals d_1 in the terminology of this report.

It states that if:

$$d_1 \leq s\,g / 150 \tag{2.15}$$

A reduction in the saturation flow for the kerbside lane occurs according to the equation:

$$s = s_0\,[d_p + f_{et}\,(1 - p)] \tag{2.16}$$

Where d_p is the proportion of the actual distance d_1 to the minimum required by equation 2.15, and f_{et} is a factor that relates the proportion of turning vehicles and the ease of turning if the lane is a shared lane. If there are no turning vehicles, F equals 0.03.

This model assumes that if d_1 is longer than required, then the length of the short lane approach does not reduce the flow rate.

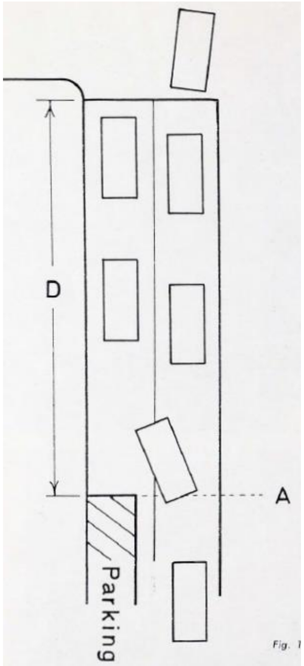


Figure 2-2 - Auxiliary lane length (Australian Road Research Board 1968)

2.11.2 Australian Road Research method

Akcelik (1981) states that if lane flows are not known, such as when designing a new site, the flows can be estimated using:

$$q_{Ti} = q_T / 2n \qquad (2.17)$$

Where q_T is the total traffic flow for the movement, q_{Ti} is the traffic flow in the underutilised lane and n is the number of lanes servicing that movement. The approach used determines the flow for the underutilised lane and divides the remaining through flow evenly amongst the remaining lanes.

Karma et al. (2010) referenced this method, simplifying to say that applying this to two, three and four lane approaches suggests underutilised lane flows of 25%, 16% and 12.5% of the total through-flow respectively.

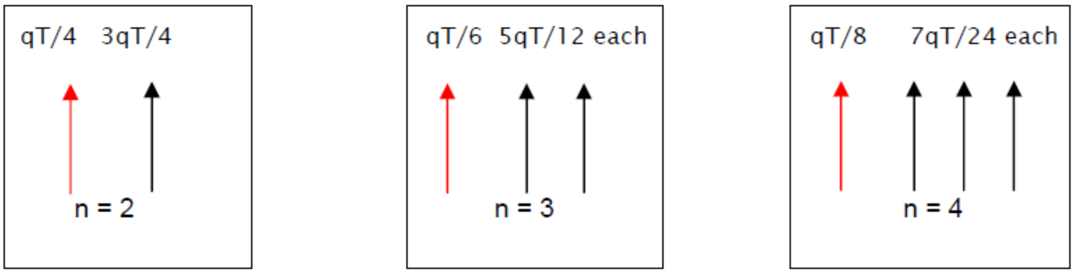


Figure 2-3 – Australian Road Research method Lane flow estimations (Karma, Douglas-Jones & Jaglal 2010)

Unfortunately, this is a rudimentary model that does not appear to have used any evidence or research in its development.

A later reprint of this report by Akcelik (1989b) provided a methodology similar to that of Australian Road Research Board (1968) in section 2.11.1 for the saturation flow rate of a short lane:

$$s_0 \quad = \quad 3600\, d_1 \,/\, j\, g \qquad (2.18)$$

where j is the average queue space (m) taken by each vehicle.

2.11.3 SIDRA method

SIDRA Intersection is a micro-analytical program developed by Akcelik & Associates Pty. Ltd., Akcelik & Associates Pty Ltd (2019) describes the short lane utilisation model used within the program, which essentially breaks utilisation into three zones:

$$\text{For } d_L > d_{full},\ p_u = 100 \qquad (2.19a)$$

$$\text{For } d_L \leq d_{min},\ p_u = p_{d,min} \qquad (2.19b)$$

$$\text{For } d_{min} < d_L \leq d_{full}\ ,\ p_u = p_{d,min} + (100 - p_{d,min})\ [(d_L - d_{min}) \,/\, (d_{full} - d_{min})]^N \quad (2.19c)$$

Where:

p_u = Lane utilisation ratio (upstream) (%)

$p_{d,min}$ = Minimum lane utilisation ratio (downstream) (%)

d_L = Downstream auxiliary lane length (m) = $d_2 + d_3$

d_{min} = Minimum downstream short lane length (m)

d_{full} = Downstream short lane length for full utilisation (m)

N = Model calibration factor (typically ~1.2)

d_3 is measured from the approach stop line to the point of merging into the adjacent lane. The program assumes d_{full} = 200 m, d_{min} = 30 m, N = 1.2 and $p_{d,min}$ = 20%. It is not stated how these values have been determined.

Akcelik & Associates Pty Ltd (2019) provides the following figure describing this relationship (where $L_s = d_L$):

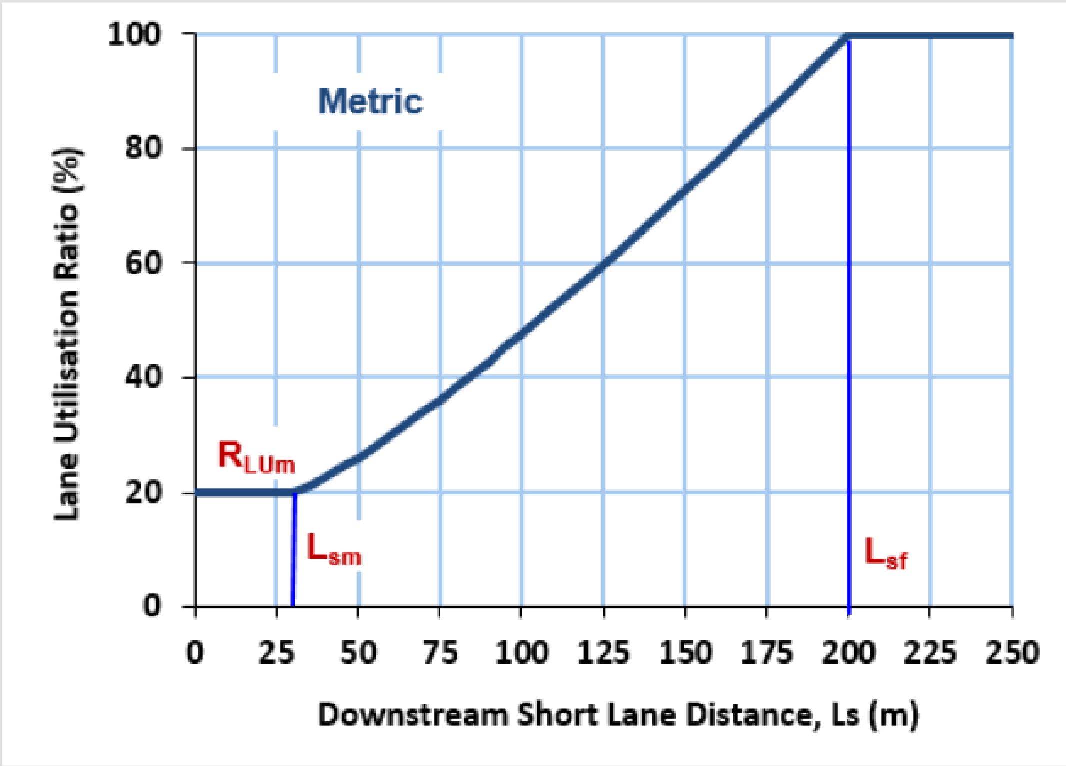


Figure 2-4 - SIDRA lane utilisation (Akcelik & Associates Pty Ltd 2019)

The method assumes a lane length of 200 m is sufficient for full utilisation; however, there is no evidence provided to support this and in fact field trials have found this to be inaccurate (Karma et al. 2010).

Sample tests by Karma et al. (2010) have shown that it significantly overestimates lane utilisation when compared to observed rates.

2.11.4 Highway Capacity Manual method

Transportation Research Board of the National Academies (2010) applies an adjustment factor, f_{LU}, to the total base flow saturation for an approach as described in section 2.3:

$$f_{LU} = q_g \; / \; N_e \, q_c \tag{2.20}$$

where:

q_g = Demand flow rate for the movement group (veh/hr)

q_c = Demand flow rate for the single exclusive lane with the highest flow rate (veh/hr)

N_e = Number of exclusive lanes in the movement group (lanes)

The report provides default values for f_{LU} for use on intersection approaches but notes that these are not applicable for situations such as short lane drops. Institute of Transportation Engineers (2008) mimics the approach taken by the Highway Capacity Manual 2010. Transportation Research Board of the National Academies (2011) reiterated that the HCM2010 does not factor in lane utilisation for short lanes, with Lee et al. (2005) making similar findings.

The method does not estimate the actual lane utilisation but instead applies a correction factor to the overall saturation flow rate for an intersection approach.

2.11.5 Transport Research Board method

Transportation Research Board of the National Academies (2011) NCHRP Report 707 suggests that the primary variable in determining ATL utilisation is the through movement degree of saturation. The report presents some empirical data and gives the following equation for calculating the short auxiliary lane vehicle volumes:

$$q_{ATL} = 20.226 + 81.791 \text{ x } X_T^2 + 1.65 \text{ x } q_T^2 \; / \; 10{,}000 \tag{2.21}$$

where:

q_{ATL} = Predicted auxiliary lane flow rate (veh/ hr)

X_T = Through movement Degree of Saturation

q_T = Through movement flow rate (veh/hr)

Upper bounds on these volumes are placed, limiting the flow to an equal degree of saturation across all lanes.

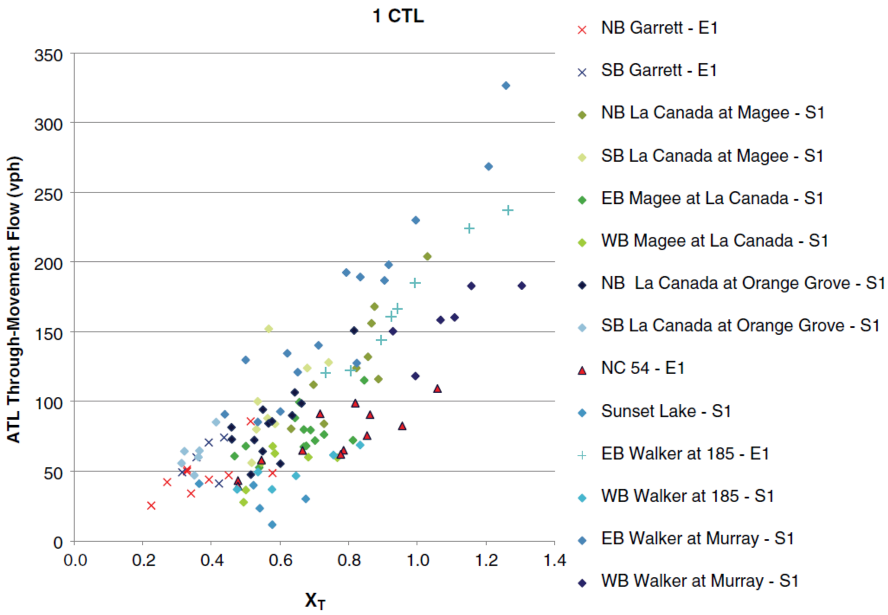


Figure 2-5 Transport Research Board model scatter plot (Transportation Research Board of the National Academies 2011)

The report states that this equation has an R-square value of 0.781 for the empirical testing undertaken.

2.11.6 Use of auxiliary through lane length as a model variable

Two of the models examined in this section use flow rates to determine lane utilisation, however, as discussed above, these do not appear to be based on any research and tend to overestimate the utilisation of the short lane.

Initial small scale research by Royce et al. (2006) identified that there was a direct correlation between the length of ATLs and their usage rates. While the length was a crucial variable in two of the models discussed, it found that the use of short lanes was substantially less than that predicted by these methods.

Karma et al. (2010) undertook further research to confirm the existence of the relationship between ATL length and utilisation at sites across New Zealand. The correlation was confirmed ($R^2 = 0.75$), though not as robust as the original research by Royce et al. (2006). Unfortunately, while the pattern was confirmed, the study collected insufficient data to develop a reliable model for predicting short lane utilisation.

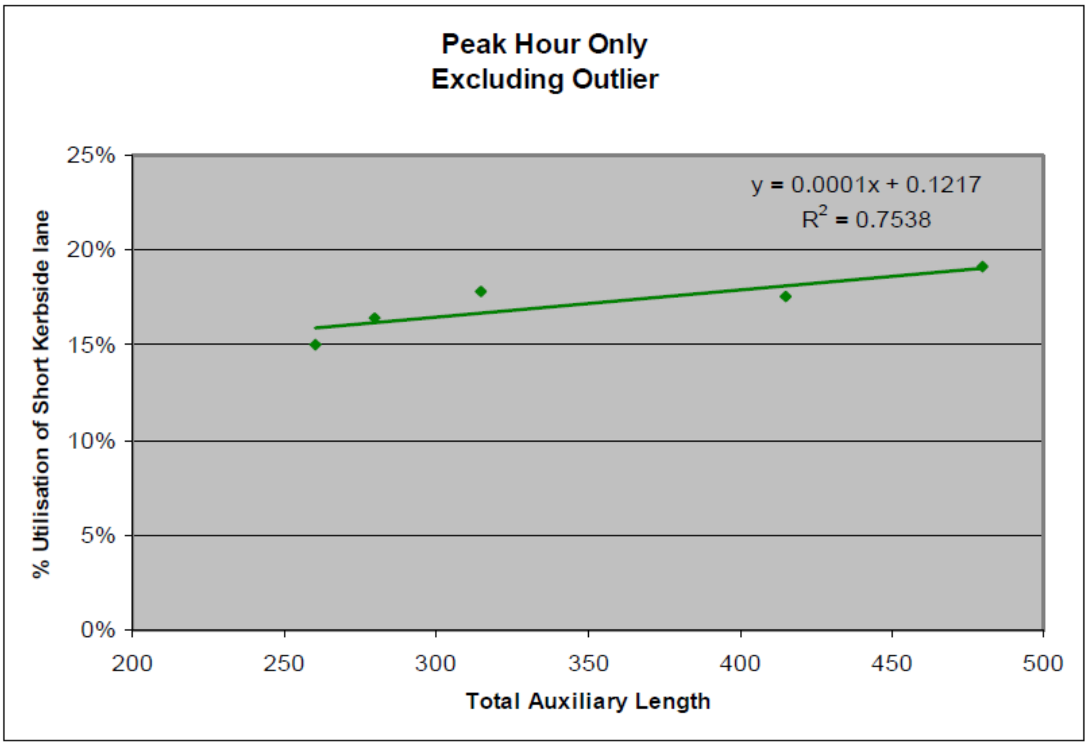


Figure 2-6 – Length to utilisation correlation (Karma et al. 2010)

Lee et al. (2005) also found that the length of the lane formed a positive relationship with its utilisation.

2.12 Short auxiliary through lane design guidance

Transportation Research Board of the National Academies (2011) gives that many of the design criteria that apply to continuous lanes should also apply to auxiliary lanes:

- The geometric design should meet driver expectations;
- Signing and pavement markings should reinforce the message conveyed by the geometric design of the lane;
- Adequate sight distance should be provided to facilitate decision making and emergency stops; and
- Locate driveways and other impedances outside of the intersection influence area (this includes the entire length of the short auxiliary lane).

Transportation Research Board of the National Academies (2011) goes on that the unique design characteristics that apply to ATLs are the determination of their length and the use of applicable signs and line marking. It suggests that the upstream length should be adequate to contain the predicted queue length for the lane, but also be longer than the queue length in the adjacent continuous lane to ensure that the ATL remains accessible throughout the cycle. The downstream length is advised to be long enough to allow a stopped vehicle to accelerate to the prevailing traffic speeds before reaching the merging taper. The availability of gaps in the adjacent CTL also needs consideration, to allow comfortable and safe merging. The report implies that extending the length may be required if the availability of gaps is low.

In section 2.11.1, Australian Road Research Board (1968) spelt out a similar requirement for approach length based on the likely back of queue length.

2.13 Research methodology used in similar research

Karma et al. (2010) used the following methodology sequence in their research to develop a correlation between ATL utilisation and auxiliary lane length:

- Development of site selection criteria
- Development of study limitations
- Selection of sites
- Grouping of sites by number of lanes
- Collection of traffic volume data from SCATS
- Collection of geometric data against aerial layout plans
- Compare utilisation to lane length
- Compare utilisation to demand volumes
- Analyse prediction methods for lane utilisation.

Following the development of this relationship, Karma et al. (2010) suggested that future research would gather a larger statistical sample of case study sites, and use multiple linear regression analysis to develop a model for relating auxiliary lane length and utilisation.

In their report on traffic distribution on three-lane freeways, Okura and Somasundaraswaran (1996) followed a similar method, by first selecting a series of sample sites and collecting the applicable data. The independent and dependent variables were then plotted to visualise the patterns in the data, and then a model was developed using stepwise regression analysis. The report developed two different models here. The models were compared against R-squared values to determine the accuracy of the fit of each. From this, it was possible to decide on which model was more accurate and which independent variables had the most significant impact on the dependent variable. Hurley (1998) followed a similar methodology in his study into the utilisation of lane reductions downstream from double turning lanes.

Bugg et al. (2013) was interested in the likelihood of a driver choosing the auxiliary lane based on a given scenario, so the methodology used statistical analysis methods such as probit and logit models to develop the relationship, however the preceding methodology aligned with that used by Karma et al. (2010) and Okura and Somasundaraswaran (1996).

Transportation Research Board of the National Academies (2011) developed their model for lane utilisation based on the degree of saturation using the following methodology:

- Set data collection requirements
- Collection of field data from case study sites
- Plot various independent variable against the dependent variable (utilisation) to compare the relationship visually.
- Develop a single variable relationship using least squares (regression) and compare the resulting accuracy of fit.
- Compare the relevance of the selected variables.
- Undertake non-linear multi-variable regression analysis to develop a model of the relationship from the key variables identified earlier. Development of two models was required, with upper limits placed on these.

3. Background Information

3.1 SCATS (Sydney Coordinated Adaptive Traffic System)

Lowrie (1992) describes SCATS as

> 'a computer-based area traffic signal control system. It is a complete system of hardware, software and control philosophy. It operates in real-time, adjusting signal timings throughout the system in response to variations in traffic demand and system capacity.'

This description carries on to state that the primary purpose of the SCATS system is to minimise stops and thus overall delays within the system; this maximises the total system capacity and reduces the possibilities of traffic jams by controlling queues.

The program is an adaptive system, as opposed to a fixed-time system; this means that the system modifies cycle times and phase splits in response to real-time vehicle demand. These variables are reviewed every cycle and can be altered every cycle if required (Lowrie 1992). The system modifies the cycle time to maintain a degree of saturation overall of around 0.9, with phase splits varied by a small amount each cycle to give equal degrees of saturation on competing approaches (Roads and Maritime Services NSW n.d.).

Figure 3-1 SCATS Interface (Department of State Growth & Wyminga 2019)

The system was developed in 1975 by the (then) Department of Main Roads NSW, now known as Roads and Maritime Services NSW, to provide more capacity and efficiency within the Sydney road network, however, it has since been adopted by Australian and New Zealand Government road authorities (as well as some overseas jurisdictions) as the preferred system of choice, with all major and minor cities in Australia and New Zealand using SCATS (Roads and Maritime Services NSW n.d.).

The SCATS system records all the data it uses in the SCATS Traffic Reporter system. There are two modules within the Traffic Reporter, namely the Strategic Monitor and Traffic Flow modules. The Strategic Monitor can be used to extract the historical timing data used at the site; for this research, it includes the cycle times and phase times used. The Traffic Flow module records the number of vehicles crossing each detector within a specified time – as small as 5-minute bins (Department of State Growth & Wyminga 2019).

3.2 Vehicle detectors at traffic signals

Federal Highway Administration (2004) advises that vehicle detectors (sensors) are used to inform a traffic signal controller (the computer that controls the traffic signals) of the presence of a vehicle at a specified location at an intersection, such as a particular lane on a specific approach. They can also be used to determine the presence of a pedestrian or cyclist.

Federal Highway Administration (2006) goes on to say that traffic monitoring data, both in a real-time and historical sense, is collected using these vehicle presence signals as well.

Roads and Maritime Services NSW (n.d.) state that while the SCATS system can operate with many detector technologies, a preference for inductive loop detectors exists because of their high measurement accuracy and reliability. Mannering et al. (2009) describe this technology as a loop (or coil) of wire embedded in the pavement that has an electrical current passed through it. When a metal object (such as a vehicle) passes over the wire loop it alters the inductance level of the wire; this change in current is picked up by a monitor and registers as a vehicle.

Figure 3-2 SCATS vehicle detector layout (Department of State Growth & Wyminga 2019)

3.3 Methods of collecting lane flow data

Akcelik (1981) describes a technique used to determine lane flow and saturation flow rates by manual counts. The methods require counting the number of vehicles flowing in three different time segments of the movement phase.

Karma et al. (2010) have shown that using detector loops to count vehicles (such as SCATS) is accurate to within 10-15% of manual vehicle counts when calculating lane utilisation at existing sites.

4. Methodology

4.1 Selection of case study sites

This research used case study sites to provide real-world data to analyse. A sufficiently large sample of case study sites must be analysed to ensure the accuracy and application of the resulting model.

As discussed in detail in Chapter 2 Literature Review, there is a significant number of variables that can contribute to the utilisation of auxiliary through lanes. Research has been undertaken to identify those variables that have the most significant effect on auxiliary through lane utilisation, these are the subject of this report, but the project cannot discount the sum of those variables with minor impacts. Case study sites were selected where these minor variables were not present to remove the effect of these variables and isolate those variables that are the focus of this study.

The scope of what constitutes a short auxiliary through lane also needed to be defined. For this study, a short auxiliary through lane is a lane that is added on the left of a continuous through lane immediately before an intersection and is dropped from the left immediately following an intersection. An upper limit of 1000 m applied to the commencement and termination of the lane. The study did not consider a drop off of a previously continuous lane after an intersection or the addition of a continuous lane following an intersection.

The following set of site selection criteria applied in the selection of sites for this study:

- The intersection is signalised.
- The intersection is controlled by SCATS.
- All approach lanes must have a separate vehicle detector.
- A short auxiliary through lane must be present, as defined earlier.
- Turning movements must separate from the auxiliary lane before the intersection.
- Bus stops should not be present within or adjacent to the auxiliary lane, upstream or downstream of the intersection.
- Sites should not have frequent parking and un-parking movements adjacent to the auxiliary lane.
- Sites should not have a situation where motorists must use the auxiliary lane to enter a side road or other destination downstream.

- Consider the presence of accesses adjacent to the auxiliary lane, low volume residential accesses are unlikely to affect results but avoid accesses with large traffic generation.
- The number of continuous through lanes should be no more than three.

Approaches were analysed individually at intersections with multiple applicable approaches.

Following the identification and selection of sites, the approaches were grouped by the number of continuous through lanes present, with analysis occurring as an aggregate of all approaches and in separate groups to control the potential variability that may result from differing lane numbers.

Two sites were reserved from the analysis as they were used to test the model against other prediction models later in the project.

4.2 Data collection

4.2.1 Auxiliary through lane lengths

The analysis determined the auxiliary lane length for each approach, dividing the length of the lane into three segments:

- the approach length (d_1);
- the intersection length (d_2); and
- the departure length (d_3).

The analysis defined the end-points for the measurements as the stop lines (or projection thereof) and the taper point, i.e. it did not include the length of the taper. As discussed in Chapter 2 Literature Review, the length of the taper is not considered to affect the utilisation, given the layout provides enough distance to merge.

Aerial imagery was used to measure the lengths, recorded to the nearest metre.

The total length, *D*, was computed from these values as well, where:

$$D = d_1 + d_2 + d_3 \tag{4.1}$$

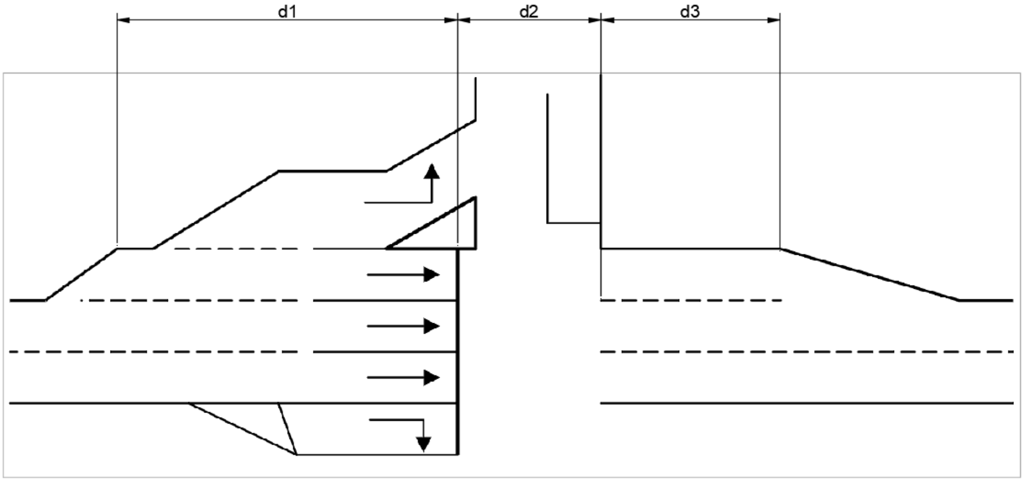


Figure 4-1 Diagram of ATL length dimensions (Karma et al. 2010)

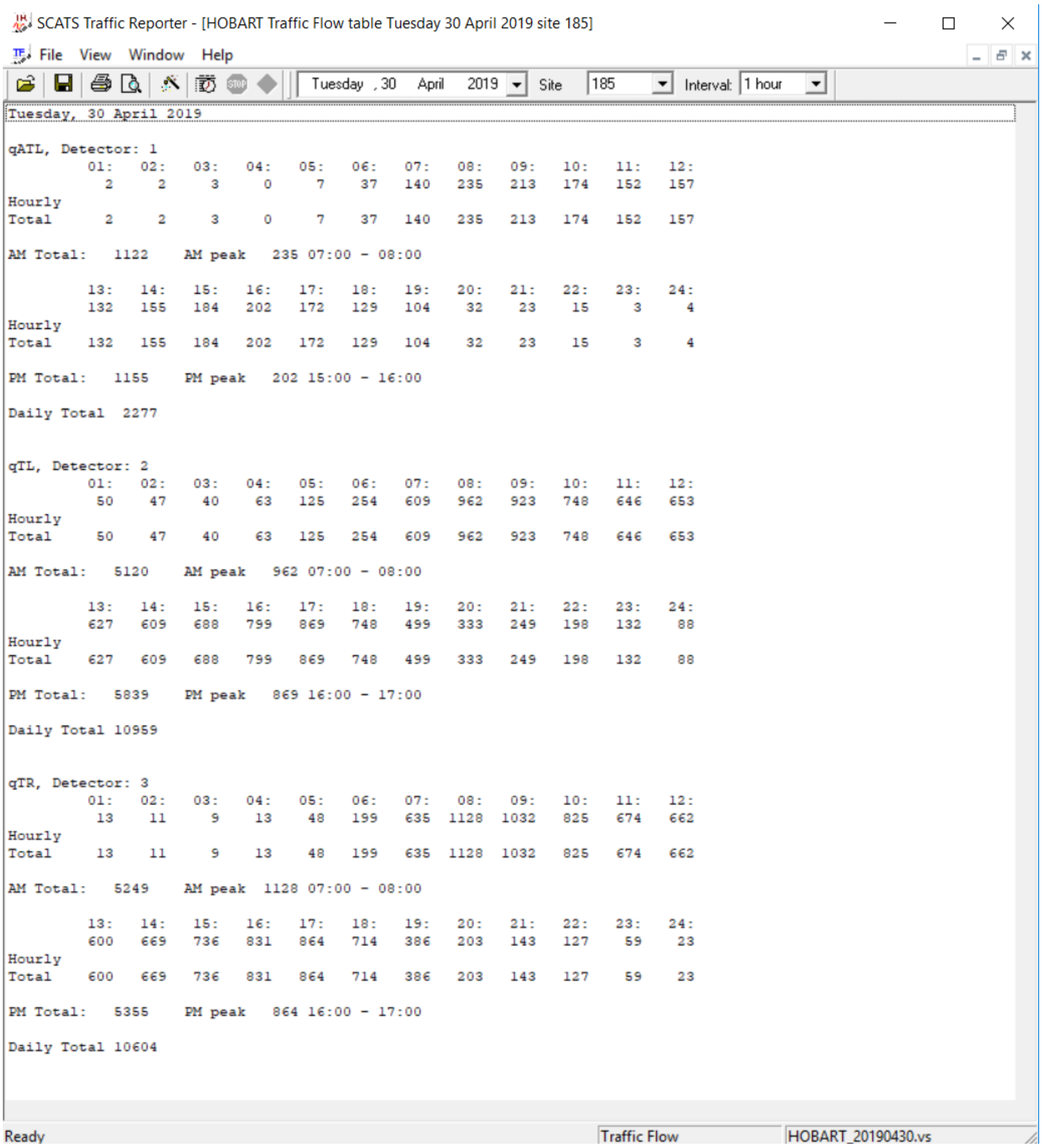
4.2.2 Approach characteristics

The study required the calculation of theoretical lane capacity (Q). For the Austroads (2017) method of calculating this, both lane widths and lateral clearances (i.e. shoulder widths) are required to determine the width factor (f_w); capture of this data used scaled measurements of aerial imagery. The analysis identified a typical lane width and clearance for each approach. Austroads (2017) bunches lane widths into three groups: 2.7 m, 3.2 m and 3.7 m, and also bunches lateral clearance into three groups: 0 m, 1 m, and 2 m. While it is possible to interpolate between these values, given the level of accuracy able to be achieved from measurements of satellite imagery, this study categorises the approaches into these groupings.

Approach grades need to be evaluated to determine the level of impact a heavy vehicle would have in the traffic stream, with the effects measured in terms of equivalent passenger car units (E_{HV}); this combined with the proportion of heavy vehicles (P_{HV}) was used to determine the heavy vehicle factor (f_{HV}) in the capacity calculations. Austroads (2017) categorises grades for this purpose as level, moderate or long sustained. Site imagery was used to determine the magnitude of the slope from visual appearance, with each approach categorised into one of the three groups.

4.2.3 Traffic flow data

The analysis captured the flow rate of vehicles in each continuous through lane and the auxiliary through lane (q_{TL}, q_{TR}, q_{TC} and q_{ATL}). As specified in the site selection criteria, SCATS controls each site, and each lane has an individual vehicle counter. Also from the selection criteria, turning traffic must separate from the lanes before the intersection, this means that the traffic counts extracted from the SCATS Traffic Reporter module will directly report the through movement flows for each lane.




```
Tuesday, 30 April 2019

qATL, Detector: 1
         01:  02:  03:  04:  05:  06:  07:  08:  09:  10:  11:  12:
           2    2    3    0    7   37  140  235  213  174  152  157
Hourly
Total      2    2    3    0    7   37  140  235  213  174  152  157

AM Total:   1122   AM peak   235 07:00 - 08:00

         13:  14:  15:  16:  17:  18:  19:  20:  21:  22:  23:  24:
         132  155  184  202  172  129  104   32   23   15    3    4
Hourly
Total    132  155  184  202  172  129  104   32   23   15    3    4

PM Total:   1155   PM peak   202 15:00 - 16:00

Daily Total  2277


qTL, Detector: 2
         01:  02:  03:  04:  05:  06:  07:  08:  09:  10:  11:  12:
          50   47   40   63  125  254  609  962  923  748  646  653
Hourly
Total     50   47   40   63  125  254  609  962  923  748  646  653

AM Total:   5120   AM peak   962 07:00 - 08:00

         13:  14:  15:  16:  17:  18:  19:  20:  21:  22:  23:  24:
         627  609  688  799  869  748  499  333  249  198  132   88
Hourly
Total    627  609  688  799  869  748  499  333  249  198  132   88

PM Total:   5839   PM peak   869 16:00 - 17:00

Daily Total 10959


qTR, Detector: 3
         01:  02:  03:  04:  05:  06:  07:  08:  09:  10:  11:  12:
          13   11    9   13   48  199  635 1128 1032  825  674  662
Hourly
Total     13   11    9   13   48  199  635 1128 1032  825  674  662

AM Total:   5249   AM peak  1128 07:00 - 08:00

         13:  14:  15:  16:  17:  18:  19:  20:  21:  22:  23:  24:
         600  669  736  831  864  714  386  203  143  127   59   23
Hourly
Total    600  669  736  831  864  714  386  203  143  127   59   23

PM Total:   5355   PM peak   864 16:00 - 17:00

Daily Total 10604
```

Figure 4-2 SCATS Traffic Reporter output (Department of State Growth & Wyminga, 2019)

As the peak hour is the most critical period for intersection capacity, it was the focus of this research. The project undertook analysis for both AM and PM peak hours for each case study site. As intersections (and indeed individual approaches) have differing peak hour times, the peak hour time period was determined individually for each approach based on the peak flow period observed.

The final variable in determining the heavy vehicle factor (f_{HV}) for capacity calculations is the proportion of heavy vehicles in the traffic stream (P_{HV}). While SCATS was the source for traffic flow data, the vehicle detectors used at most intersections are unable to discern the type of vehicle detected — instead, collation of this data used nearby traffic counter sites. The applicable State Road Authorities make this information available on publicly available websites, and this was the source used for this data.

4.2.4 Speed limits

The project used the legal posted speed limit data as a variable in the model. While prevailing traffic speeds will vary, it is related to the posted speed limit. Each approach to the same intersection can have different speed limits, so the speed limit was captured for each approach individually.

The applicable State road authorities upload speed limit information as a spatial dataset; these were the primary source for the speed limit data. Where this information was unavailable or missing, online ‘street view’ imagery was used to locate speed limit signs to determine the applicable speed limit.

4.2.5 Traffic signal cycle and phase timing data

The project identified the following variables for consideration in the analysis:

- Cycle Time (c)
- Through Movement Green Time (G)
- Through Movement Green Phase Split (G/c)

The assumption that start loss and end gain can be considered equal, presented in section 2.2, was assumed here:

$$g = G - \text{start loss} + \text{end gain} \tag{2.1}$$

Thus,

$$g = G$$

This information is also required to calculate the capacity (Q) of each approach and thus, the movement degree of saturation (X).

As discussed in section 3.1, the timing at a signalised intersection varies with the prevailing traffic conditions; this can be as often as every cycle. Timing parameters for AM and PM peaks were captured separately, as there are typically different timing parameters for each period.

The analysis required selection of a typical representation of the cycle time and green time/ phase splits for each approach; these values are not constant throughout the hour, but the project is dealing with a single hour resolution.

The Strategic Monitor module within the SCATS Traffic Reporter was used to view historical timing data to understand the typical timing arrangements; this included review of the traffic signal configuration plan and specific reference to the phase diagram.

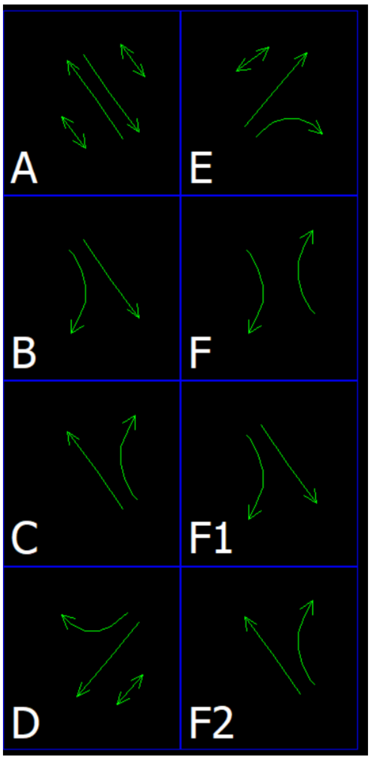


Figure 4-3 SCATS Phase diagram (Department of State Growth & Wyminga, 2019)

The capture of the timing arrangements was for the same period as the traffic flow data for each approach, and where possible were from the same day.

```
Tuesday 30-April-2019 08:38 SS  47M+ PL 4.4# PVa18.4 CT  180 +0 RL183, SA 223 DS 107
 Int    SA/LK     PH  PT!  DS  VO  VK!  DS  VO  VK!  DS  VO  VK!  DS  VO  VK! ADS
  185 S 230  '     1 103!  19   7   8!  97  51  53!  82  46  47!  -        -!   96
  185 S 231  '     2  99!  12   5   5!  61  29  30!  54  27  28!  -        -!   64
  185 S 232 ^'     3  20> 107   8  10> 125  10  12!  -        -!  -        -!   91
  185 S 233 ^      4  17!  54   4   4!  -        -!  -        -!  -        -!   38
  185 S 234 ^'     D  19!  98   7   7> 111   9   9!  -        -!  -        ->  111
  185 S 235  '     E  39!  73  11  13!  33   4   4!  63  10  11!  -        -!   81
Tuesday 30-April-2019 08:41 SS  47M+ PL 1.4# PVa15.4 CT  180 +0 RL184, SA 223 DS 105
 Int    SA/LK     PH  PT!  DS  VO  VK!  DS  VO  VK!  DS  VO  VK!  DS  VO  VK! ADS
  185 S 230  '     1 118!  40  18  19!  95  51  59!  88  52  58!  -        -!   96
  185 S 231  '     2  95!  22   8   9!  55  23  26!  62  29  31!  -        -!   62
  185 S 232 ^'     3  22> 111  12  12> 116  11  12!  -        -!  -        ->  110
  185 S 233 ^      4   0!   0   0   0!  -        -!  -        -!  -        -!   18
  185 S 234 ^'     D  25!  86   8   8!  73   6   8!  -        -!  -        ->  107
  185 S 235  '     E  35! 100  13  16!   0   0   0> 119  15  19!  -        -!   97
Tuesday 30-April-2019 08:44 SS  47M+ PL 1.4# PVa11.4 CT  180 +0 RL175, SA 223 DS 107
 Int    SA/LK     PH  PT!  DS  VO  VK!  DS  VO  VK!  DS  VO  VK!  DS  VO  VK! ADS
  185 S 230  '     1  98!  26  10  10!  84  43  44!  74  40  41!  -        -!   90
  185 S 231  '     2  92!  82   8  31!  72  31  33!  72  33  35!  -        -!   71
  185 S 232 ^'     3  24!  82   9  10!  94   9  11!  -        -!  -        ->  108
  185 S 233 ^      4  16!  45   4   3!  -        -!  -        -!  -        -!   32
  185 S 234 ^'     D  29!  73   8   8!  69   7   9!  -        -!  -        -!   86
  185 S 235  '     E  29!  95  13  13!  44   4   4> 102  12  13!  -        ->  101
Tuesday 30-April-2019 08:47 SS  47M+ PL 2.4# PVa35.4 CT  180 +0 RL181, SA 223 DS 108
 Int    SA/LK     PH  PT!  DS  VO  VK!  DS  VO  VK!  DS  VO  VK!  DS  VO  VK! ADS
  185 S 230  '     1 102!   0   0   0!  88  43  47!  98  51  56!  -        -!   93
  185 S 231  '     2  91!  20   7   7!  57  20  26!  66  31  32!  -        -!   70
  185 S 232 ^'     3  26!  72   8   9!  71   7   9!  -        -!  -        -!   89
  185 S 233 ^      4  15!  20   2   1!  -        -!  -        -!  -        -!   24
  185 S 234 ^'     D  36!  52   8   7!  76  11  12!  -        -!  -        -!   77
  185 S 235  '     E  24> 113  11  12!  57   4   4> 101   9  11!  -        ->  111
```

Figure 4-4 SCATS Strategic Monitor output (Department of State Growth & Wyminga, 2019)

4.3 Data analysis

4.3.1 Determination of observed lane utilisation

The dependent variable in this study is the utilisation ratio of the auxiliary through lane. The utilisation ratio for the case study sites needed to be calculated to relate to other site variables in the analysis. The utilisation ratio of the subject lane (as identified in section 2.10) is calculated by dividing the degree of saturation of the subject lane by that of the critical lane:

$$p_i \;=\; X_i \;/\; X \;=\; (q_1\, c_1 \;/\; s_1\, g_1) \;/\; (q\, c \;/\; s\, g)$$

This equation can be simplified when considering lanes involving the same movement at the same approach, resulting in equation 2.14 (see section 2.10 for the derivation):

$$p_i \;=\; q_1 \;/\; q \qquad (2.14)$$

Simplification of the equation means that the only variables required to determine the utilisation ratio for the auxiliary lane (p_{ATL}) are the flow rate in the auxiliary lane (q_{ATL}) and the flow rate in the critical lane (q_c).

The critical lane is the lane having the highest flow rate in the traffic flow data. The study only calculated the utilisation of the auxiliary through lane.

The flow rates at approaches vary with time over a day. As such, utilisation was calculated for each auxiliary through lane for their respective AM and PM peak hour periods.

4.3.2 Analysis of length parameters

The first objective of this research was to confirm that a strong relationship exists between the length of auxiliary lanes and their utilisation, as suggested by previous research (see Chapter 2).

Section 4.2.1 identified three length variables, the relationship between lane utilisation and these length variables, as well as a combination of these length variables, were analysed:

- Approach Length (d_1)
- Departure Length (d_3)
- Approach Length + Departure Length ($d_1 + d_3$)
- Approach Length + Intersection Width ($d_1 + d_2$)
- Departure Length + Intersection Width ($d_3 + d_2$)
- Approach Length + Departure Length + Intersection Width ($D = d_1 + d_3 + d_2$)

While there are two periods, the AM and PM peak periods, the analysis took the average utilisation across the two periods as the length variable is constant for both. Approaches were also analysed as an aggregate, as well as in their one, two and three continuous lane groupings.

A scatter plot was produced for each variable (or set of variables) against the lane utilisation to provide a visual representation of the relationship.

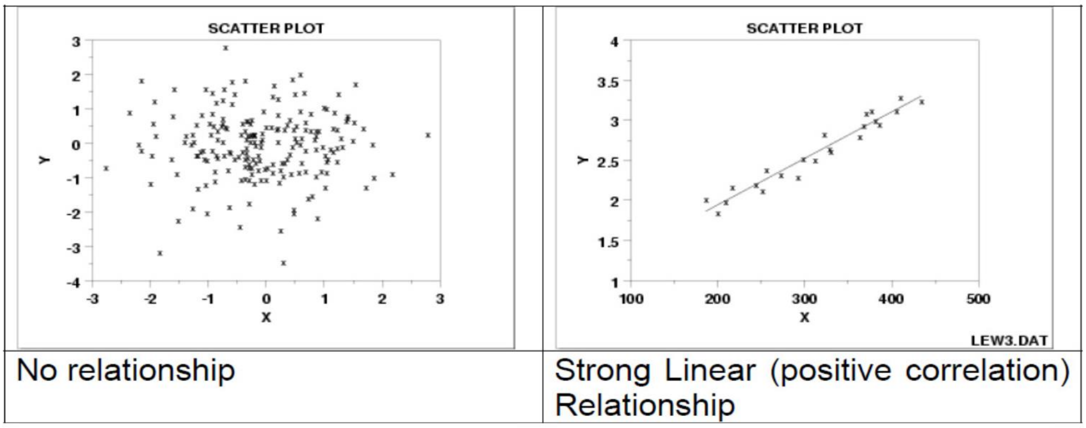


Figure 4-5 Scatter plot relationship (Karma et al. 2010)

A line of best fit was generated using the linear regression/ least-squares method to define this relationship, and an R-squared value created to provide a measure of the goodness of fit of this relationship.

The visual plot was considered alongside the R-squared value to determine if these relationships are strong.

4.3.3 Analysis of traffic signal timing parameters

Previous research (see Chapter 2) also identified a correlation between lane utilisation and the green time parameters at traffic signals. While length is the primary variable investigated in this study; these variables were also considered alongside as they may provide some explanation of model variance that might not be explained by length alone.

In particular, the analysis focused on the following traffic signal timing parameters:

- Cycle Time (c)
- Through Movement Green Time (G)
- Green Time / Cycle Time Split (g/c)

The start-loss was assumed to be equal to the end-gain, as discussed in section 2.2. Then from equation 2.1, g is equivalent to G:

$$g = G - \text{start loss} + \text{end gain} \tag{2.1}$$

Analysis of these three variables followed the same process as the length variables discussed in section 4.3.2; generation of a scatter plot, a line of best fit and an R-square value to determine the goodness of fit. As there are two periods with differing traffic signal timing parameters, the AM and PM periods were analysed separately for each approach. Approaches were also analysed as an aggregate, as well as in their one, two and three continuous lane groupings.

The plots and R-squared values were considered to determine if the relationship is compelling.

4.3.4 Analysis of the degree of saturation

As with the traffic signal timing variables in section 4.3.3, the movement degree of saturation (X) was considered alongside the primary length variables, as they may be able to explain some model variance that is unexplained by the length variables alone.

Firstly, the analysis required the calculation of the degree of saturation for the through movement (X_T). Section 2.3 discussed the degree of saturation, with the following equation given:

$$X = q \;/\; Q = q\, c \;/\; s\, g \tag{2.6}$$

It defines the degree of saturation (X) as the ratio of the arrival flow rate (q) to the capacity (Q).

The arrival flow rate for the through movement (q_T) is simply the sum of the through vehicle movements in all through lanes (q_{TL}, q_{TR}, q_{TC} and q_{ATL}) with this data captured in section 4.2.3.

The determination of the actual on-site lane capacity can be a complex task and is outside the scope of this study. Instead, the theoretical lane capacity was determined. The capacity was calculated by multiplying the saturation flow rate by the through movement green phase split. This study used the Austroads (2017) method presented in section 2.5 to calculate the capacity of a single uninterrupted lane, equivalent to the saturation flow rate:

$$Q = 2400\, f_w\, f_{HV} \tag{2.11}$$

The lane widths and lateral clearances for each approach were compared with Table 2-1 to determine the width factor (f_w) for that approach, and the approach grades and heavy vehicle proportions for each approach were compared with Table 2-2 to determine the heavy vehicle factor (f_{HV}) for that approach.

Then using equation 2.11 the through movement capacity for each lane was calculated. The overall capacity for the through movement (Q) at each approach is the product of the lane capacity and the number of through lanes (continuous lanes and auxiliary lane).

All the information was then in a form that allowed calculation of the degree of saturation for the through movement at each approach.

Using the same technique as discussed in sections 4.3.2 and 4.3.3, the analysis compared the degree of saturation and the auxiliary lane utilisation. As there are two periods, the AM and PM periods were analysed separately for each approach. Approaches were also examined as an aggregate, as well as in their one, two and three continuous lane groupings.

Again, the relationship was reviewed to determine the strength of it.

4.3.5 Speed limit analysis

Previous research into the effect of speed limits on lane-changing behaviour are conflicting, and it is unclear how applicable this research is to lane changing behaviour at a signalised intersection, including lane changes into an auxiliary through lane.

This study included the approach speed limit as a variable that may provide some explanation into the variance of lane utilisation.

As with other variables, the analysis produced a scatter plot relating lane utilisation and the speed limit for each approach; this provided a visual representation of the relationship. The R-square value was also determined; both were reviewed to conclude the goodness of fit of this relationship.

4.3.6 Multiple linear regression analysis

After reviewing all the variables individually, multiple linear regressions methods were used to analyse the relationship between auxiliary lane utilisation and the variables combined. The project produced an equation of the following form:

$$p_{ATL} = a + \propto_1 b + \propto_2 c \ldots + \propto_n z$$

where:

p_{ATL}	=	Auxiliary Through Lane Utilisation Ratio
a	=	Constant
$\propto_{1,} \propto_{2,} \propto_n$	=	Variable of Coefficient
b, c, z	=	Explanatory Variables

A correlation matrix was also developed to refine the model further. The matrix shows the correlation between all the variables. The matrix was used to identify those variables that were highly correlated, enabling further analysis and removal from the model if applicable. The project defined that variables are correlated when the correlation value exceeds 0.5.

As there are two periods, the AM and PM periods were analysed separately for each approach. Analysis of approaches was undertaken as an aggregate, as well as in their one, two and three continuous lane groupings.

An R-squared value was produced to determine the goodness of fit of the relationship; however, the ‘adjusted’ R-squared value was used instead to account for the fact that the relationship involves multiple variables. A plot was produced to compare the predicted results made using the relationship and the observed results. Both results were considered to determine the strength of the relationship.

4.3.7 Selection of final model

In considering the relationships for the individual variables, the correlation matrix and the multiple variable analysis conducted in sections 4.3.2 to 4.3.6, the project selected the relationship with the most robust fit as the final mathematical model for this research.

4.4 Model comparison

4.4.1 Predict utilisation using study model

The model selected in section 4.3.7 was used to estimate auxiliary through lane utilisation at two sites to test its applicability. The sites used in this stage were different from those used to develop the model, as mentioned in section 4.1, with the data for these sites captured alongside the case study approaches.

4.4.2 Predict utilisation using comparison models

The short auxiliary lane utilisation at these sites was also predicted using the following methods discovered during the literature review:

- Australian Road Capacity Guide method (see section 2.11.1)
- Australian Road Research method (see section 2.11.2)
- SIDRA method (see section 2.11.3)
- Transport Research Board method (see section 2.11.5)

4.4.3 Determine observed utilisation

The observed lane utilisation was determined at the comparison test sites using the same method described in section 4.3.1.

4.4.4 Model results comparison

The analysis undertook an assessment of the observed utilisation and the predicted utilisations generated by each of the models. The actual and relative difference was calculated, with the average absolute difference used as a measure of error for each model.

While this is only a small number of test sites for comparison, it will enable some discussion on the applicability of the model developed within this study.

5. Results

5.1 Selection of case study sites

The project sought intersections across multiple States to get a sufficiently large sample size. In the end, site selection incorporated intersections from Tasmania, Victoria and Western Australia. The first pass identified a large number of sites; however, application of the site selection criteria produced in section 4.1 reduced the size of the sample significantly. The final list of case study sites selected for this study, as well as their site ID, is below:

Tasmania

T1 – Brooker Highway/ Lampton Avenue, Derwent Park.

T2 – Brooker Highway/ Derwent Park Road, Derwent Park.

T3 – Brooker Highway/ Bowen Road, Ashbolt Crescent, Lutana.

T4 – Brooker Highway/ Risdon Road, Lutana.

T5 – Hobart Road, Kings Meadows Connector, Kings Meadows.

T6 – Tarleton Street/ Wright Street, East Devonport.

Victoria

V1 – North Road/ Warrigal Road, Oakleigh.

V2 – Warrigal Road/ South Road, Oakleigh South.

V3 – Springvale Road/ Governor Road/ Hutton Road, Keysborough.

V4 – South Gippsland Highway/ Camms Road, Cranbourne.

V5 – Springvale Road/ Burwood Highway, Burwood East.

V6 – Boronia Road/ Scoresby Road, Boronia.

V7 – Canterbury Road/ Dorset Road, Bayswater North.

V8 – Mt Dandenong Road/ Dorset Road, Croydon.

V9 – Melton Highway/ Calder Park Drive, Sydenham.

V10 – Princes Highway/ Geelong Street, Footscray.

V11 – Heaths Road/ Derrimut Road, Hoppers Crossing.

V12 – Princes Highway/ Murrumbeena Road, Murrumbeena.

V13 – Princes Highway/ Warrigal Road, Oakleigh.

V14 – Princes Highway/ Wellington Road, Clayton.

V15 – Princes Highway/ Gladstone Road, Dandenong.

V16 – Princes Highway/ Old Geelong Road, Laverton.

Western Australia

W1 – Reid Highway/ Lord Street/ Daviot Road, Beechboro.

W2 – Reid Highway/ West Swan Road, Caversham.

W3 – Roe Highway/ Toodyay Road, Stratton.

W4 – Lloyd Street/ Clayton Street, Midland.

W5 – Albany Highway/ Royal Street, Kenwick.

W6 – South Western Highway/ Thomas Road, Byford.

W7 – Armadale Road/ Railway Avenue, Armadale.

W8 – Cockburn Road/ Orsino Boulevard, Coogee.

W9 – Tonkin Highway/ Thomas Road, Oakford.

The analysis includes a total of 31 sites for review, with 57 applicable approaches. Categorising sites by their number of approach lanes gives 20 single-lane approach, 24 two-lane approach and 13 three-lane approach sites. Attempts were made to increase the number of three full-length through lane approaches for analyses, but such large signalised intersections that fit the selection criteria are not overly frequent.

The project reserved the western approach at site V2 and southern approach at site V9 from the case study pool for inclusion in the model comparison as proposed in section 4.1.

5.2 Data collection

5.2.1 Auxiliary through lane lengths

The analysis determined three length variables (d_1, d_2 & d_3, as defined in section 4.2.1) through measurement of aerial imagery (to the nearest meter), with the total length D calculated from the sum of these elements. Table B-1 presents this data.

5.2.2 Approach characteristics

Typical lane widths, lateral clearances and approach grades were determined using aerial and street view imagery rounded to the nearest values presented in Table 2-1 and Table 2-2. Table B-2 presents this data.

5.2.3 Traffic flow data

The project captured traffic flow data for the 30th April 2019; this day was representative of traffic flows unaffected by weekends, public holidays or school holidays.

Traffic signal site plans and configuration sheets were required to determine the loop number applicable to each respective lane at a site to enable extraction of traffic flow data; the project sourced this information from the Department of State Growth and Wyminga (2019), VicRoads (2019b) & Main Roads Western Australia and Saunders (2019).

Department of State Growth and Wyminga (2019), VicRoads (2019a), Main Roads Western Australia and Saunders (2019) and VicRoads and Lee (2019) supplied the SCATS traffic detector counts for this study.

Collection of heavy vehicle flow proportions used the Department of State Growth (2019b), Main Roads Western Australia (2019a) and VicRoads (2019c) databases.

Table B-3 presents the traffic flows for each lane and the proportion of heavy vehicles in the traffic stream.

5.2.4 Speed limits

The Department of State Growth (2019a), Main Roads Western Australia (2019b) and VicRoads (2019d) databases provided the speed limit data. These datasets are spatially based with the speed limit directly read off the map using a legend, or by clicking on the road centreline to bring up the properties, depending on the source. Table B-4 tabulates this data.

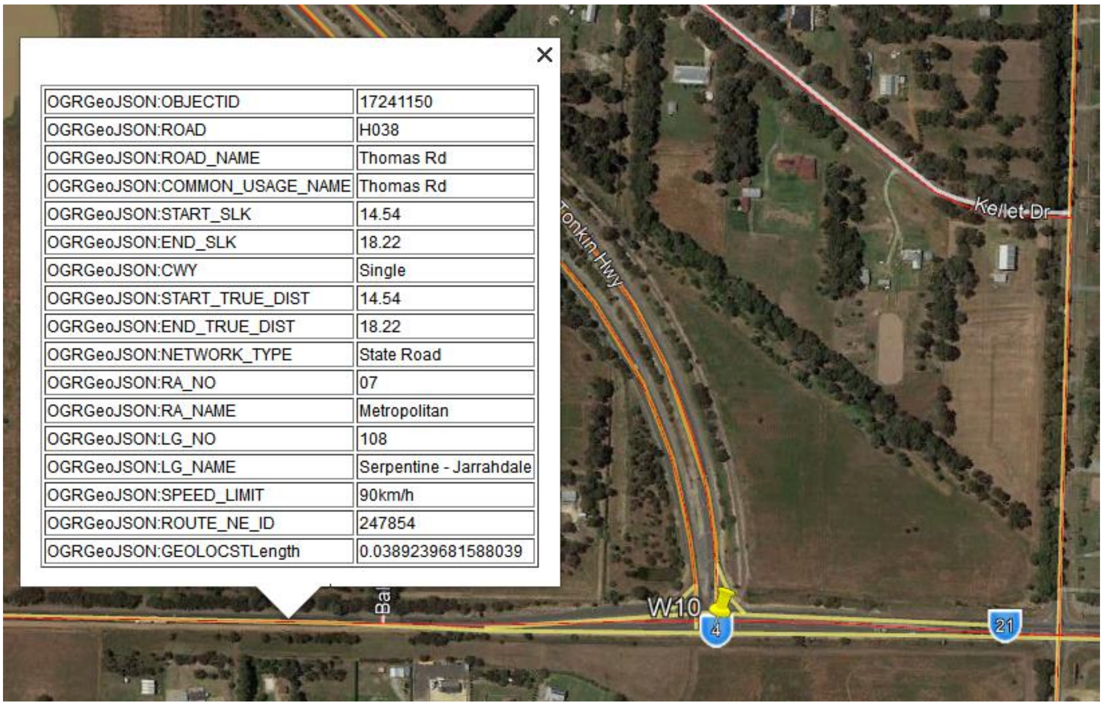


Figure 5-1 Speed limit dataset (Main Roads Western Australia 2019)

5.2.5 Traffic signal cycle and phase timing data

Extraction of this data required the review of the configuration sheet for each signalised intersection to determine which phase(s) applied to the through movements in question; the Department of State Growth and Wyminga (2019), VicRoads (2019b) & Main Roads Western Australia and Saunders (2019) supplied these.

The Department of State Growth and Wyminga (2019), Main Roads Western Australia and Saunders (2019), VicRoads and Lee (2019) and VicRoads (2019e) also supplied the SCATS Strategic Monitor data required to determine the timing settings used on the date and time in question.

Unfortunately, data was unavailable for sites T5N and W9W; the project removed these sites from further analysis.

Due to the nature of the SCATS system, cycle times and phase times are variable. The analysis selected a typical cycle times and phase time for each approach based on a review of times used over the peak period; Table B-5 presents this information.

5.3 Data analysis

5.3.1 Determination of observed lane utilisation

Calculation of the utilisation of each test case ATLs followed the proposed methodology given in section 4.3.1; Table C-3 presents these results.

5.3.2 Analysis of length parameters

Figure 5-2 through to Figure 5-13 presents the relationship for the length variables, and combinations of length variables, to the ATL utilisation in scatter plots. The first graph for each variable shows the association for all sites combined, while the second graph groups the sites by the number of CTLs present.

Approach length (d_1)

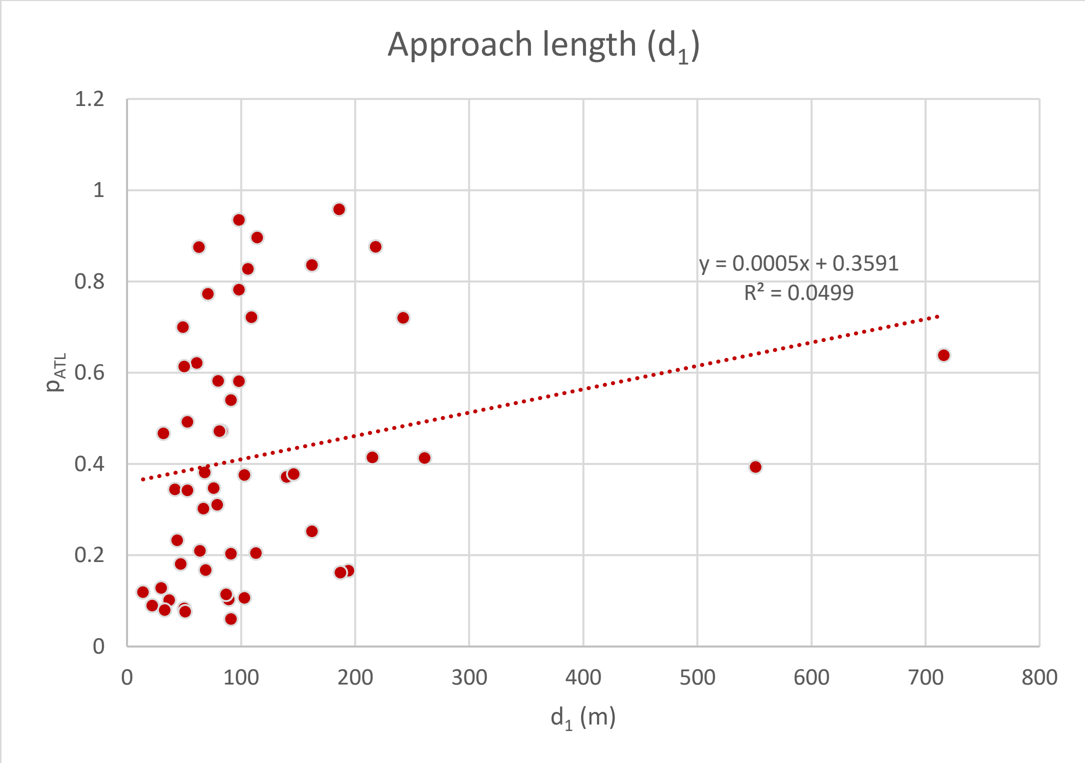


Figure 5-2 Approach length scatter plot

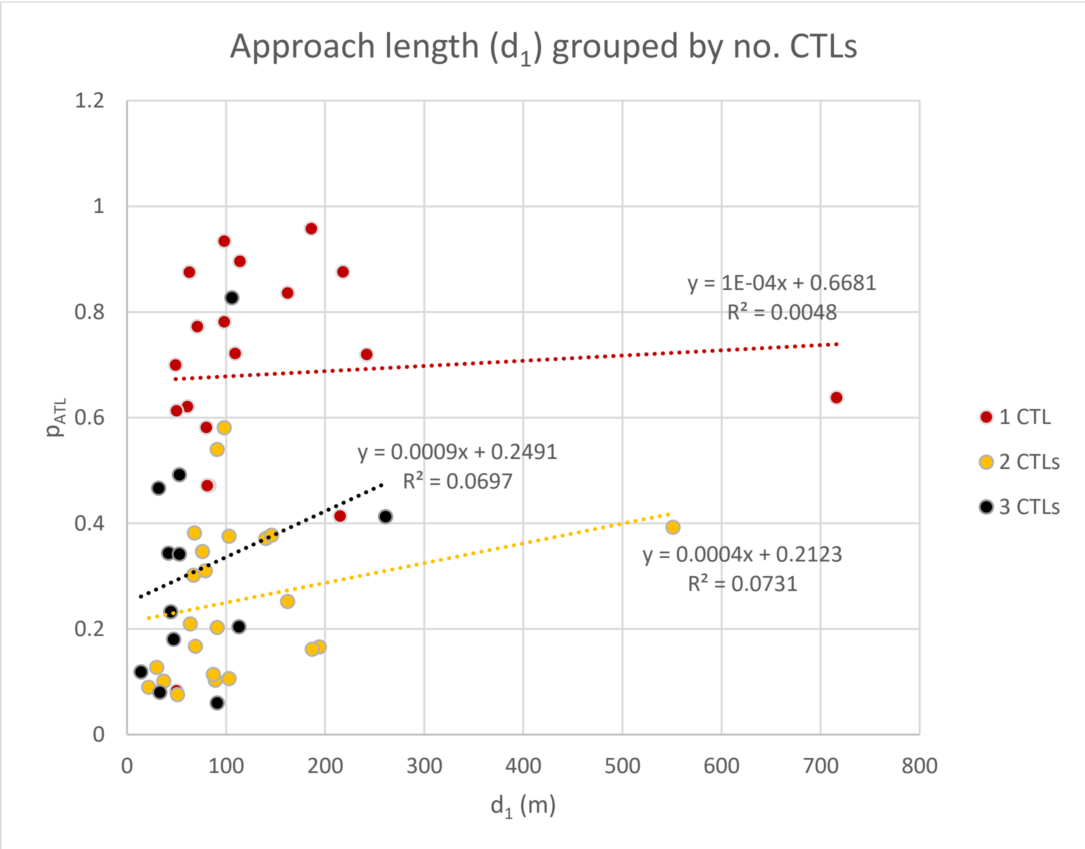


Figure 5-3 Approach length (by CTLs) scatter plot

Departure length (d_3)

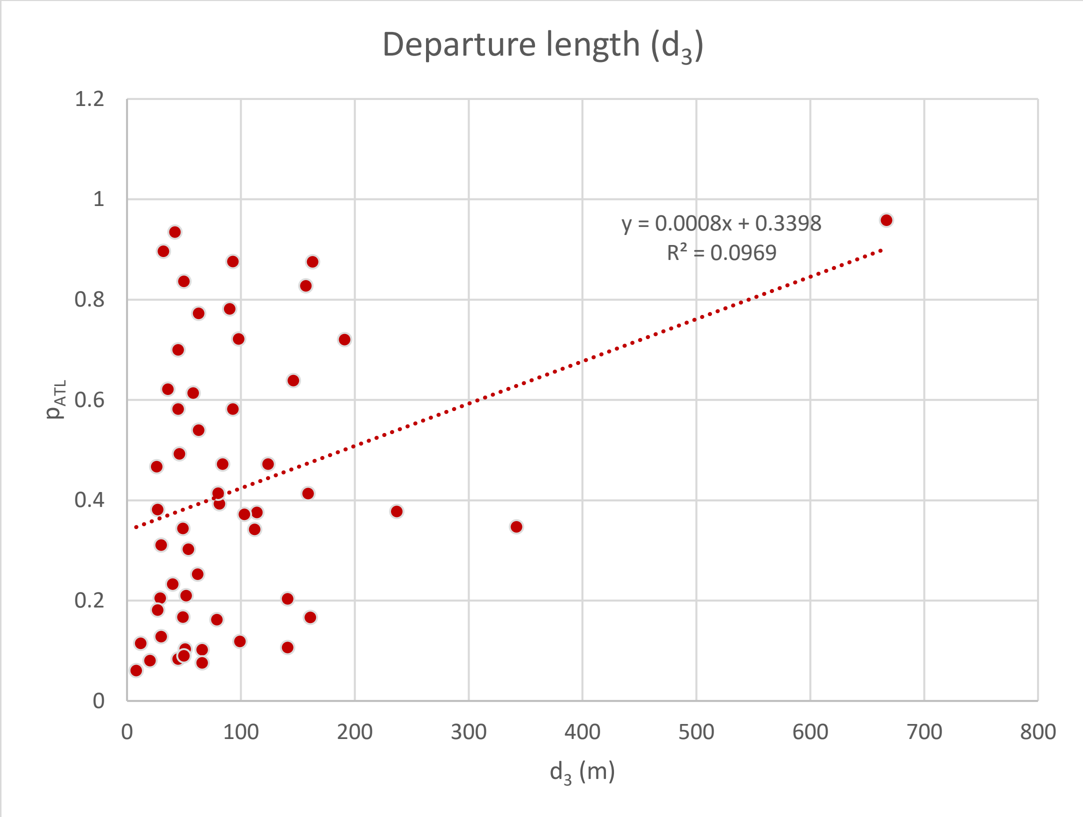


Figure 5-4 Departure length scatter plot

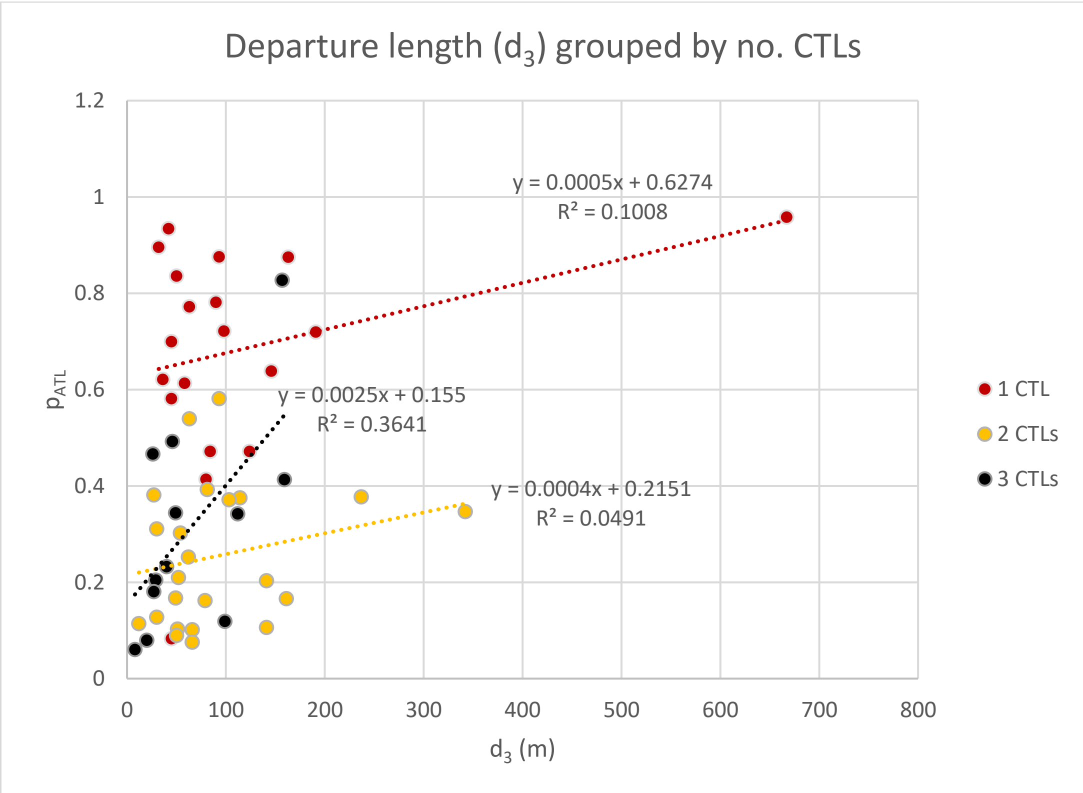


Figure 5-5 Departure length (by CTLs) scatter plot

Approach length + departure length ($d_1 + d_3$)

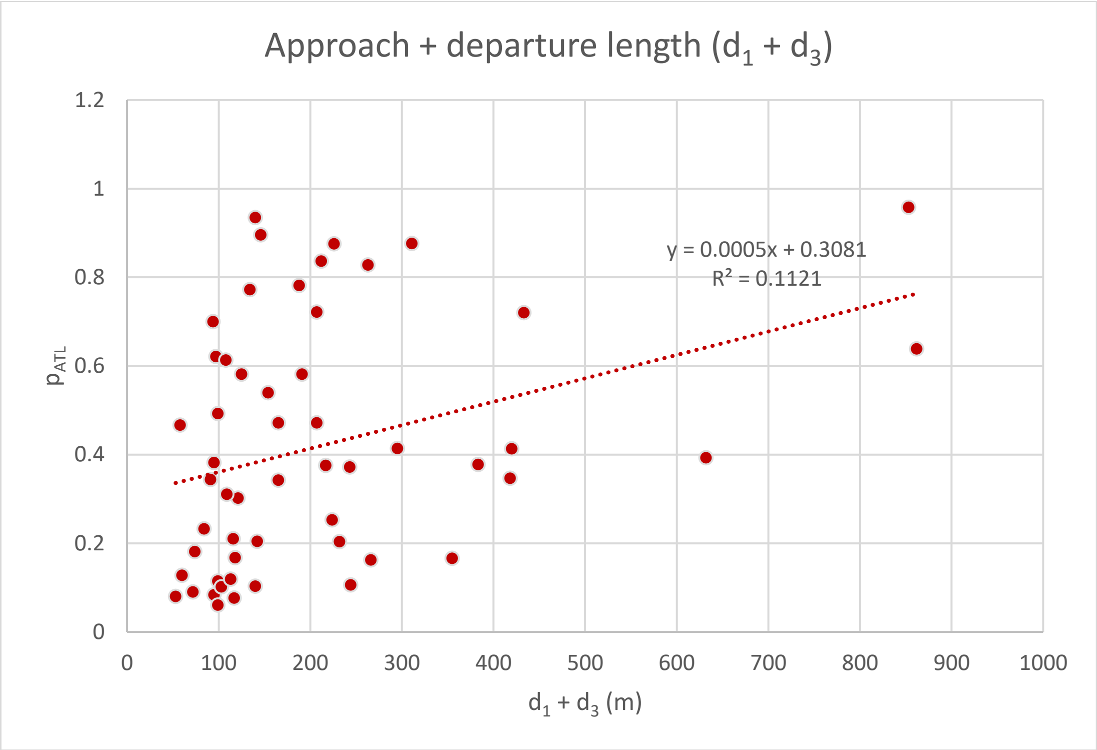


Figure 5-6 Approach & departure length scatter plot

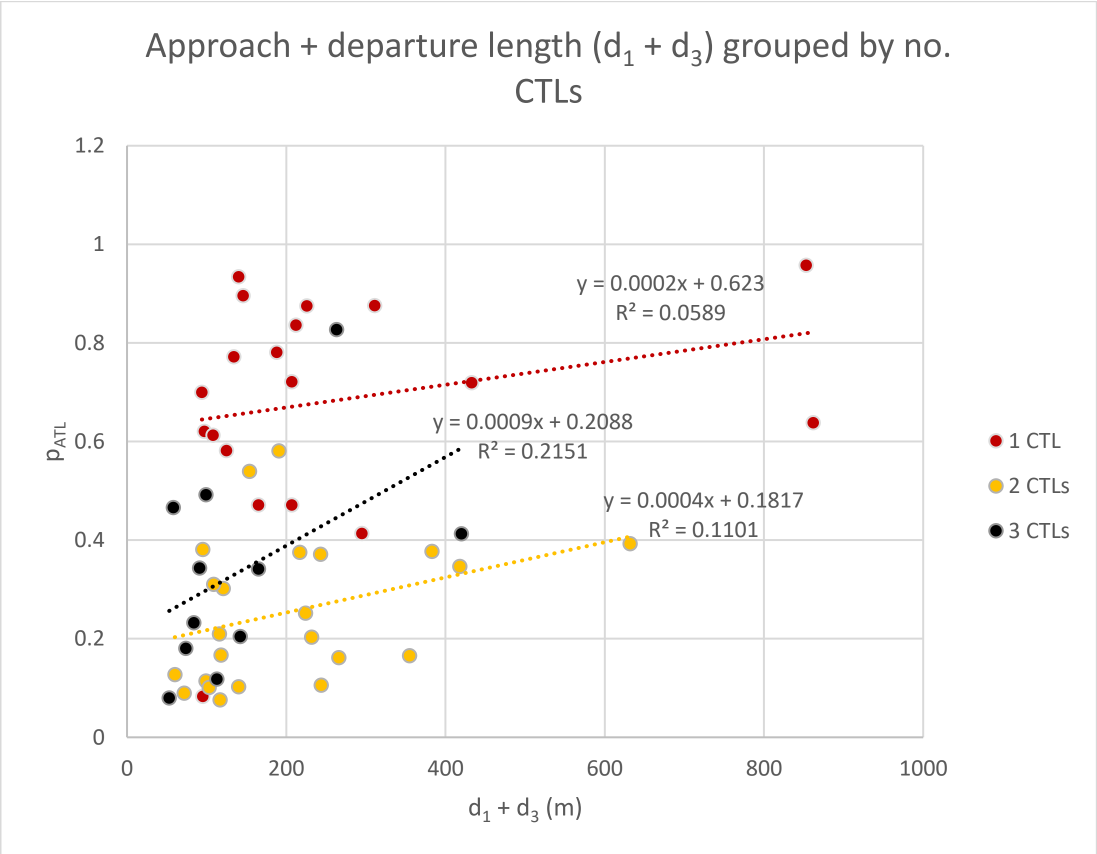


Figure 5-7 Approach & departure length (by CTLs) scatter plot

Approach length + intersection width (d_1 + d_2)

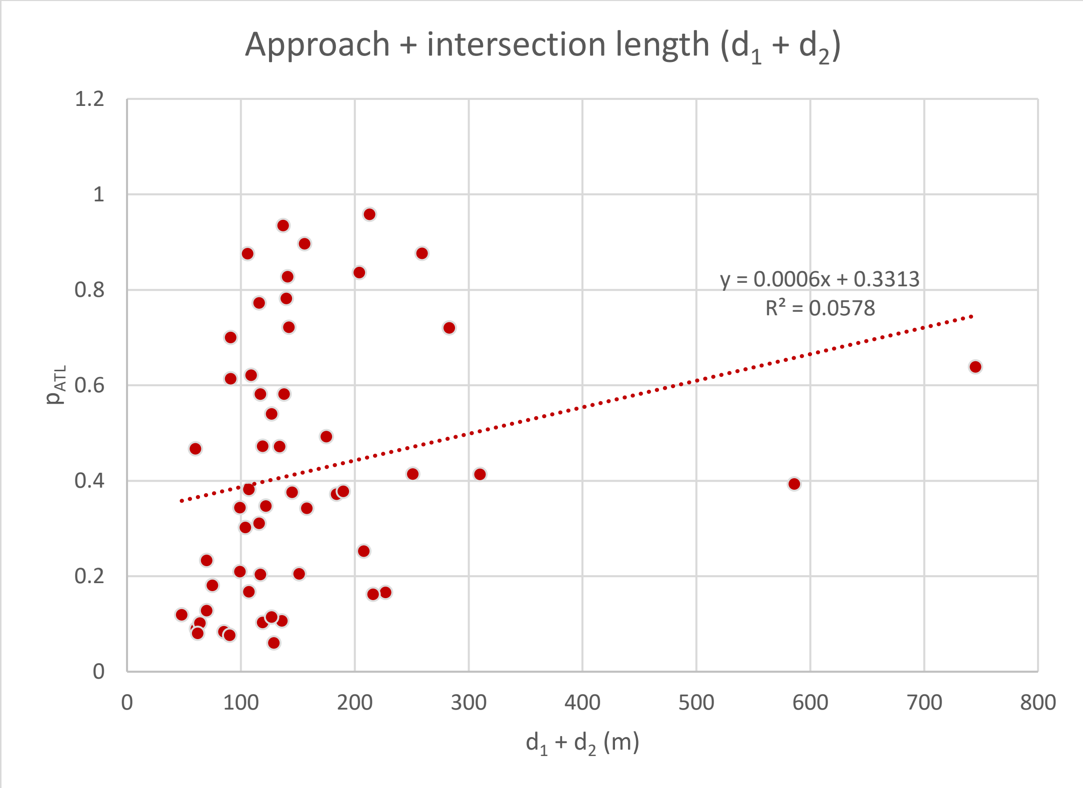


Figure 5-8 Approach & intersection length scatter plot

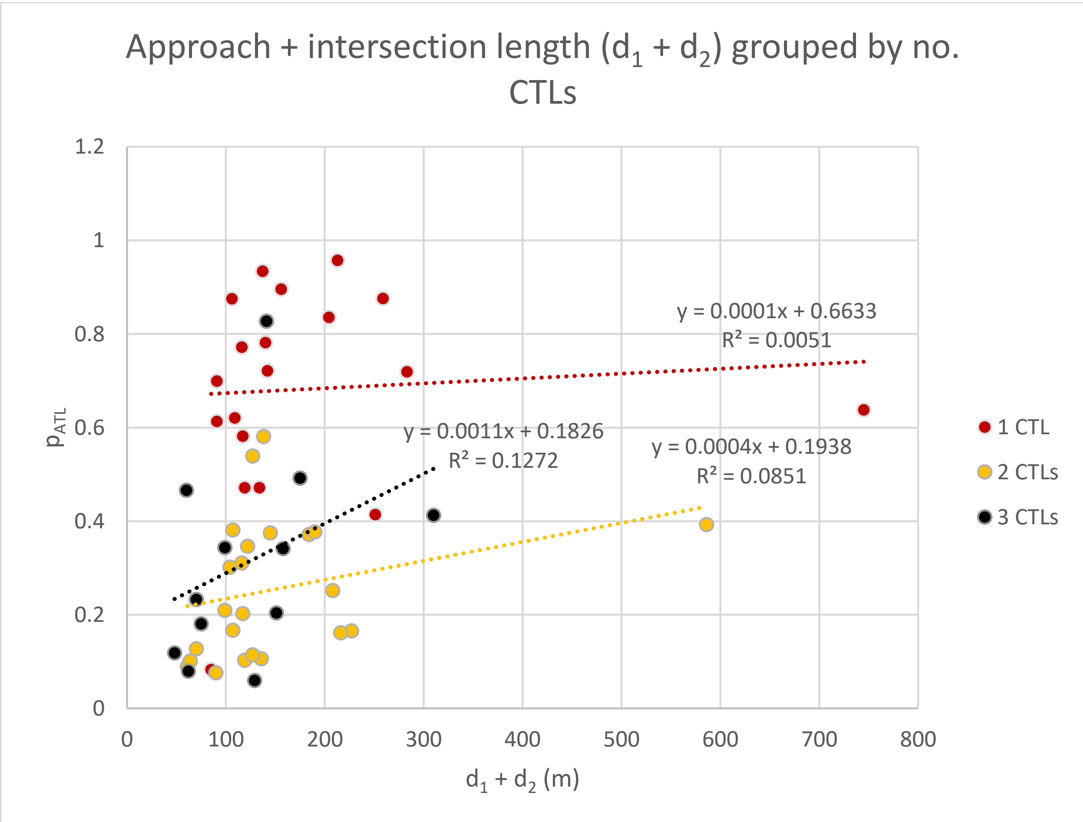


Figure 5-9 Approach & intersection length (by CTLs) scatter plot

Departure length + intersection width (d_3 + d_2)

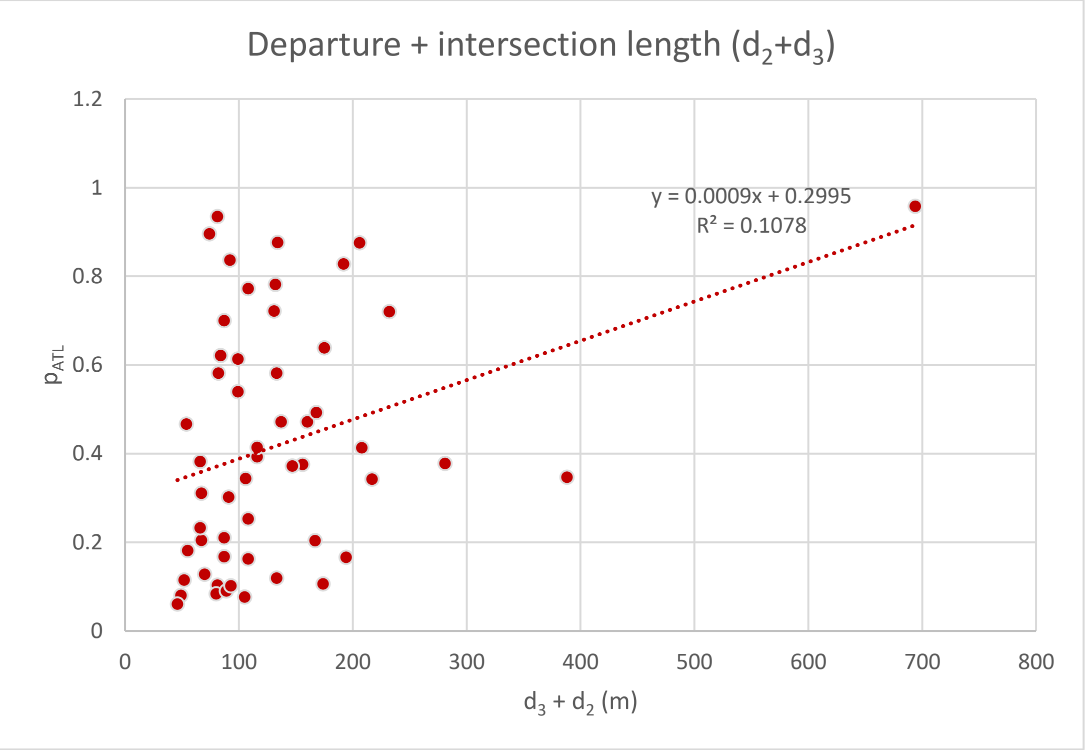


Figure 5-10 Departure & intersection length scatter plot

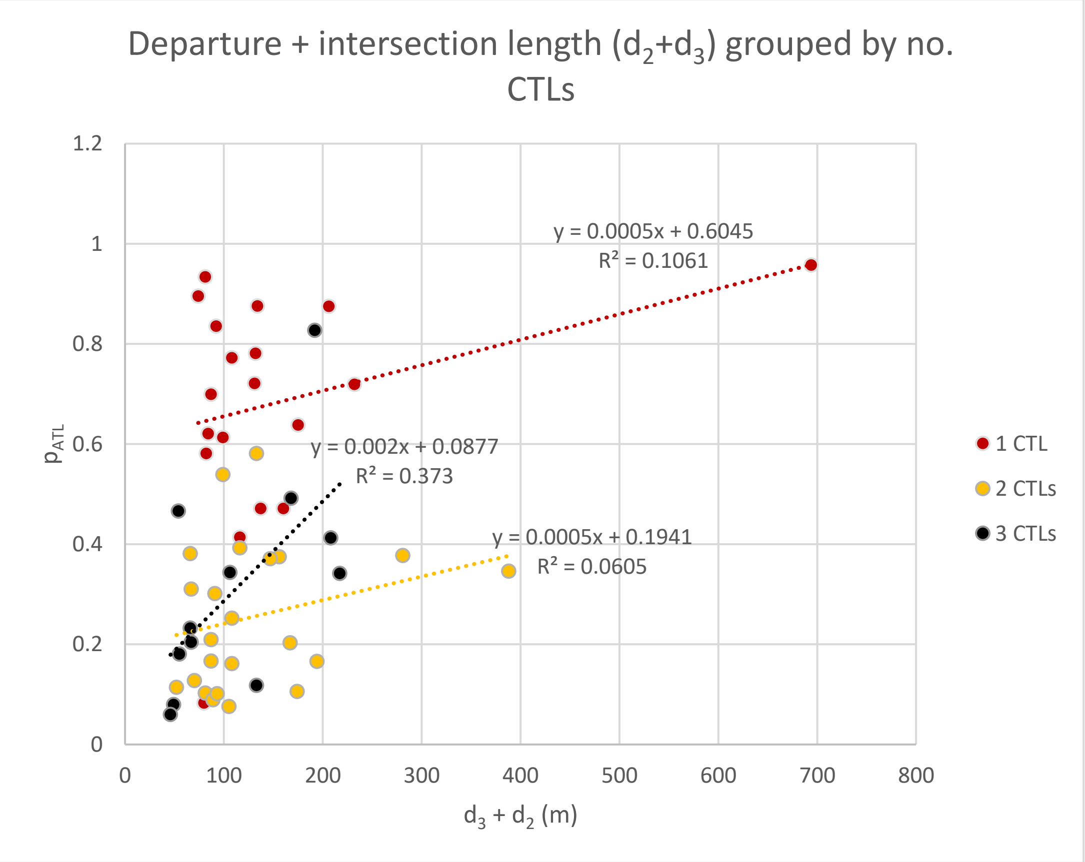


Figure 5-11 Departure & intersection length (by CTLs) scatter plot

Approach length + departure Length + intersection Width ($D = d_1 + d_3 + d_2$)

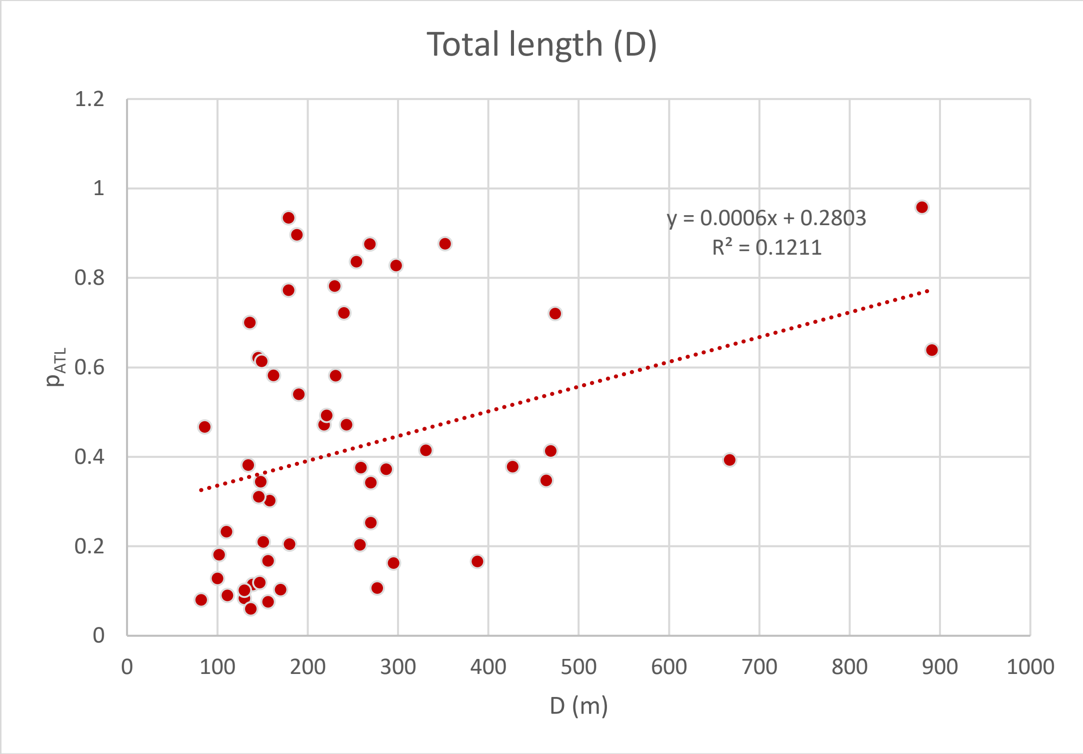


Figure 5-12 Approach, departure & intersection length scatter plot

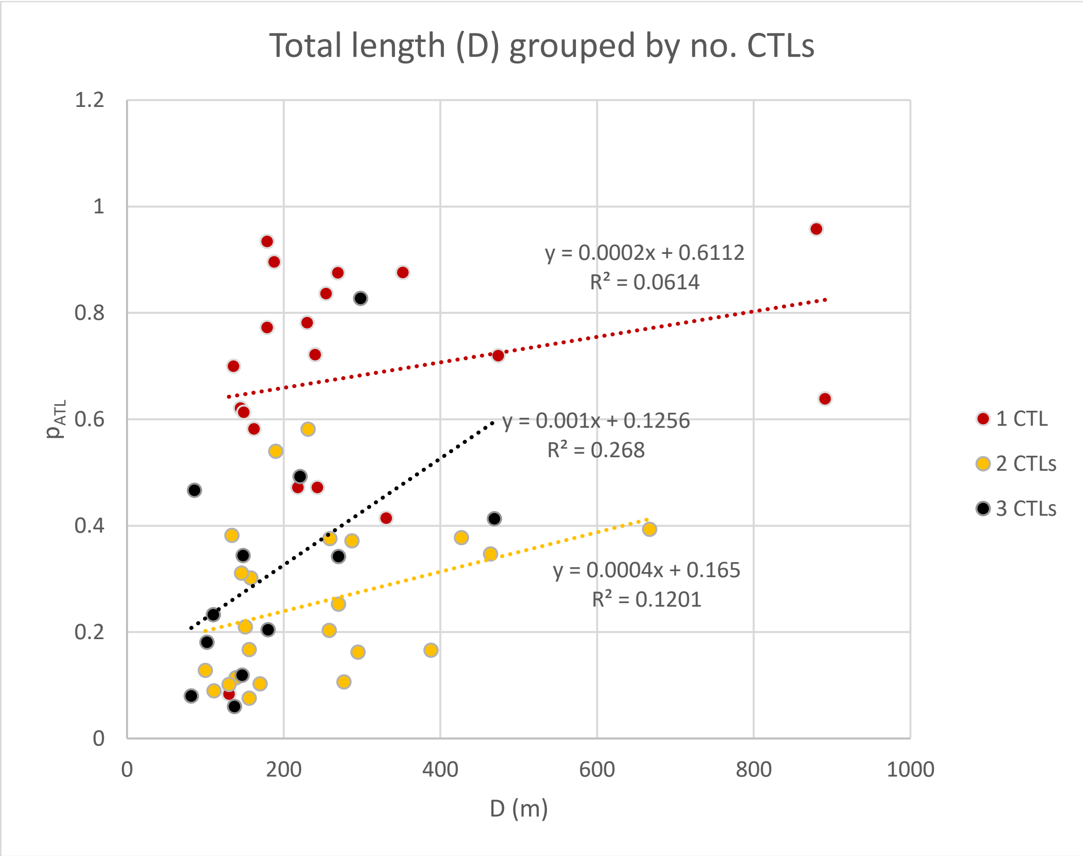


Figure 5-13 Approach, departure & intersection length (by CTLs) scatter plot

5.3.3 Analysis of traffic signal timing parameters

The relationship for the three traffic signal timing parameters has been presented in Figure 5-14 to Figure 5-19. The first graph shows the relationship with all sites combined, the second shows the relationships with the sites grouped by the number of CTLs.

Cycle time (*c*)

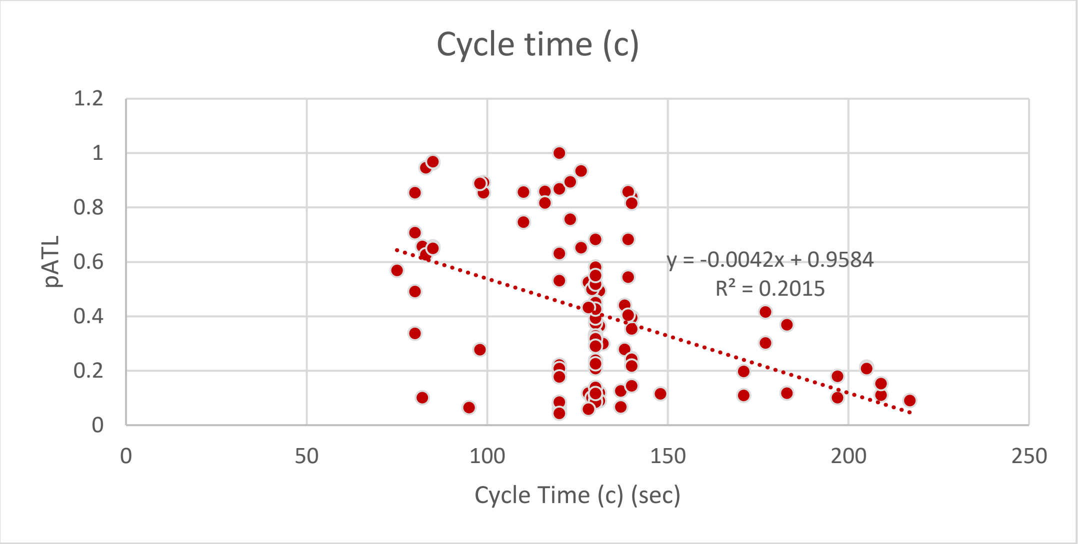


Figure 5-14 Cycle time scatter plot

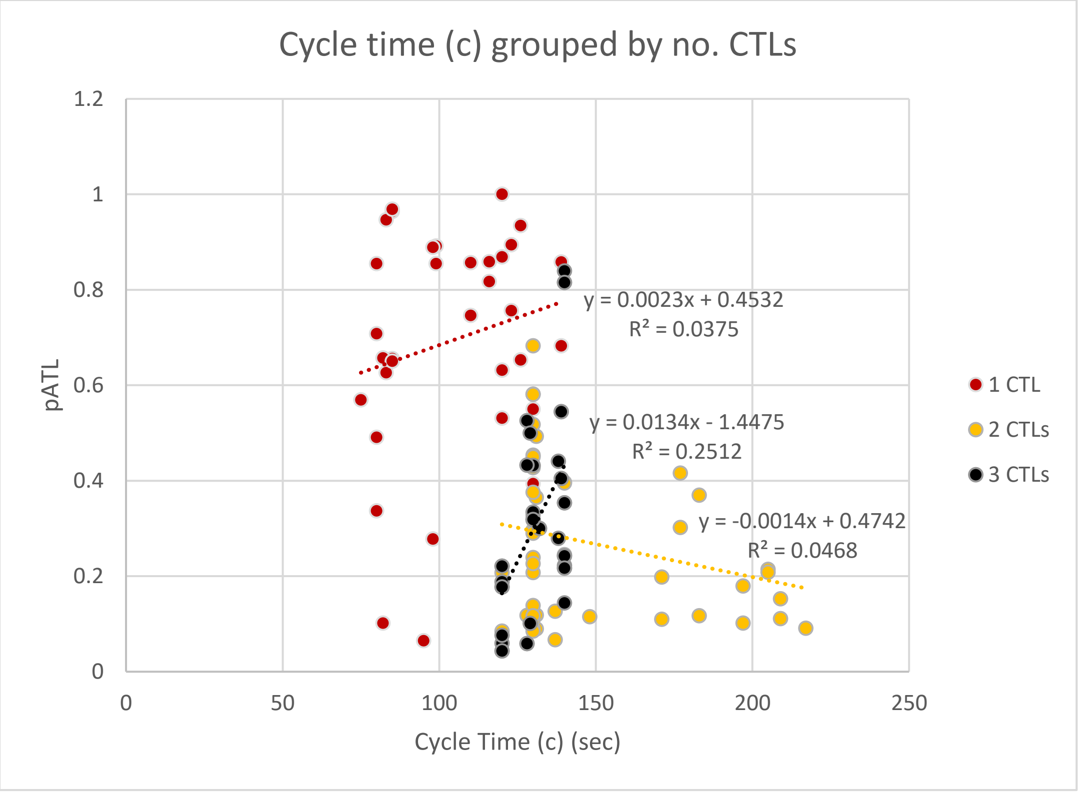


Figure 5-15 Cycle time (by CTLs) scatter plot

Green time (*G*)

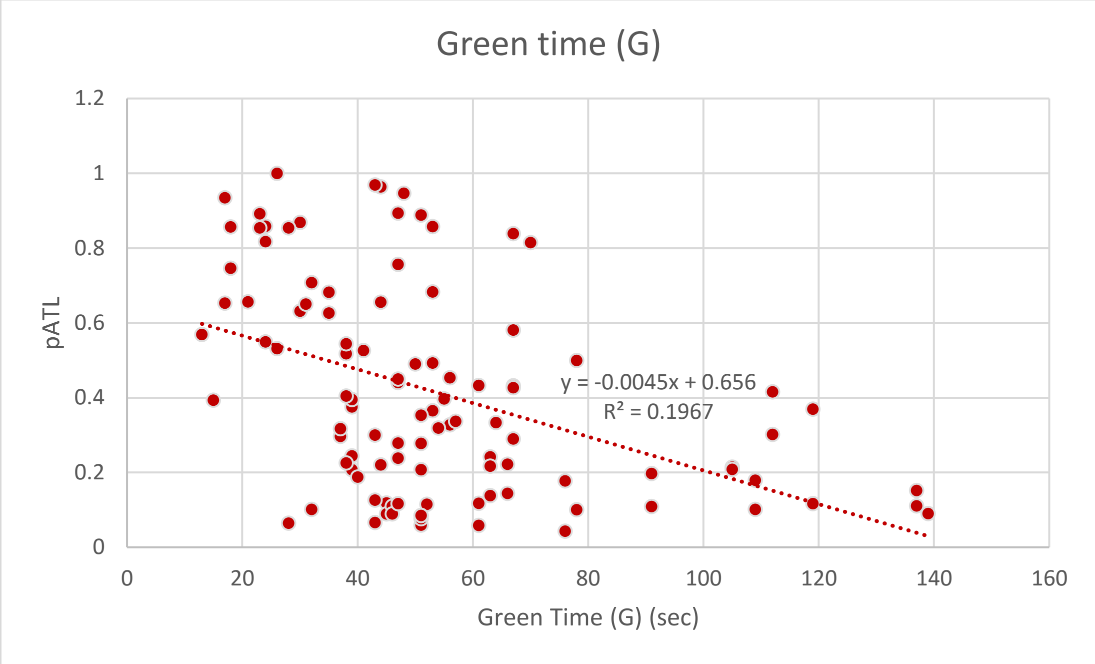


Figure 5-16 Green time scatter plot

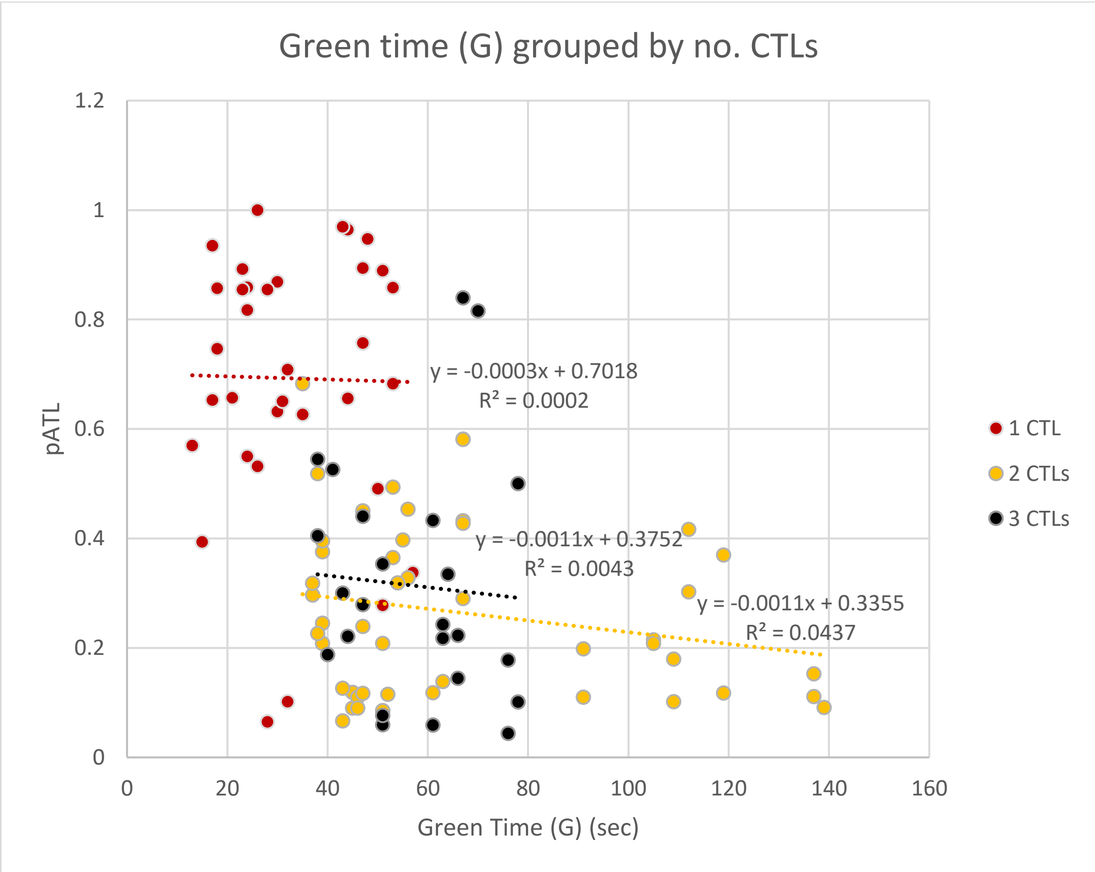


Figure 5-17 Green time (by CTLs) scatter plot

Green time / cycle time split (*g/c*)

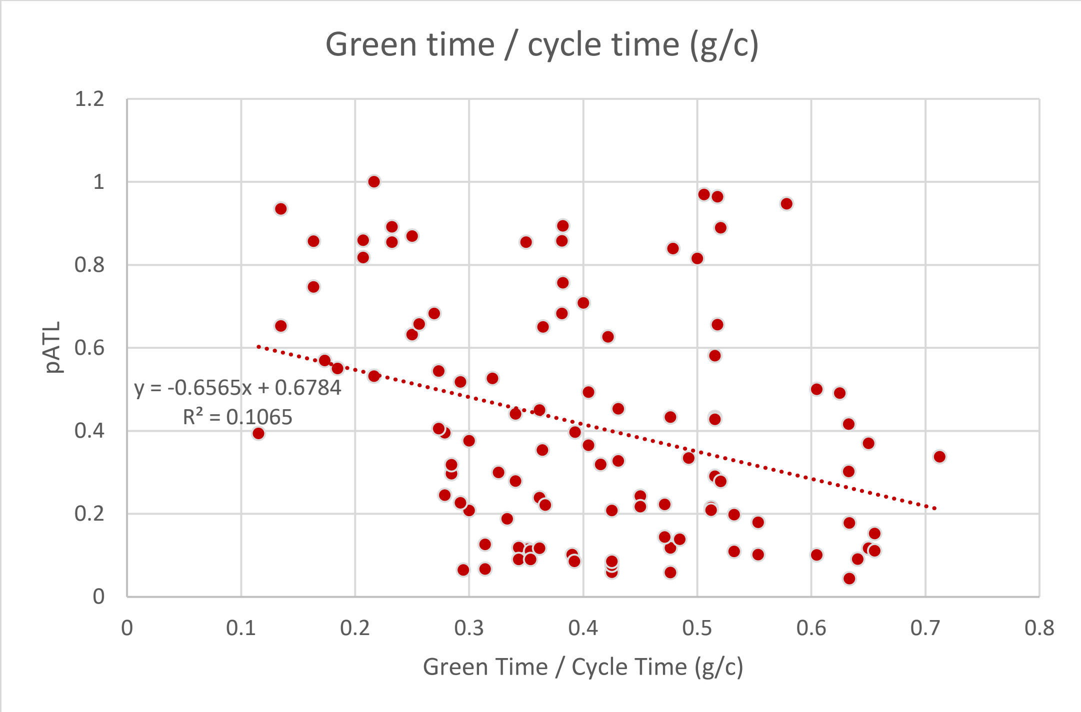


Figure 5-18 Green time/ cycle time split scatter plot

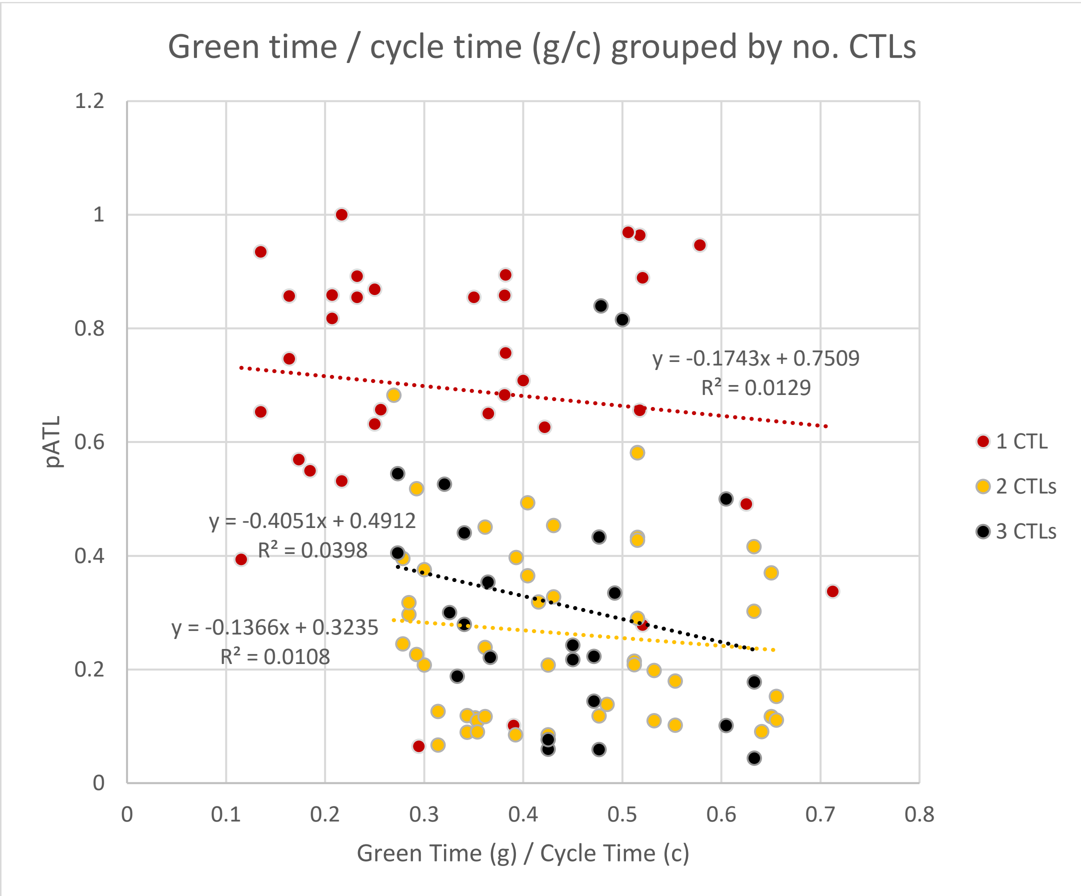


Figure 5-19 Green time/ cycle time split (by CTLs) scatter plot

5.3.4 Analysis of the degree of saturation

Figure 5-20 and Figure 5-21 show the relationship between the degree of saturation and the ATL utilisation, with the first graph showing the relationship for all sites combined and the second graph showing the relationship by the CTL quantity grouping.

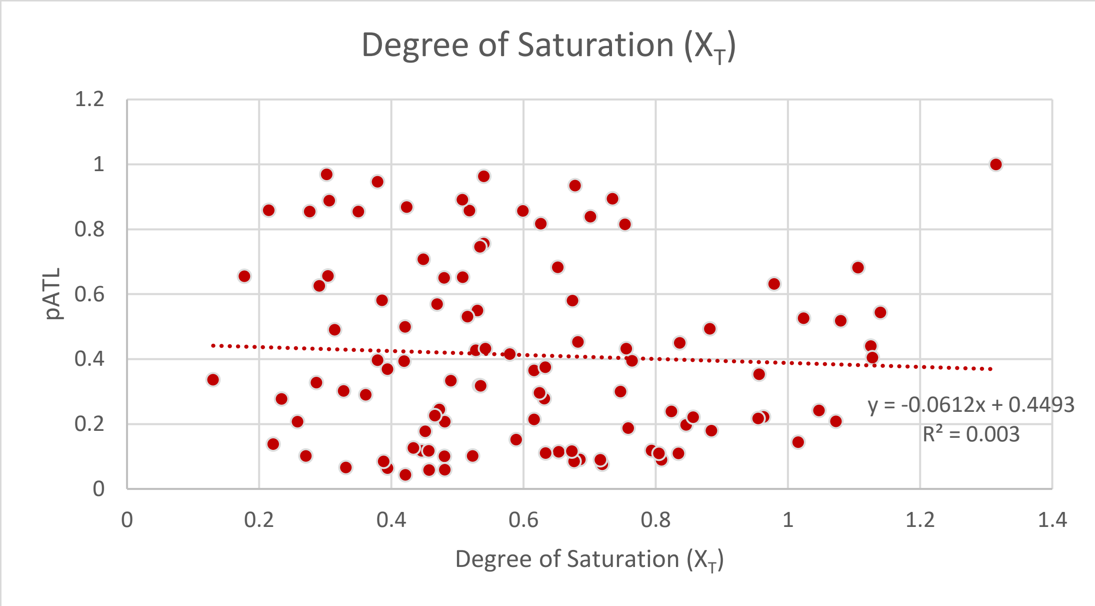


Figure 5-20 Degree of saturation scatter plot

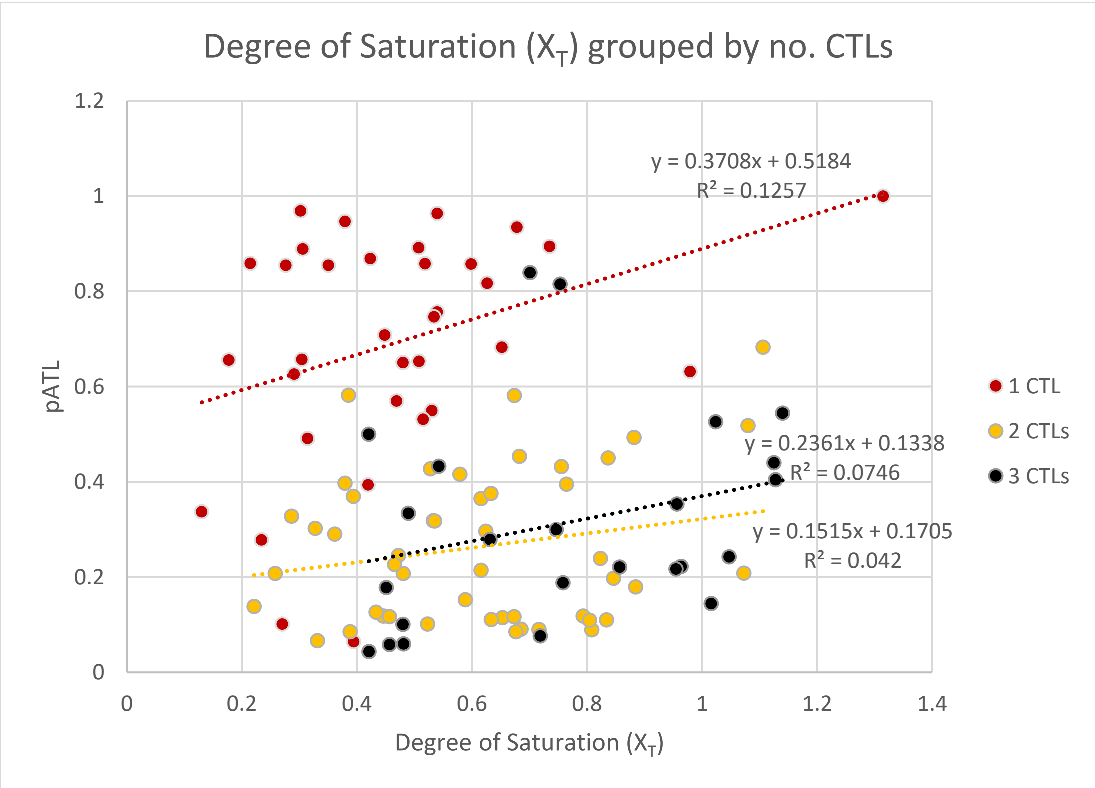


Figure 5-21 Degree of saturation (by CTLs) scatter plot

5.3.5 Speed limit analysis

Figure 5-22 represents the relationship between the posted speed limit on the approach and the ATL utilisation.

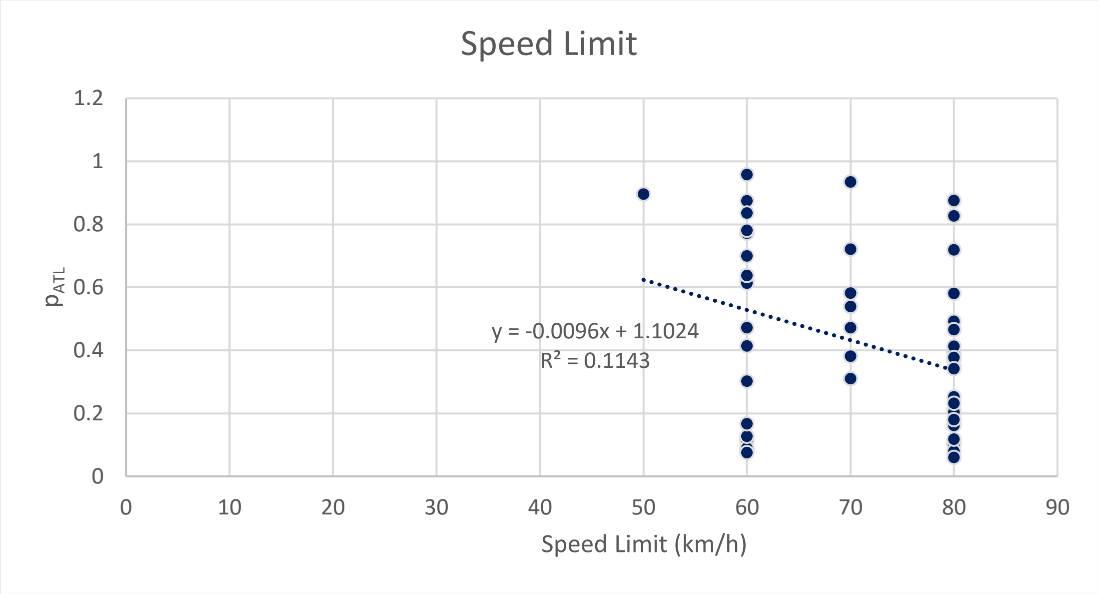


Figure 5-22 Speed limit scatter plot

5.3.6 Multiple linear regression analysis

The relationships between the individual independent variables and ATL utilisation were found to be linear, so a linear regression analysis is appropriate.

Initially, the multiple regression analysis considered all variables except the number of CTLs, producing 'Model 1', this resulted in an adjusted R-square value of 0.353. Section C.4.1 presents the full results of this regression analysis.

The analysis then included the number of CTLs present as a variable along with all other variables and undertook another multiple regression analysis; giving 'Model 2'. Section C.4.2 shows the full results, with the relationship giving an adjusted R-square value of 0.484.

The project then removed the green time and cycle time/ green time split from the regression analysis, as results showed that they had large P-values; this produced the third model run, 'Model 3'. This relationship provided an R-square value of 0.487, section C.4.3 presenting the full results.

Attempting to improve model accuracy further, the variable with the next highest P-value, the speed limit, was removed, and a fourth regression step analysed giving 'Model 4'. The relationship holds an R-square value of 0.482, with section C.4.4 showing the full results.

The analysis undertook another regression step based on 'Model 3' with just the total length (D) variable and not the approach (d_1) or departure (d_3) length variables in an attempt to simplify the relationship; giving 'Model 5'; this produced an R-square value of 0.472 with the full results shown in section C.4.5.

Table 5-1 – Correlation matrix for all variables

	d_1	d_3	D	c	G	g/c	XT	v	p_{ATL}
d_1	1.0000								
d_3	0.2344	1.0000							
D	0.8183	0.7433	1.0000						
c	0.0011	-0.0863	-0.0559	1.0000					
G	0.1324	0.0391	0.0908	0.7921	1.0000				
g/c	0.1679	0.1464	0.1741	0.3465	0.8315	1.0000			
XT	-0.1265	-0.1415	-0.1482	0.3593	0.0481	-0.1973	1.0000		
v	0.0177	0.0280	0.0369	0.5821	0.5501	0.3817	0.3350	1.0000	
p_{ATL}	0.2196	0.3034	0.3415	-0.4489	-0.4435	-0.3263	-0.0551	-0.3275	1.0000

The analysis produced a correlation matrix for all variables. Reviewing this suggested a high degree of correlation between D and the d_1 and d_3 variables; the hypothesis expected this as they are not wholly independent of each other. The d_1 and d_3 variables are independent of each other and do not have a strong correlation, so an attempt was made to improve on 'Model 3' (which had the highest R-square value so far) by removing the D variable from the regression analysis; giving 'Model 6'; this did not improve the model's variability as the R-square value dropped to 0.469 from 0.487. Section C.4.6 gives the full results of this analysis.

A high correlation was also found to exist between the G variable and the c, g/c and v variables. However, the project had previously removed the G variable from Models 3-6 due to the high P-value, so further analysis was not required here.

The v and c variables also register as having a degree of correlation; however, the project did not remove either from the analysis; this is discussed further in the discussion in section 6.3.6.

5.3.7 Selection of final model

The third multiple variable regression attempt had the highest R-square value; however, the D, d_1 and d_3 variables exhibited high correlation and are not independent of each other, this is discussed further in section 6.3.6. Model 6 has been selected as the best relationship to use as a model for this study as the model with the least variability with independent variables. The variable coefficients determined by the analysis, shown in Table C-21, have been placed into an equation to produce the ATL utilisation model:

$$\rho_{ATL}\ (\%) = 34.58\ X_T - 21.96\ CTL + 0.48\ v + 0.05\ d3 + 0.02\ d1 - 0.38\ c + 0.71 \qquad \text{(EQN 5.1)}$$

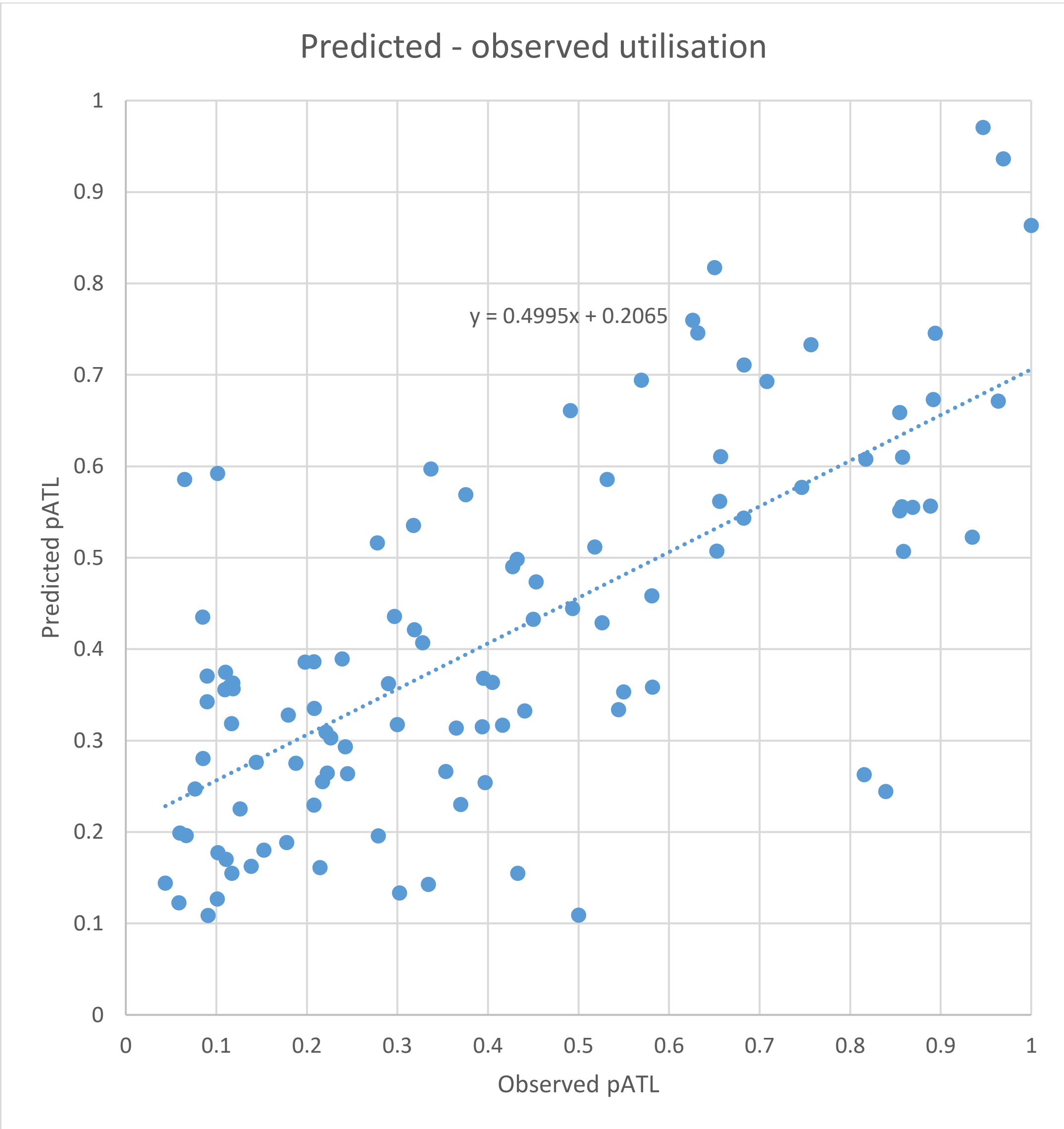


Figure 5-23 Study developed model validity plot

Figure 5-23 shows the results of a scatter plot created to compare the predicted ATL utilisation to the observed ATL utilisation.

5.4 Model comparison

5.4.1 Predicted utilisation using study model

The model developed in this study, presented at section 5.3.7, was used to predict the ATL utilisation for both AM and PM periods at the V2W and V9S sites; Table C-22 shows the model inputs and Table C-23 gives the results of the analysis.

5.4.2 Predicted utilisation using comparison models

The models selected for the comparison were each used to predict the ATL utilisation for AM and PM periods at the V2W and V9S test sites, the tables below show the inputs and results for the respective models:

- Australian Road Capacity Guide method, in Table C-24.
- Australian Road Research method, in Table C-25.
- SIDRA method, in Table C-26.
- Transport Research Board method, in Table C-27.

5.4.3 Observed utilisation

Section 5.3.1 included the calculation of these results, with Table C-3 showing these results as well.

5.4.4 Model results comparison

Table 5-2 summarises the results from the model predictions made in sections 5.4.1 and 5.4.2, along with the observed ATL utilisation for the test sites from section 5.4.3.

The residual value is the difference between the predicted and observed value for the ATL utilisation.

The analysis computed the average of the absolute values of the residual for each model; this gave a measure for the average error for each model across all four test scenarios.

Table 5-2 Model comparison results

		Observed	Study Model	ACRG Method	ARR Method	SIDRA Method	TRB Method
V2W AM	pATL	0.41775	0.29188	1.00000	0.42857	0.64872	1.18234
	Residual	0.00000	-0.1259	0.58225	0.01082	0.23097	0.76459
V2W PM	pATL	0.48812	0.28124	1.00000	0.42857	0.64872	1.22511
	Residual	0.00000	-0.2069	0.51188	-0.05955	0.16060	0.73699
V9S AM	pATL	0.63265	0.50387	1.00000	0.33333	0.49487	0.29724
	Residual	0.00000	-0.1288	0.36735	-0.29932	-0.13778	-0.33541
V9S PM	pATL	0.69565	0.45167	0.70586	0.33333	0.49487	0.22618
	Residual	0.00000	-0.244	0.01021	-0.36232	-0.20078	-0.46947
Absolute Average Residual			0.17638	0.36792	0.18300	0.18254	0.57662

6. Discussion

6.1 Site selection

The importance of rigorous site selection criteria cannot be understated here. As discussed by Royce et al. (2006), many variables can affect the utilisation of a lane at an intersection, with Lee and Park (2012) discussing the many factors affecting lane choice more generally. This research focuses on and examines only a select few of these factors. Outside factors need to be limited as far as practicable to isolate the effect of the subject variables. The site selection criteria set out in section 4.1 for this study aimed to eliminate any variance within the total sample caused by these external factors.

Despite the best efforts to control the environment in this case study, the results show that the model still does not account for a significant amount of variance. The most robust adjusted R-squared value of around 0.47 suggests that the model only accounts for 47 % of the variation in utilisation. It is likely that external factors occurring at the intersections also influenced the utilisation, leading to the additional variation between the predicted results and the observed results.

There is inevitable variability in traffic streams and human behaviour generally, and whilst we can quantify most values at an intersection, no two intersections will operate or perform the same, so it is unlikely that a model could be produced to account for all variation, but the R-square value provided does suggest that some variables were unaccounted for.

It’s possible that if the methodology had made inclusions for some of these other variables, rather than attempting to exclude them, the variability could have been reduced; this would also provide the benefit of minimising the applicability limits of the model, as it would allow more accurate predictions at intersections where these additional variables are at play.

The number of sites selected resulted in some 57 approaches, though two of these were unable to be analysed for all variables except for the ATL length variables due to strategic monitor data being unavailable. In the end, the research examined 55 approaches during the multiple regression analysis; this is considered to be a reasonable sample size from which to conclude. The sites also varied in their locality, with locations from around Tasmania, greater Melbourne and greater Perth.

Two sites were removed from the development of the model to ensure that the model comparison, undertaken in section 5.4, was independent of the model’s development.

6.2 Data collection

6.2.1 Auxiliary through lane lengths

Identification of ATL lengths used scaled measurements of aerial imagery. While there is scope for some errors to be introduced using this method, the level of accuracy required for this data was not high; lengths captured to the nearest meter were acceptable. Collection of this data through a survey or manual measurements would not be practicable and would not have increased the accuracy of the research.

6.2.2 Approach characteristics

The resolution of the image limits the accuracy of capturing lane widths and lateral clearances from aerial imagery; in some cases, it was difficult to achieve accuracy greater than +/- 0.2 m. For this reason, and simplicity, it was decided to group approaches into the lane width and lateral clearance groups provided within Table 2-1, reproduced from Austroads (2017).

The process of using predefined groupings instead of using the precise lane widths and interpolating the width factor does lead to some source of error in the approach capacity calculations; however, the effect is only considered to be minor. The method of theoretically calculating the approach capacity is only approximate, to begin with, so introducing this error source is not deemed to affect the results significantly. Capturing more accurate measurements than proposed by this project's methodology would be impracticable and would not likely improve the outcomes of this research.

Within this study, all approaches had multiple lanes; however, the approach capacity calculation methodology, given by Austroads (2017), only allowed for input of a single lane width or lateral clearance. Fortunately, most approaches had similar lane widths. The analysis averaged lane widths where this was not the case before placing the approach into the appropriate lane width bin. A single value for lateral clearance also needed to be specified, yet lateral clearance applies to both sides of all lanes and is typically dissimilar. Lateral clearances on each side of each lane were averaged, with the average of lateral clearances for all lanes then determined before placement in the standard lateral clearance bins.

Improvements to the accuracy of the calculated approach capacity were possible had the methodology calculated the capacity for each lane individually and then summed them to form an approach capacity. The tactic taken in this research was to generalise a typical lane for the approach and determine it is capacity, then multiplying this by the number of lanes on the approach to reach the approach capacity.

Determining the passenger car equivalent of a heavy vehicle requires the specification of the approach grade to the intersection. Table 2-2, reproduced from Austroads (2017), has three bins for approach grade: level, moderate or long sustained. These terms are more a qualitative measure than a quantitative measure, so rather than seek to determine the actual approach grade as a numerical value, a qualitative analysis of street-level imagery was undertaken for each approach. Given the broad categories, this was considered acceptable with negligible errors as a result.

The use of typical lane widths and lateral clearances may affect the accuracy of capacity calculated capacity for each approach; however, the capacity calculations themselves rely on several assumptions anyhow. This report considers that the methodology was acceptable for the purpose and the context of how the capacity value applied to this research.

6.2.3 Traffic flow data

The collection of vehicle flow data through SCATS was quite efficient and aligned well with the methodology required for this study. As discussed earlier, Karma et al. (2010) found that vehicle counts given by SCATS were acceptably similar to vehicle counts undertaken manually.

As SCATS vehicle detectors are unable to classify vehicles, the methodology proposed using alternate sources of data for determining the proportion of heavy vehicles. For most sites, vehicle counters were present within a reasonable distance upstream to consider them directly relevant to that approach. Unfortunately, there were some sites where this was not the case and, in these situations, the analysis used the nearest applicable vehicle counter. For those sites without a directly relevant vehicle counter, the estimated heavy vehicle proportion would introduce some margin of error, notably, if the approach experienced significantly different heavy vehicle flows to that assumed in this research.

In all cases, heavy vehicles flow proportions were only available as an average annual daily value. This value is not necessarily indicative of heavy vehicle flows during the morning and afternoon peak-hour flows. However, due to a lack of alternative data availability, the value was used for the approach capacity calculations regardless this is another source of error in determining the approach capacity, as heavy vehicle proportions can vary drastically over the day, typically being much lower during peak times.

6.2.4 Speed limits

Collection of speed limit data using road authority published maps was an effective method. Unfortunately, there was a minimal spread of speed limits within the study, with all but one site having a speed limit between 60 km/h and 80 km/h; this is reflective of the locations of traffic signals though, as their placement is typically on arterial roads which often have a minimum speed limit of 60 km/h, and seldom are traffic signals placed on roads with speed limits higher than 80 km/h.

6.2.5 Traffic signal cycle and phase timing data

The selection of typical cycle times and green phase times for the entire peak period was a difficult task and may lead to the largest source of error in the calculation of approach capacities, as well as consideration as input variables to the model.

The nature of the SCATS system, indeed one of its greatest strengths, is that these timing parameters are variable. The degree of saturation for each movement is monitored every phase to determine if the current time settings are appropriate, with changes potentially made every phase based on the current traffic flow (Lowrie 1992). The unit used for traffic flow is ‘vehicles per hour’; however, the flow rate usually varies within this period – particularly if the peak period is quite short. This variance in traffic flow throughout the period results in changes to the cycle time and green time throughout the peak hour.

Cycle times tended to remain relatively stable over the peak period, shifting by no more than 10 seconds, making the selection of a typical cycle time relatively easy. Phase times were much more erratic though, with some sites seeing variances in the order of 20 and 30 seconds across the peak hour. The selection of a typical phase time from within this range was seen as reasonable but may introduce errors when looking at the phase time as a variable influencing the use of the ATL – as this may change at every cycle. The methodology adopted in this research did not make provision for such variation in these parameters.

If future work was undertaken to improve on the results of this research, the methodology needs to account for the sometimes-erratic behaviour of these variables over the study period. It may be that multiple shorter periods, say 5-minute intervals, need to be analysed for the timing parameter variables.

6.3 Data analysis

6.3.1 Observed lane utilisation

Calculation of ATL utilisation using the methodology in section 4.3.1, adapted from Akcelik (1989a), required the assumption that the ATL and all CTLs on the approach would have the same saturation flow rate. Given that these are all dedicated through lanes and the sites have been selected to minimise the effect of other external variables, this assumption is considered to be fair. However, there is likely to be small differences in the actual saturation flow rate across the lanes. While this methodology does introduce some error into the results, the difference is likely to be negligible and to have minimal effect on the calculated utilisation ratio.

6.3.2 Length analysis

Approach Length

Comparing the observed ATL utilisation to the approach lengths for all sites combined (as seen in Figure 5-2) resulted in a poor relationship, the R-square value was only 0.05. A large proportion of the sites had approach lengths of between 10 m to 100m, with the ATL utilisation ranging from 8 % to 93 % in this small range alone. Most of the remaining sites had approach lengths between 100 m and 250 m, with similar patterns observed.

Breaking the sites into CTL groups (shown in Figure 5-3) did not show any real improvement. The strongest relationship was the two CTL group, but this only improved the approach length relationship marginally to an R-square value of 0.073. The analysis finds though that the sites with 1 CTL tended to have higher ATL utilisation than those with two or three, which both had similar patterns of ATL utilisation.

While the relationship is weak in all cases, the relationship was positive, suggesting that increased ATL approach lengths increase their utilisation; this supports the research by Karma et al. (2010) and Royce et al. (2006).

The analysis considered removing two outlying sites with approach lengths of 551 m and 716 m, but testing showed only marginal improvements in the relationship. The sites were also valid for all other variables, so they would still warrant inclusion in the multiple variable regression analysis. As such, it was decided to keep them in the dataset.

The weak relationship developed in this study suggests that ATL approach length alone cannot be used as a determinant of ATL utilisation, though there does appear to be a positive relationship. The data may also imply that a site with a single CTL will have higher ATL utilisation than one with two or three CTLs.

Departure length

The departure length to ATL utilisation relationship for all sites (shown in Figure 5-4) was marginally stronger than that of the approach length with an R-square value of around 0.1, however, this is still considered to be a weak relationship. All but three sites had departure lengths between 0 m and 200 m, with high and low utilisation rates seen at both ends of this length range. While the relationship may not explain ATL utilisation variation well on its own, it does suggest a positive relationship between departure length and ATL utilisation. As with the approach length results, this supports the research by Karma et al. (2010) and Royce et al. (2006).

Looking at the CTL groups individually (seen in Figure 5-5) there appears to be a negligible improvement – except for the 3 CTL group, where the relationship achieved an R-square value of 0.36.

As with the approach length, single CTL sites tended to have higher utilisation than two or three CTL sites. These two results suggest that the number of CTLs does have an impact on ATL utilisation.

The analysis considered removing the outlying site with a departure length of 667 m; as with the outliers for the approach length though, the site was valid for other variables and removal did not produce a stronger relationship, so the decision was made to keep it in the dataset.

The results of the study suggest that departure length cannot be used to determine ATL utilisation independently accurately. A positive relationship appears to exist between the two, suggesting that longer departure lengths typically lead to higher ATL utilisations though. The data repeats that founded in the approach length analysis that the number of CTLs has an impact on ATL utilisation as well.

Approach and departure length combined

Considering all sites together for the combination of approach and departure lengths (seen in Figure 5-6) resulted in a marginally stronger relationship with ATL utilisation than either of the variables alone, though it is still weak with an R-square value of around 0.11 - the positive correlation observed in the approach and departure length results individually remains.

The CTL groups (shown in Figure 5-7) tended to average the results seen between the approach and departure length individual analyses. The 3 CTL group still had the most substantial relationship to length, though the R-square value dropped to around 0.22. Single CTL sites maintained higher utilisation than two or three CTL sites; reflecting the results of the approach and departure length variables analysed individually.

This study shows that combining these two lengths produced a marginally stronger relationship than the length variables analysed individually, however, the association is too weak to be able to predict ATL utilisation without consideration of other explanatory variables.

Approach and intersection length combined

The addition of intersection length to approach length (shown in figures Figure 5-8 and Figure 5-9) made a negligible difference to the strength of the relationships. It tended to move the results along the length axis by the value of the intersection length. Minimal difference was observed between the sites as all but two intersections had intersection lengths in the range of 30 m to 50 m.

Departure and intersection length combined

The results of adding intersection length to the departure length reflect those made with the approach length (seen in Figure 5-10 and Figure 5-11).

Total length

When considering all sites in aggregate, the relationship of the total length to ATL utilisation (shown in Figure 5-12) produced a stronger relationship than approach and departure length did individually or combined, with an R-square value of some 0.12. As all the components of the total length had positive relationships, the total length also has a positive relationship with ATL utilisation; this supports the research by Karma et al. (2010) and Royce et al. (2006).

Breaking the sites into CTL groups (as in Figure 5-13) reflected similar results to those in the approach and departure length analyses.

The study would suggest that use of total length as a variable is marginally superior to using approach or departure lengths individually; however, the relationship is still too weak to be used to predict ATL utilisation with this variable alone. A positively correlating relationship does seem to exist between ATL length and ATL utilisation, and the number of CTLs does appear to affect ATL utilisation, with single CTL sites typically having higher ATL utilisation than two or three CTL sites, and three CTL sites having a stronger relationship with ATL length than one or two CTL sites.

ATL length overall

This study was unable to produce the closely correlating relationships given by Royce et al. (2006) and Karma et al. (2010), where relations between ATL length and utilisation were found to have R-square values of above 0.6. Though these studies do still cite that the relationship is still somewhat weak due to the small gradient in the line of best fit. These studies only considered small sample sizes, which may explain the difference in results. The positive relationship between length and ATL utilisation produced by this study did correlate with previous research, however.

6.3.3 Traffic signal timing parameters analysis

Cycle Time

When looking at the relationship of cycle time to ATL utilisation at sites overall (presented in Figure 5-14) there is a clear negative relationship, suggesting that shorter cycle times result in increased ATL utilisation; this aligns with the research by Bugg et al. (2013) and McCoy and Tobin (1982). However, when broken into CTL groupings (as shown in Figure 5-15) the patterns differ. While the sites with two CTLs maintained a negative relationship, the single and three CTL sites have a positive relationship with ATL utilisation. Driver behaviour may change based on the number of lanes available, but this is outside the scope of this study; this is another example suggesting that the number of CTLs effects ATL utilisation.

The strength of the relationship is stronger than that of the ATL lengths with an R-square value for all sites of 0.2, though this is still a weak relationship. Disaggregating the sites by CTL groups reveals particularly feeble relationships for the one and two CTL sites, but a slightly stronger relationship for the three CTL sites with an R-square of 0.25.

The study reaffirms that cycle time is undoubtedly a variable for consideration when predicting ATL utilisation, but that the relationship with this variable alone is not enough without considering other factors.

Green time

A negative relationship was confirmed with the relationship between ATL utilisation and the green time at all sites (presented in Figure 5-16), though this was less pronounced after disaggregating sites by CTLs (as seen in Figure 5-17); this aligns with the research by Bugg et al. (2013) and McCoy and Tobin (1982).

The strength of the relationship when considering all sites were still moderately weak with an R-square value of around 0.2 but becoming substantially weaker when considering by CTL groups individually.

As with the cycle time variable, this study reaffirms that green time is a crucial variable in considering the utilisation of ATLs and that a negative relationship exists. However, the association is too weak at accurately predicting ATL utilisation on its own.

Green time/ cycle time split

The relationship between the green time/ cycle time split and ATL utilisation is much weaker than the relationship with cycle time or green time independently, whether looking at all sites combined (shown in Figure 5-18) or by CTL groups (shown in Figure 5-19). The analysis gives an R-square value of 0.11 for all sites combined, with significantly weaker relationships seen when disaggregating by CTL groups. Again, the study finds a negative correlation.

This project considers that cycle time or green time would be more appropriate input variables than this ratio of the two values, given their stronger relationship independently.

6.3.4 Degree of saturation analysis

Research by Transportation Research Board of the National Academies (2011) suggested quite a strong relationship between the degree of saturation of an approach and the ATL utilisation, with an R-square value of 0.78 reported in their research. That study also produced a positive relationship between the two variables.

Comparing the results of that previous research and the results of this research gives mixed conclusions. When looking at the sites grouped by CTLs (seen in Figure 5-21) the positive relationship is reflected in this research, but the aggregate of all sites (shown in Figure 5-20) shows a slight negative relationship; this suggests a need to consider the number of CTLs as a variable when looking to predict ATL utilisation - a pattern emerging in most results of this research.

The study did not observe a relationship between ATL utilisation and the degree of saturation when sites were aggregated, with an R-square of 0.003. However, analysing sites by CTL groups showed comparatively stronger relationships with R-square values of 0.13, 0.08 and 0.04 for one, two and three CTL sites respectively; this suggests that a stronger relationship is present where a site only has one CTL than for sites with two and three CTLs, again highlighting the importance of the CTL variable.

While this study could not reproduce the strong relationship between the degree of saturation and ATL utilisation, it did suggest that it is an important factor for consideration in a multiple variable analysis. The study confirmed positive correlations when reviewed by CTL groups, in line with previous research by the Transportation Research Board of the National Academies (2011).

6.3.5 Speed limit analysis

Excluding the single site with a 50 km/h speed limit, all sites fell between the three speed limit points of 60, 70 and 80 km/h. Given this limited spread, it is challenging to develop a definitive relationship with this variable alone.

On average, the sites with 60 km/h speed limits had slightly higher ATL utilisation than those with 80 km/h speed limits, with the 70 km/h sites sitting somewhere in between. The 50 km/h site also had quite high ATL utilisation. Plotting a line of best fit between these then resulted in a negative relationship between the speed limit and ATL utilisation, confirming the hypothesis made in section 2.9 that higher speeds would result in less lane-changing behaviour, based on the research by Golbabaei et al. (2014), Archer et al. (n.d.) and Brubacher et al. (2018). The results do oppose those found by Soriguera et al. (2017), whose research showed the opposite trends to the papers mentioned above.

The relationship itself is still moderately weak with an R-square value of 0.11; however, the study suggests that it is still likely to be a contributing variable in a multiple variable analysis but could not be used on its own to predict ATL utilisation.

6.3.6 Multiple linear regression analysis

Multiple regression analysis allows consideration of numerous independent variables to form a relationship with a dependent variable; in this case, the ATL utilisation was the dependent variable.

The first multiple regression analysis included all variables except for intersection width. Based on the results discussed above this variable appeared to have negligible effect on ATL utilisation so was excluded. However, at this stage in the project, CTLs had not been considered an independent variable in the study and was not included. This first pass, dubbed Model 1 (see section C.4.1), had a somewhat weak relationship with ATL utilisation with an adjusted R-square value of around 0.35.

From a review of the results, it appeared evident that the number of CTLs was a significant factor in ATL utilisation, so in the second pass at multiple regression (Model 2, see section C.4.2), the CTL variable was included, this proved to form a moderately strong relationship with an adjusted R-square value of 0.484.

The study then sought to remove variables that may be having a negligible effect on the model to reduce the variability further. The P-value for each variable was considered, with large values indicating that the variable is less significant in changes to the dependent variable.

From Model 2, it was evident that the green time and green time/ cycle time split had quite high P-values; the next multiple regression step (Model 3, see section C.4.3) excluded these accordingly. This relationship had a slightly higher R-square value of 0.487.

A further attempt was made to improve on this model by removing the variable with the next highest P-value, the speed limit, from the analysis in Model 4 (see section C.4.4), but this reduced the strength of the relationship slightly.

An attempt was made to simplify the model in Model 5 (see section C.4.5) by removing the approach and departure length variables and relying on total length alone. However, this again reduced the strength of the relationship.

The correlation matrix undertaken showed that the total length variable, D, was highly correlated with the approach and departure length variables, d_1 and d_3; this is expected, as D is not independent of the approach and departure length variables, as it is the sum of the these and the intersection length variable, d_2. The analysis found little correlation between d_1 and d_3;

on this basis, the project proceeded to remove the total length ‘D’ from the regression analysis, giving Model 6 (see section C.4.6); this slightly increased the model’s variance, but was considered to be a stronger model than Model 3 as it relied upon independent variables only.

The correlation matrix also identified a strong correlation between the green time variable and the cycle time and green time/ cycle time split variables; this is logical, as the cycle time is the sum of all phase times at a signalised intersection, and an increase in the subject phase time would increase the cycle time as well – unless reduction to another phase time occurred concurrently. However, the project had already removed the green time variable from Models 3 to 6, so no further analysis was required.

The final two variables identified as having a degree of correlation was the speed limit, v, and the cycle time, c; this does not appear immediately logical, as the two variables are set by road authorities independently. However, sites with higher speed limits typically sit on higher category arterial roads, of which there is more traffic demand. These sites will also usually have a longer cycle time to cater for this traffic demand, though this is not necessarily always the case. This project considered that as the variables are independent, although they may tend to correlate, they will remain in the analysis together.

6.3.7 Final model selection

The model selected as the study model was Model 6. It contains the following variables as inputs, in order of significance:

- Degree of Saturation
- Number of CTLs
- Posted Speed Limit
- Cycle Time
- Departure Length
- Approach Length

While all these variables only had weak relationships with utilisation individually, when combined, they were able to produce a moderately strong association. The adjusted R-square value of 0.469 suggests that the model only accounts for 47% of the variance of ATL utilisation; this indicates that other variables are affecting the utilisation that the model has not considered. Attempts were made to limit this by carefully choosing the site selection criteria, but this may not have captured all possible external factors. It is also worth noting that traffic flows and human behaviour generally is inherently variable – not all drivers will respond the same way to a given set of circumstances. So, it is unlikely that any model could account for all variance in ATL utilisation.

The comparison of observed to predicted ATL utilisation using this model in Figure 5-23 shows the unexplained variance clearly. However, it also shows that the predicted values typically follow the trend of the observed values.

6.4 Model comparison

Table 5-2 gives the results of the model comparison; these show that amongst the models tested, the model developed in this study has the smallest average absolute residual; this indicates that over the four scenarios tested the study developed model had the least average error when predicting the ATL utilisation. The comparison found that the study model was not always the closest; in fact, it was only the closest on one occasion. Overall though the study model produced the least error.

The second smallest average absolute residual was from the Australian Road Research model, though this was only an insignificantly small margin lower than the SIDRA model. Interestingly, both of these models rely purely on a single variable – number of CTLs in the case of the Australian Road Research model and departure ATL length in the case of the SIDRA model, despite the results of this study suggesting that neither variable in their own right has a strong enough relationship to predict ATL utilisation accurately.

The fourth and fifth smallest average absolute residuals, the Australian Road Capacity Guide and Transport Research Board models, respectively, get progressively larger.

The model comparison only considered two sites in AM and PM periods for a total of 4 samples, so it is not a conclusive test of the comparison of these models. However, it does indicate that the model developed in this study is likely to be comparably accurate, if not more accurate, than the current methods of estimating ATL utilisation. Had the methodology isolated more sites for comparison testing, the results of the comparison may have been able to be more conclusive. On the same token though, this would have reduced the number of case study sites used to develop the model which may have reduced its accuracy.

6.5 Future research

The model, in its current form, outperforms the comparison model in the limited testing undertaken. However, the sample size of the test conducted is insufficient to be able to say if the model is validated and that it performs better than alternative methods with any confidence. The first task for any further research would be to validate the performance of this model against a significantly large number of case study sites – those sites being in addition to those used to develop the model in this research. The comparative models could also be used at these case study sites to give a more valid set of results for the average error made by each model.

There still exists a large amount of variability in the model; this variability will need to be reduced to make accurate predictions of ATL utilisation. The specifications of this research attempted to remove the effects of external factors not considered as input variables to the model by eliminating sites where such factors were present. However, the variability suggests that there may be other influencing factors not considered. The second task for additional research would be the identification of additional variables to be included within the model. It may be that factors previously identified within the literature review as having a negligible effect may indeed have a noticeable impact; this hypothesis needs to be tested. It may also be that the analysis needs to include previously unconsidered variables. For instance, the literature review for this project did not identify that the posted speed limit would affect ATL utilisation; however, this research does show a correlation. Such other variables will need to be identified, considered and tested.

The method of restricting site selection to exclude those sites with specific characteristics is valid for analysing the effects of the subject variables. However, this presents limitations in the model's potential uses in that it cannot accurately make predictions at sites that do have these characteristics. While engineering judgement could be used to factor the model's results; this goes against the project aims of providing an accurate prediction. The third task for future research would be to test the applicability of the current model against sites with these characteristics to determine if the accuracy does decrease. If this is the case, the research should undertake the expansion of the model’s applicability by the inclusion of some or all of these previously excluded variables; a similar approach to the methodology used in this research would be applicable. Potentially, only a select few additional variables would be required.

This project developed the model for the explicit purpose of determining auxiliary through lane utilisation, with the project defining these lanes as additional through lanes that commence and terminate immediately before and after a signalised intersection. Many roads will see a new lane added or a previously continuous lane dropped at signalised intersections. While these lane additions and drops have inherently different functions and subsequently unique factors affecting their utilisation, there may be some crossover. Further research could consider adapting the model to capture these situations as well; there is the potential for substantial crossover.

7. Conclusions

This research project has achieved the objectives set out at section 1.3; it has examined current methods of determining lane utilisation, identified key variables contributing to the use of ATLs and analysed these relationships using case study sites. The project has developed a mathematical model and tested and compared it to other prediction models. At least in the limited testing undertaken, the model has greater accuracy over current prediction methods, achieving the aim of the project in improving analytical and microsimulation model's ability to estimate intersection capacity. While a sub-aim of the project was to build a model primarily based on the length of an ATL, this did not prove to be feasible. Instead, the model included other key variables alongside the length variables.

The project has advanced this field of knowledge by proposing a model for the prediction of ATL utilisation at signalised intersections that may prove more accurate than current methods. The project achieved this by undertaking original research in combining ATL length variables with other contributing factors in a multiple linear regression analysis for the first time. The research also tested a new hypothesis that the posted speed limit would influence ATL utilisation.

The key findings of this research are summarised below:

- Total ATL length produces a stronger relationship than individual ATL length elements alone.
- A positive relationship exists between ATL length variables and ATL utilisation, suggesting ATL utilisation increases as length increases.
- The relationship between ATL length and ATL utilisation is relatively weak, suggesting the variable cannot be used to predict ATL utilisation on its own.
- Sites with 1 CTL tend to have higher ATL utilisation than sites with 2 or 3 CTLs.
- Sites with 3 CTLs tended to have a stronger relationship between ATL length and ATL utilisation.
- Cycle time, green time & cycle/ green time split produce moderately weak relationships with ATL utilisation on their own, but indeed, appear to be contributing factors.
- A negative relationship exists between cycle time, green time or green/ cycle time split and the resulting ATL utilisation, suggesting shorter times (or smaller splits) result in higher utilisation.
- The study shows a positive relationship between the degree of saturation and ATL utilisation, but only when considered in context with the number of CTLs.

- The relationship between the degree of saturation and ATL utilisation is too weak at predicting ATL utilisation on its own but is a contributing factor.
- The study suggests there is a negative relationship between the posted speed limit and ATL utilisation, indicating that higher speed limits result in lower utilisation.
- The relationship between the speed limit and ATL utilisation is too weak to be used as a variable on its own to predict ATL utilisation but is likely a contributing factor.
- The project has created a multiple variable relationship with moderate strength; the project proposes the following model:

$$\rho_{ATL}\ (\%) = 34.58\ X_T - 21.96\ CTL + 0.48\ v + 0.05\ d3 + 0.02\ d1 - 0.38\ c + 0.71 \qquad \text{(EQN 5.1)}$$

- The model developed by this study is comparably accurate, if not more accurate than current methods of predicting ATL utilisation. However, further comparison of these models would be required to confirm this with any degree of certainty.

As discussed within the report, many external factors contribute to ATL utilisation that have not been considered in the development of this model. Use of this model at sites where such variables are present may lead to inaccurate results. As such, the outcomes of this research are limited to those sites that comply with the defined site selection criteria. If it is used outside of these parameters, consideration would need to be given to the likely effects of the external factors.

8. References

Akcelik & Associates Pty Ltd 2019, *SIDRA Intersection 8 User Guide*, SIDRA Solutions, Greythorn, Victoria, Australia.

Akcelik, R 1981, *Traffic Signals - Capacity and Timing Analysis*, Research Report No. 123, Australian Road Research Board.

Akcelik, R 1989a, *On the estimation of lane flows for intersection analysis*, Research Report No. 19, Australian Road Research Board.

Akcelik, R 1989b, *Traffic Signals: Capacity and Analysis (7th Reprint)*, Research Report No. 123, Australian Road Research Board.

Archer, J, Fotheringham, N, Symmons, M & Corben, B n.d., *The Impact of Lowered Speed Limits in Urban Areas*, Monash University Accident Research Centre.

Australian Road Research Board 1968, *Provisional Introduction and Signalized Intersections (Bulletin No. 4)*, Australian Road Capacity Guide, Australian Road Research Board, Melbourne.

Austroads 2013, *Guide to Traffic Management Part 6: Intersections, Interchanges and Crossings*, AGTM06-13, Austroads, Sydney, NSW.

Austroads 2015, *Guide to Traffic Management Part 2: Traffic Theory*, AGTM02-15, Austroads, Sydney, NSW.

Austroads 2016, *Guide to Traffic Management Part 9: Traffic Operations*, AGTM09-16, Austroads, Sydney, NSW.

Austroads 2017, *Guide to Traffic Management Part 3: Traffic Studies and Analysis*, Austroads Ltd.,, Sydney NSW Australia.

Brubacher, JR, Chan, H, Erdelyi, S, Lovegrove, G & Faghihi, F 2018, 'Road Safety Impact of Increased Rural Highway Speed Limits in British Columbia, Canada', *Sustainability*.

Bugg, Z, Rouphail, NM & Schroeder, B 2013, 'Lane Choice Model for Signalized Intersections with an Auxiliary Through Lane', *Journal of Transportation Engineering*, vol. 139, no. 4, pp. 371-8.

Chen, P, Nakamura, H & Asano, M 2012, 'Lane Utilization Analysis of Shared Left-turn Lane Based on Saturation Flow Rate Modeling', *Procedia - Social and Behavioural Sciences*, vol. 43, pp. 178-91.

Department of State Growth 2019a, *LISTmap: Speed Limits on State Roads Layer*, 22 July 2019, <https://maps.thelist.tas.gov.au/listmap/app/list/map>.

Department of State Growth 2019b, *RoadsTas Traffic Stats*, 22 July 2019, <http://geocounts.com/traffic/au/stategrowth>.

Department of State Growth & Wyminga, Y 2019, *Provision of SCATS recorded data and How it is extracted*.

Federal Highway Administration 2004, *Signalized Intersections: Informational Guide*, US Department of Transport, FHWA-HRT-04-091, Portland OR USA.

Federal Highway Administration 2006, *Traffic Detector Handbook*, UDo Transport, FHWA-HRT-06-108, McLean VA USA.

Garber, NJ & Hoel, LA 2010, *Traffic and Highway Engineering*, Cengage Learning, Stamford CT USA.

Golbabaei, F, Moghadas-Nejad, F & Noory, AR 2014, 'A microscopic analysis of speed deviation impacts on lane-changing behaviour', *Transportation Planning and Technology*, vol. 37, no. 4, pp. 391-407.

Hurley, JW 1998, 'Utilization of Double Left-Turn Lanes with Downstream Lane Reductions', *Journal of Transportation Engineering*, vol. 123, no. 3, pp. 235-9.

Institute of Transportation Engineers 2008, *Canadian Capacity Guide for Signalized Intersections*, Committee on the Canadian Capacity Guide for Signalized Intersections, Ottawa ON Canada.

Karma, R, Douglas-Jones, R & Jaglal, M 2010, *Utilisation of the kerbside through-lane at signalised intersections*, Research Report No. 398, New Zealand Transport Agency.

Lee, J-J, Hummer, JE & Rouphail, NM 2005, *False Capacity for Lane Drops*, North Carolina Department of Transportation Research and Analysis Group, North Carolina State University, Raleigh NC USA.

Lee, J & Park, BB 2012, 'Determining Lane Use Distributions Using Basic Freeway Segment Density Measures', *Journal of Transportation Engineering*, vol. 138, no. 2, pp. 210-7.

Lowrie, PR 1992, *SCATS - Sydney Co-ordinated Adaptive Traffic System: A Traffic Responsive Method of Controlling Urban Traffic*, Roads and Traffic Authority NSW, Darlinghurst NSW Australia.

Main Roads Western Australia 2019a, *Traffic Digest*, 22 July 2019, <https://catalogue.data.wa.gov.au/dataset/traffic-digest>.

Main Roads Western Australia 2019b, *Legal Speed Limits*, 22 July 2019, <https://catalogue.data.wa.gov.au/dataset/legal-speed-limits>.

Main Roads Western Australia & Saunders, H 2019, *Traffic Signal Data for Research Project*.

Mannering, FL, Washburn, SS & Kilareski, WP 2009, *Principles of Highway Engineering and Traffic Analysis*, 4 edn, Wiley, Hoboken NJ USA.

McCoy, PT & Tobin, JR 1982, 'Use of additional through lanes at signalized intersections', *Transport Research Record*, no. 869, pp. 1-5.

Ogden, KW & Taylor, SY 1999, *Traffic Engineering and Management*, Monash University, Clayton, Victoria.

Okura, I & Somasundaraswaran, K 1996, 'Analysis of Traffic Distribution in Three Lane Uni-Directional Freeway', *JIPM*, vol. 13, pp. 885-92.

Roads and Maritime Services NSW n.d., *SCATS 6 Functional Description*.

Royce, B, Jurisich, I & Dunn, R 2006, *Through-lane Use at Traffic Signals*, Research Report No. 297, Land Transport New Zealand.

Soriguera, F, Martinez, I, Sala, M & Menendez, M 2017, 'Effects of low speed limits on freeway traffic flow', *Transportation Research Journal*, vol. 77, pp. 257-74.

Tarawneh, MS & Tarawneh, TM 2002, 'Effect on Utilization of Auxiliary Through Lanes of Downstream Right-Turn Volume', *Journal of Transportation Engineering*, vol. 128, no. 5, pp. 458-64.

Transportation Research Board of the National Academies 2010, *Highway Capacity Manual 2010*, National Academy of Sciences, Washington DC, USA.

Transportation Research Board of the National Academies 2011, *Guidelines on the Use of Auxiliary Through Lanes at Signalized Intersections*, National Cooperative Highway Research Program, Washington DC USA.

VicRoads 2019a, *Traffic Signal Volume Data*, 22 July 2019, <https://discover.data.vic.gov.au/dataset/traffic-signal-volume-data>.

VicRoads 2019b, *Traffic Signal Configuration Data Sheets*, 22 July 2019, <https://discover.data.vic.gov.au/dataset/traffic-signal-configuration-data-sheets>.

VicRoads 2019c, *Department of Transport Open Data: Traffic Volume*, 22 July 2019, <https://vicroadsopendata-vicroadsmaps.opendata.arcgis.com/datasets/traffic-volume>.

VicRoads 2019d, *Speed Zone Data*, 22 July 2019, <https://discover.data.vic.gov.au/dataset/speed-zone-data>.

VicRoads 2019e, *Traffic Signal Strategic Monitor Data*, 22 July 2019, <https://discover.data.vic.gov.au/dataset/traffic_signal_strategic_monitor_data>.

VicRoads & Lee, C 2019, *Short Lane Utilisation Data*.

Yang, L, Li, X, Guan, W, Zhang, HM & Fan, L 2018, 'Effect of traffic density on drivers' lane change and overtaking maneuvers in freeway situation - A driving simulator based study', *Traffic Injury Prevention*.

Appendix A Project Specification

University of Southern Queensland
FACULTY OF ENGINEERING AND SURVEYING

ENG4111/4112 Research Project

Project Specification

For: Nicholas Raymond Browne

Title: Developing a model for the utilisation of auxiliary through lanes at signalised intersections

Major: Civil Engineering

Supervisors: Soma Somasundaraswaran

Enrolment: ENG4111 – EXT S1, 2019

ENG4112 – EXT S2, 2019

Project Aim: The overarching aim is to improve the accuracy of analytical & microsimulation models to determine the capacity of signalised intersections with short auxiliary through lanes.

The project aims to confirm previous research that a relationship exists between the utilisation of auxiliary through lanes and their length and develop a model for predicting utilisation based on this carriable. The project will also consider other variables that may explain variations in lane utilisation for inclusion in the model

Programme: Version 3, 23rd July 2019

The objectives of this project are to:

1. Examine current methods of determining lane utilisation for existing signalised intersections and the prediction models currently available.
2. Undertake case studies to determine the utilisation of auxiliary through lanes at existing intersections.
3. Determine the key variables contributing to utilisation of auxiliary through lanes at signalised intersections.
4. Analyse the relationship between the key variables identified.
5. Develop a mathematical relationship between the selected variables and the utilisation of the auxiliary through lane.
6. Estimate lane utilisation of auxiliary through lanes using the developed model and compare the results with other prediction models and case study results.

Appendix B Data Collection

B.1 Auxiliary through lane lengths

Table B-1 ATL Length

Site ID	Approach	d_1 (m)	d_2 (m)	d_3 (m)	D (m)
T1	S	89	30	51	170
T2	S	64	35	52	151
T2	N	551	35	81	667
T3	S	103	33	141	277
T3	N	194	33	161	388
T4	S	187	29	79	295
T4	N	91	26	141	258
T5	N	55	37	50	142
T6	N	50	35	45	130
V1	S	83	36	124	243
V1	N	67	37	54	158
V1	E	87	40	12	139
V1	W	22	39	50	111
V2	W	123	48	87	258
V2	E	261	49	159	469
V3	S	103	42	114	259
V3	N	98	40	93	231
V4	N	37	27	66	130
V5	S	42	57	49	148
V6	W	30	40	30	100
V7	S	162	46	62	270

V7	N	76	46	342	464
V8	S	91	36	63	190
V8	E	79	37	30	146
V8	W	68	39	27	134
V9	W	140	44	103	287
V9	E	146	44	237	427
V9	S	61	51	53	165
V9	N	81	53	84	218
V10	E	113	38	29	180
V11	W	69	38	49	156
V11	E	51	39	66	156
V12	W	44	26	40	110
V12	E	47	28	27	102
V13	W	106	35	157	298
V14	N	53	105	112	270
V14	S	53	122	46	221
V15	N	32	28	26	86
V15	S	33	29	20	82
V16	E	14	34	99	147
V16	W	91	38	8	137
W1	W	242	41	191	474
W1	E	218	41	93	352
W2	S	98	39	42	179
W2	N	80	37	45	162
W3	W	63	43	163	269

W3	E	162	42	50	254
W4	W	71	45	63	179
W4	E	61	48	36	145
W5	W	50	41	58	149
W6	N	215	36	80	331
W6	S	98	42	90	230
W7	S	114	42	32	188
W7	N	49	42	45	136
W8	S	186	27	667	880
W8	N	716	29	146	891
W9	W	109	33	98	240

B.2 Approach characteristics

Table B-2 Approach Characteristics

Site ID	Approach	Lateral Clearance (m)	Lane Width (m)	Approach Grade
T1	S	1	3.7	Moderate
T2	S	1	3.7	Level
	N	1	3.7	Level
T3	S	1	3.7	Moderate
	N	1	3.7	Moderate
T4	S	0	3.7	Level
	N	0	3.7	Level
T5	N	0	3.2	Level
T6	N	1	3.2	Level
V1	S	0	3.2	Level
	N	0	3.2	Level
	E	0	3.2	Level
	W	0	3.2	Level
V2	W	0	3.2	Level
	E	0	3.2	Level
V3	S	1	3.2	Level
	N	1	3.2	Level
V4	N	1	3.2	Level
V5	S	0	3.2	Moderate
V6	W	1	3.2	Level
V7	S	1	3.2	Level
	N	1	3.2	Level
V8	S	0	3.7	Moderate

	E	0	3.7	Level
	W	0	3.7	Moderate
V9	W	1	3.2	Level
	E	1	3.2	Level
	S	1	3.2	Level
	N	1	3.2	Level
V10	E	0	3.7	Level
V11	W	1	3.2	Level
	E	1	3.2	Level
V12	W	0	3.2	Level
	E	0	3.2	Level
V13	W	0	3.2	Level
V14	N	0	3.2	Level
	S	0	3.2	Level
V15	N	0	3.2	Level
	S	0	3.2	Level
V16	E	1	3.2	Level
	W	1	3.2	Level
W1	W	1	3.2	Level
	E	1	3.2	Level
W2	S	0	3.2	Level
	N	0	3.2	Level
W3	W	1	3.2	Level
	E	1	3.2	Level
W4	W	0	3.2	Level
	E	0	3.2	Level
W5	W	0	3.7	Level
	N	1	3.7	Level

W6	S	1	3.7	Level
W7	S	0	3.7	Level
	N	0	3.7	Level
W8	S	1	3.7	Moderate
	N	1	3.7	Level
W9	W	0	3.7	Level

B.3 Traffic flow data

Table B-3 Traffic flow data

Site ID	Approach	Lane	q (Lane) AM (veh/hr)	q (Movement) AM (veh/hr)	q (Lane) PM (veh/hr)	q (Movement) PM (veh/hr)	P_{hv}
T1	S	T(L)	515	1174	1032	2245	0.089
		T(R)	591		1112		
		A	68		101		
T2	S	T(L)	526	1224	904	1752	0.098
		T(R)	536		742		
		A	162		106		
	N	T(L)	828	2162	634	1512	0.098
		T(R)	942		641		
		A	392		237		
T3	S	T(L)	699	1456	947	2089	0.096
		T(R)	686		1028		
		A	71		114		
	N	T(L)	1043	2462	883	1941	0.096
		T(R)	1203		918		
		A	216		140		
T4	S	T(L)	565	1335	789	1812	0.106
		T(R)	634		922		
		A	136		101		
	N	T(L)	962	2325	869	1905	0.106
		T(R)	1128		864		

		A	235		172		
T5	N	A	208	470	285	616	0.09
		T	262		331		
T6	N	T	325	358	370	394	0.132
		A	33		24		
V1	S	T(L)	538	1440	502	1221	0.07
		T(R)	604		493		
		A	298		226		
	N	T(L)	452	1006	565	1202	0.07
		T(R)	389		502		
		A	165		135		
	E	T(L)	498	1111	537	1203	0.06
		T(R)	548		600		
		A	65		66		
	W	T(L)	488	1132	439	1033	0.06
		T(R)	591		545		
		A	53		49		
V2	W	T(L)	358	1397	340	1444	0.06
		T(C)	384		415		
		T(R)	462		463		
		A	193		226		
	E	T(L)	420	1765	313	1309	0.07
		T(C)	466		411		
		T(R)	576		450		
		A	303		135		

V3	S	T(L)	817	2116	504	1203	0.06
		T(R)	907		530		
		A	392		169		
	N	T(L)	435	1080	725	1887	0.06
		T(R)	392		735		
		A	253		427		
V4	N	T(L)	502	1155	613	1442	0.06
		T(R)	584		764		
		A	69		65		
V5	S	T(L)	338	1177	469	1701	0.06
		T(C)	328		512		
		T(R)	383		532		
		A	128		188		
V6	W	T(L)	296	588	590	1335	0.05
		T(R)	251		667		
		A	41		78		
V7	S	T(L)	332	762	418	939	0.09
		T(R)	356		397		
		A	74		124		
	N	T(L)	442	1022	365	820	0.07
		T(R)	414		339		
		A	166		116		
V8	S	T(L)	224	576	410	1152	0.07
		T(R)	252		441		
		A	100		301		

	E	T(L)	375	940	265	601	0.06
		T(R)	405		274		
		A	160		62		
	W	T(L)	207	522	498	1252	0.06
		T(R)	253		496		
		A	62		258		
V9	W	T(L)	587	1568	397	993	0.08
		T(R)	675		462		
		A	306		134		
	E	T(L)	299	671	576	1478	0.06
		T(R)	274		632		
		A	98		270		
	S	T	98	160	92	156	0.07
		A	62		64		
	N	T	127	177	231	358	0.05
		A	50		127		
V10	E	T(L)	392	1460	499	1815	0.08
		T(C)	495		587		
		T(R)	480		597		
		A	93		132		
V11	W	T(L)	372	753	279	607	0.04
		T(R)	334		270		
		A	47		58		
	E	T(L)	245	565	413	897	0.06
		T(R)	300		446		

		A	20		38		
V12	W	T(L)	755	2468	808	2560	0.06
		T(C)	752		801		
		T(R)	786		755		
		A	175		196		
	E	T(L)	803	2602	685	2335	0.06
		T(C)	831		731		
		T(R)	846		755		
		A	122		164		
V13	W	T(L)	450	1823	516	2047	0.06
		T(C)	457		518		
		T(R)	498		558		
		A	418		455		
V14	N	T(L)	255	1158	423	1660	0.07
		T(C)	330		484		
		T(R)	448		536		
		A	125		217		
	S	T(L)	513	2063	449	1678	0.07
		T(C)	618		485		
		T(R)	647		480		
		A	285		264		
V15	N	T(L)	211	1382	274	1404	0.06
		T(C)	433		485		
		T(R)	492		435		
		A	246		210		

	S	T(L)	414	1578	302	1183	0.06
		T(C)	521		413		
		T(R)	584		442		
		A	59		26		
V16	E	T(L)	507	1481	720	2070	0.06
		T(C)	537		674		
		T(R)	405		548		
		A	32		128		
	W	T(L)	702	2214	403	1932	0.06
		T(C)	784		756		
		T(R)	668		740		
		A	60		33		
W1	W	T	517	870	411	722	0.097
		A	353		311		
	E	T	366	680	510	966	0.116
		A	314		456		
W2	S	T	145	271	364	730	0.124
		A	126		366		
	N	T	391	638	190	291	0.105
		A	247		101		
W3	W	T	85	158	222	420	0.078
		A	73		198		
	E	A	197	438	100	217	0.136
		T	241		117		
W4	W	T	154	255	234	442	0.037

		A	101		208		
	E	A	373	760	72	331	0.059
		T	387		259		
W5	W	T	137	227	151	237	0.07
		A	90		86		
W6	N	A	88	349	243	738	0.149
		T	261		495		
	S	T	394	673	248	460	0.151
		A	279		212		
W7	S	T	138	267	154	286	0.069
		A	129		132		
	N	T	121	200	146	255	0.069
		A	79		109		
W8	S	A	356	732	251	510	0.098
		T	376		259		
	N	A	191	496	279	708	0.068
		T	305		429		
W9	W	T	189	300	270	501	0.15
		A	111		231		

B.4 Speed limits

Table B-4 Speed limits

Site ID	Approach	Speed Limit (km/h)
T1	S	80
T2	S	80
T2	N	80
T3	S	80
T3	N	80
T4	S	80
T4	N	80
T5	N	60
T6	N	60
V1	S	60
V1	N	60
V1	E	60
V1	W	60
V2	W	80
V2	E	80
V3	S	80
V3	N	80
V4	N	80
V5	S	80
V6	W	60
V7	S	80

V7	N	80
V8	S	70
V8	E	70
V8	W	70
V9	W	80
V9	E	80
V9	S	70
V9	N	70
V10	E	80
V11	W	60
V11	E	60
V12	W	80
V12	E	80
V13	W	80
V14	N	80
V14	S	80
V15	N	80
V15	S	80
V16	E	80
V16	W	80
W1	W	80
W1	E	80
W2	S	70
W2	N	70
W3	W	60

W3	E	60
W4	W	60
W4	E	60
W5	W	60
W6	N	60
W6	S	60
W7	S	50
W7	N	60
W8	S	60
W8	N	60
W9	W	70

B.5 Traffic signal cycle and phase timing data

Data for T5N and W9W were unavailable due to communications errors at the sites.

Table B-5 Traffic signal cycle and phase time data

Site ID	Approach	c AM (sec)	G AM (sec)	g/c AM	c PM (sec)	G PM (sec)	g/c PM
T1	S	148	52	0.351351	217	139	0.640553
T2	S	177	112	0.632768	183	119	0.650273
	N	177	112	0.632768	183	119	0.650273
T3	S	197	109	0.553299	209	137	0.655502
	N	197	109	0.553299	209	137	0.655502
T4	S	205	105	0.512195	171	91	0.532164
	N	205	105	0.512195	171	91	0.532164
T6	N	82	32	0.390244	95	28	0.294737
V1	S	131	53	0.40458	130	47	0.361538
	N	131	53	0.40458	130	47	0.361538
	E	131	45	0.343511	130	46	0.353846
	W	131	45	0.343511	130	46	0.353846
V2	W	128	41	0.320313	132	43	0.325758
	E	128	41	0.320313	132	43	0.325758
V3	S	130	67	0.515385	130	54	0.415385
	N	130	67	0.515385	130	67	0.515385
V4	N	128	61	0.476563	130	51	0.392308
V5	S	130	64	0.492308	140	51	0.364286
V6	W	130	63	0.484615	130	47	0.361538
V7	S	130	39	0.3	130	37	0.284615
	N	130	39	0.3	130	37	0.284615
V8	S	140	55	0.392857	130	35	0.269231
	E	140	39	0.278571	130	38	0.292308
	W	140	39	0.278571	130	38	0.292308
V9	W	130	56	0.430769	130	67	0.515385
	E	130	56	0.430769	130	67	0.515385

	S	130	15	0.115385	130	24	0.184615
	N	130	15	0.115385	130	24	0.184615
V10	E	120	40	0.333333	120	44	0.366667
V11	W	137	43	0.313869	120	51	0.425
	E	137	43	0.313869	120	51	0.425
V12	W	140	66	0.471429	140	63	0.45
	E	140	66	0.471429	140	63	0.45
V13	W	140	67	0.478571	140	70	0.5
V14	N	138	47	0.34058	139	38	0.273381
	S	138	47	0.34058	139	38	0.273381
V15	N	129	78	0.604651	128	61	0.476563
	S	129	78	0.604651	128	61	0.476563
V16	E	120	51	0.425	120	76	0.633333
	W	120	51	0.425	120	76	0.633333
W1	W	139	53	0.381295	123	47	0.382114
	E	139	53	0.381295	123	47	0.382114
W2	S	120	30	0.25	120	26	0.216667
	N	120	30	0.25	120	26	0.216667
W3	W	116	24	0.206897	99	23	0.232323
	E	116	24	0.206897	99	23	0.232323
W4	W	85	44	0.517647	98	51	0.520408
	E	85	44	0.517647	98	51	0.520408
W5	W	82	21	0.256098	75	13	0.173333
W6	N	80	57	0.7125	80	50	0.625
	S	80	32	0.4	80	28	0.35
W7	S	126	17	0.134921	110	18	0.163636
	N	126	17	0.134921	110	18	0.163636
W8	S	83	48	0.578313	85	43	0.505882
	N	83	35	0.421687	85	31	0.364706

Appendix C Data Calculations

C.1 Capacity width factors and heavy vehicle factors

This data was computed using Table 2-1 and Table 2-2.

Table C-1 Capacity width factors and heavy vehicle factors

Site ID	Approach	Lateral Clearance (m)	Lane Width (m)	f_w	E_{hv}	P_{hv}	f_{hv}
T1	S	1	3.7	0.9	4	0.089	0.789266
T2	S	1	3.7	0.9	2	0.098	0.910747
	N	1	3.7	0.9	2	0.098	0.910747
T3	S	1	3.7	0.9	4	0.096	0.776398
	N	1	3.7	0.9	4	0.096	0.776398
T4	S	0	3.7	0.6	2	0.106	0.904159
	N	0	3.7	0.6	2	0.106	0.904159
T5	N	0	3.2	0.6	2	0.09	0.917431
T6	N	1	3.2	0.8	2	0.132	0.883392
V1	S	0	3.2	0.6	2	0.07	0.934579
	N	0	3.2	0.6	2	0.07	0.934579
	E	0	3.2	0.6	2	0.06	0.943396
	W	0	3.2	0.6	2	0.06	0.943396
V2	W	0	3.2	0.6	2	0.06	0.943396
	E	0	3.2	0.6	2	0.07	0.934579
V3	S	1	3.2	0.8	2	0.06	0.943396
	N	1	3.2	0.8	2	0.06	0.943396
V4	N	1	3.2	0.8	2	0.06	0.943396
V5	S	0	3.2	0.6	4	0.06	0.847458
V6	W	1	3.2	0.8	2	0.05	0.952381
V7	S	1	3.2	0.8	2	0.09	0.917431
	N	1	3.2	0.8	2	0.07	0.934579
V8	S	0	3.7	0.6	4	0.07	0.826446
	E	0	3.7	0.6	2	0.06	0.943396
	W	0	3.7	0.6	4	0.06	0.847458
V9	W	1	3.2	0.8	2	0.08	0.925926
	E	1	3.2	0.8	2	0.06	0.943396

	S	1	3.2	0.8	2	0.07	0.934579
	N	1	3.2	0.8	2	0.05	0.952381
V10	E	0	3.7	0.65	2	0.08	0.925926
V11	W	1	3.2	0.8	2	0.04	0.961538
	E	1	3.2	0.8	2	0.06	0.943396
V12	W	0	3.2	0.6	2	0.06	0.943396
	E	0	3.2	0.6	2	0.06	0.943396
V13	W	0	3.2	0.6	2	0.06	0.943396
V14	N	0	3.2	0.6	2	0.07	0.934579
	S	0	3.2	0.6	2	0.07	0.934579
V15	N	0	3.2	0.6	2	0.06	0.943396
	S	0	3.2	0.6	2	0.06	0.943396
V16	E	1	3.2	0.8	2	0.06	0.943396
	W	1	3.2	0.8	2	0.06	0.943396
W1	W	1	3.2	0.8	2	0.097	0.911577
	E	1	3.2	0.8	2	0.116	0.896057
W2	S	0	3.2	0.6	2	0.124	0.88968
	N	0	3.2	0.6	2	0.105	0.904977
W3	W	1	3.2	0.8	2	0.078	0.927644
	E	1	3.2	0.8	2	0.136	0.880282
W4	W	0	3.2	0.6	2	0.037	0.96432
	E	0	3.2	0.6	2	0.059	0.944287
W5	W	0	3.7	0.6	2	0.07	0.934579
W6	N	1	3.7	0.9	2	0.149	0.870322
	S	1	3.7	0.9	2	0.151	0.86881
W7	S	0	3.7	0.6	2	0.069	0.935454
	N	0	3.7	0.6	2	0.069	0.935454
W8	S	1	3.7	0.9	4	0.098	0.772798
	N	1	3.7	0.9	2	0.068	0.93633
W9	W	0	3.7	0.6	2	0.15	0.869565

C.2 Approach capacity and degree of saturation

Table C-2 Approach capacity and degree of saturation

Site ID	**Approach**	**Q (AM)**	**Q (PM)**	**X_T AM**	**X_T PM**
T1	S	1796.967	3276.072	0.653323	0.685272
T2	S	3734.371	3837.678	0.327766	0.456526
	N	3734.371	3837.678	0.578946	0.393988
T3	S	2783.681	3297.869	0.523048	0.633439
	N	2783.681	3297.869	0.884441	0.588562
T4	S	2167.336	2251.832	0.615964	0.804678
	N	2167.336	2251.832	1.072746	0.845978
T6	N	1323.796	999.814	0.270435	0.394073
V1	S	1633.445	1459.669	0.881572	0.836491
	N	1633.445	1459.669	0.615876	0.823474
	E	1399.971	1442.09	0.793588	0.834206
	W	1399.971	1442.09	0.808588	0.716321
V2	W	1740.566	1770.154	0.802612	0.815748
	E	1724.299	1753.611	1.023604	0.74646
V3	S	2800.581	2257.184	0.755558	0.532965
	N	2800.581	2800.581	0.385634	0.673789
V4	N	2589.623	2131.785	0.446011	0.676428
V5	S	2403.129	1778.208	0.489778	0.956581
V6	W	2658.462	1983.297	0.221181	0.673122
V7	S	1585.321	1504.023	0.48066	0.624326
	N	1614.953	1532.135	0.632836	0.535201
V8	S	1519.481	1041.322	0.379077	1.106286
	E	1229.919	1290.566	0.764278	0.465687
	W	1104.843	1159.322	0.472465	1.079942
V9	W	2297.436	2748.718	0.6825	0.361259
	E	2340.784	2800.581	0.286656	0.527748
	S	414.0906	662.5449	0.386389	0.235456
	N	421.978	675.1648	0.419453	0.530241

V10	E	1925.926	2118.519	0.758077	0.856731
V11	W	1738.349	2353.846	0.43317	0.257876
	E	1705.55	2309.434	0.331271	0.388407
V12	W	2561.725	2445.283	0.963413	1.046914
	E	2561.725	2445.283	1.015722	0.9549
V13	W	2600.539	2716.981	0.701008	0.75341
V14	N	1833.401	1471.66	0.631613	1.127978
	S	1833.401	1471.66	1.125231	1.140209
V15	N	3285.652	2589.623	0.420617	0.542164
	S	3285.652	2589.623	0.48027	0.456823
V16	E	3079.245	4588.679	0.480962	0.45111
	W	3079.245	4588.679	0.719007	0.421036
W1	W	1334.706	1337.573	0.651829	0.539784
	E	1311.983	1314.8	0.5183	0.734712
W2	S	640.5694	555.1601	0.423061	1.314936
	N	651.5837	564.7059	0.979153	0.515313
W3	W	736.997	827.5707	0.214384	0.50751
	E	699.3686	785.318	0.626279	0.276321
W4	W	1437.631	1445.299	0.177375	0.305819
	E	1407.765	1415.274	0.539863	0.233877
W5	W	746.7518	505.4206	0.303983	0.468916
W6	N	2678.851	2349.869	0.13028	0.31406
	S	1501.303	1313.64	0.448277	0.350172
W7	S	393.7815	477.5916	0.678041	0.598838
	N	393.7815	477.5916	0.507896	0.533929
W8	S	1930.69	1688.881	0.379139	0.301975
	N	1705.699	1475.215	0.29079	0.47993

C.3 ATL observed utilisation

Table C-3 Observed utilisation

Site ID	**Critical Lane AM**	**q_c AM (veh/hr)**	**Critical Lane PM**	**q_c PM (veh/hr)**	**q_{ATL} AM (veh/hr)**	**q_{ATL} PM (veh/hr)**	**p_{ATL} AM**	**p_{ATL} PM**
T1S	T(R)	591	T(R)	1112	68	101	0.115	0.091
T2S	T(R)	536	T(L)	904	162	106	0.302	0.117
T2N	T(R)	942	T(R)	641	392	237	0.416	0.370
T3S	T(L)	699	T(R)	1028	71	114	0.102	0.111
T3N	T(R)	1203	T(R)	918	216	140	0.180	0.153
T4S	T(R)	634	T(R)	922	136	101	0.215	0.110
T4N	T(R)	1128	T(L)	869	235	172	0.208	0.198
T5N	T	262	T	331	208	285	0.794	0.861
T6N	T	325	T	370	33	24	0.102	0.065
V1S	T(R)	604	T(L)	502	298	226	0.493	0.450
V1N	T(L)	452	T(L)	565	165	135	0.365	0.239
V1E	T(R)	548	T(R)	600	65	66	0.119	0.110
V1W	T(R)	591	T(R)	545	53	49	0.090	0.090
V2W	T(R)	462	T(R)	463	193	226	0.418	0.488
V2E	T(R)	576	T(R)	450	303	135	0.526	0.300
V3S	T(R)	907	T(R)	530	392	169	0.432	0.319
V3N	T(L)	435	T(R)	735	253	427	0.582	0.581
V4N	T(R)	584	T(R)	764	69	65	0.118	0.085
V5S	T(R)	383	T(R)	532	128	188	0.334	0.353
V6W	T(L)	296	T(R)	667	41	78	0.139	0.117
V7S	T(R)	356	T(L)	418	74	124	0.208	0.297

V7N	T(L)	442	T(L)	365	166	116	0.376	0.318
V8S	T(R)	252	T(R)	441	100	301	0.397	0.683
V8E	T(R)	405	T(R)	274	160	62	0.395	0.226
V8W	T(R)	253	T(L)	498	62	258	0.245	0.518
V9W	T(R)	675	T(R)	462	306	134	0.453	0.290
V9E	T(L)	299	T(R)	632	98	270	0.328	0.427
V9S	T	98	T	92	62	64	0.633	0.696
V9N	T	127	T	231	50	127	0.394	0.550
V10E	T(C)	495	T(R)	597	93	132	0.188	0.221
V11W	T(L)	372	T(L)	279	47	58	0.126	0.208
V11E	T(R)	300	T(R)	446	20	38	0.067	0.085
V12W	T(R)	786	T(L)	808	175	196	0.223	0.243
V12E	T(R)	846	T(R)	755	122	164	0.144	0.217
V13W	T(R)	498	T(R)	558	418	455	0.839	0.815
V14N	T(R)	448	T(R)	536	125	217	0.279	0.405
V14S	T(R)	647	T(C)	485	285	264	0.440	0.544
V15N	T(R)	492	T(C)	485	246	210	0.500	0.433
V15S	T(R)	584	T(R)	442	59	26	0.101	0.059
V16E	T(C)	537	T(L)	720	32	128	0.060	0.178
V16W	T(C)	784	T(C)	756	60	33	0.077	0.044
W1W	T	517	T	411	353	311	0.683	0.757
W1E	T	366	T	510	314	456	0.858	0.894
W2S	T	145	A	366	126	366	0.869	1.000
W2N	T	391	T	190	247	101	0.632	0.532
W3W	T	85	T	222	73	198	0.859	0.892

W3E	T	241	T	117	197	100	0.817	0.855
W4W	T	154	T	234	101	208	0.656	0.889
W4E	T	387	T	259	373	72	0.964	0.278
W5W	T	137	T	151	90	86	0.657	0.570
W6N	T	261	T	495	88	243	0.337	0.491
W6S	T	394	T	248	279	212	0.708	0.855
W7S	T	138	T	154	129	132	0.935	0.857
W7N	T	121	T	146	79	109	0.653	0.747
W8S	T	376	T	259	356	251	0.947	0.969
W8N	T	305	T	429	191	279	0.626	0.650
W9W	T	189	T	270	111	231	0.587	0.856

C.4 Multiple regression analysis

C.4.1 Model 1

Table C-4 Model 1 regression statistics

Regression Statistics	
Multiple R	0.634555331
R Square	0.402660468
Adjusted R Square	0.353395352
Standard Error	0.224470468
Observations	106

Table C-5 Model 1 ANOVA statistics

	df	*SS*	*MS*	*F*	*Significance F*
Regression	8	3.294639445	0.411829931	8.173338465	2.15105E-08
Residual	97	4.887538115	0.050386991		
Total	105	8.182177561			

Table C-6 Model 1 variable statistics

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	1.612315	0.402555	4.005201	0.000121	0.813354	2.411277
d1	-0.001086	0.001514	-0.717465	0.474811	-0.004091	0.001919
d3	-0.000725	0.001525	-0.475153	0.635746	-0.003751	0.002302
D	0.001527	0.001512	1.010490	0.314775	-0.001473	0.004527
c	-0.009054	0.003513	-2.577035	0.011471	-0.016027	-0.002081
G	0.010929	0.006094	1.793295	0.076042	-0.001167	0.023025
g/c	-1.750748	0.772355	-2.266766	0.025625	-3.283659	-0.217837
XT	0.158964	0.108121	1.470235	0.144734	-0.055627	0.373554
v	-0.002637	0.003093	-0.852486	0.396045	-0.008775	0.003502

C.4.2 Model 2

Table C-7 Model 2 regression statistics

Regression Statistics	
Multiple R	0.726803173
R Square	0.528242852
Adjusted R Square	0.484015619
Standard Error	0.200520176
Observations	106

Table C-8 Model 2 ANOVA statistics

	df	*SS*	*MS*	*F*	*Significance F*
Regression	9	4.322176807	0.480241867	11.94383686	1.92532E-12
Residual	96	3.860000753	0.040208341		
Total	105	8.182177561			

Table C-9 Model 2 variable statistics

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
CTLs	-0.211506	0.041839	-5.055228	0.000002	-0.294555	-0.128456
XT	0.279172	0.099469	2.806616	0.006063	0.081727	0.476616
Intercept	0.901081	0.386147	2.333517	0.021707	0.134585	1.667577
D	0.002495	0.001364	1.829421	0.070441	-0.000212	0.005202
c	-0.005629	0.003211	-1.753175	0.082764	-0.012003	0.000744
d1	-0.002291	0.001373	-1.668111	0.098552	-0.005017	0.000435
v	0.004955	0.003145	1.575815	0.118358	-0.001287	0.011198
d3	-0.001926	0.001383	-1.392548	0.166973	-0.004670	0.000819
g/c	-0.701550	0.720488	-0.973715	0.332645	-2.131707	0.728607
G	0.004319	0.005599	0.771361	0.442388	-0.006795	0.015433

C.4.3 Model 3

Table C-10 Model 3 regression statistics

Regression Statistics	
Multiple R	0.722112131
R Square	0.52144593
Adjusted R Square	0.487263497
Standard Error	0.19988809
Observations	106

Table C-11 Model 3 ANOVA statistics

	df	*SS*	*MS*	*F*	*Significance F*
Regression	7	4.266563188	0.609509027	15.25479247	2.35271E-13
Residual	98	3.915614372	0.039955249		
Total	105	8.182177561			

Table C-12 Model 3 variable statistics

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
CTLs	-0.228052	0.039272	-5.807045	0.000000	-0.305986	-0.150119
c	-0.003432	0.000845	-4.059048	0.000099	-0.005109	-0.001754
Intercept	0.610873	0.163302	3.740751	0.000309	0.286805	0.934941
XT	0.303703	0.090313	3.362779	0.001102	0.124480	0.482927
D	0.002751	0.001296	2.121803	0.036376	0.000178	0.005323
d1	-0.002556	0.001296	-1.972383	0.051384	-0.005129	0.000016
d3	-0.002227	0.001309	-1.700747	0.092162	-0.004825	0.000371
v	0.004480	0.003089	1.449983	0.150256	-0.001651	0.010610

C.4.4 Model 4

Table C-13 Model 4 regression statistics

Regression Statistics	
Multiple R	0.714967992
R Square	0.511179229
Adjusted R Square	0.481553728
Standard Error	0.200997973
Observations	106

Table C-14 Model 4 ANOVA statistics

	df	*SS*	*MS*	*F*	*Significance F*
Regression	6	4.18255922	0.697093203	17.25470314	1.44731E-13
Residual	99	3.99961834	0.040400185		
Total	105	8.182177561			

Table C-15 Model 4 variable statistics

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	0.785502	0.110898	7.083089	0.000000	0.565456	1.005549
CTLs	-0.196208	0.032737	-5.993379	0.000000	-0.261166	-0.131250
c	-0.002890	0.000763	-3.789588	0.000259	-0.004403	-0.001377
XT	0.304024	0.090814	3.347746	0.001153	0.123828	0.484219
D	0.002834	0.001302	2.175893	0.031943	0.000250	0.005418
d1	-0.002595	0.001303	-1.991337	0.049199	-0.005180	-0.000009
d3	-0.002253	0.001316	-1.711759	0.090072	-0.004866	0.000359

C.4.5 Model 5

Table C-16 Model 5 regression statistics

Regression Statistics	
Multiple R	0.705367749
R Square	0.497543661
Adjusted R Square	0.472420845
Standard Error	0.202760622
Observations	106

Table C-17 Model 5 ANOVA statistics

	df	*SS*	*MS*	*F*	*Significance F*
Regression	5	4.070990582	0.814198116	19.80445357	1.13211E-13
Residual	100	4.111186978	0.04111187		
Total	105	8.182177561			

Table C-18 Model 5 variable statistics

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	0.701425	0.158523	4.424754	0.000025	0.386920	1.015929
D	0.000344	0.000125	2.761742	0.006842	0.000097	0.000592
c	-0.003836	0.000837	-4.583382	0.000013	-0.005496	-0.002176
XT	0.339772	0.089223	3.808133	0.000242	0.162756	0.516787
v	0.004686	0.003132	1.496083	0.137781	-0.001528	0.010900
CTLs	-0.216076	0.039433	-5.479505	0.000000	-0.294310	-0.137841

C.4.6 Model 6

Table C-19 Model 6 regression statistics

Regression Statistics	
Multiple R	0.706725909
R Square	0.49946151
Adjusted R Square	0.469125844
Standard Error	0.203392808
Observations	106

Table C-20 Model 6 ANOVA statistics

	df	*SS*	*MS*	*F*	*Significance F*
Regression	6	4.086683	0.681114	16.4645	4.47E-13
Residual	99	4.095495	0.041369		
Total	105	8.182178			

Table C-21 Model 6 variable statistics

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*
Intercept	0.711779	0.158963	4.477633	0.000020	0.396362	1.027197
d1	0.000168	0.000180	0.935645	0.351733	-0.000188	0.000524
d3	0.000517	0.000209	2.478180	0.014897	0.000103	0.000931
c	-0.003796	0.000842	-4.505766	0.000018	-0.005467	-0.002124
XT	0.345800	0.089652	3.857142	0.000204	0.167911	0.523689
v	0.004769	0.003140	1.518573	0.132057	-0.001462	0.011000
CTLs	-0.219597	0.039754	-5.523895	0.000000	-0.298477	-0.140716

C.5 Model comparisons

C.5.1 Study developed model

Table C-22 Study developed model inputs

		V2W		V9S	
	Coefficients	AM	PM	AM	PM
CTLs	-0.219597	3	3	1	1
c	-0.003796	128	132	130	130
Intercept	0.711779	1	1	1	1
XT	0.345800	0.802612466	0.815748	0.386389	0.235455729
d1	0.000168	123	123	61	61
d3	0.000517	87	87	53	53
v	0.004769	80	80	70	70

Table C-23 Study developed model results

Site ID	Period	Predicted pATL	Residuals
V2W	AM	0.29188305	-0.12587
	PM	0.281243276	-0.20688
V9S	AM	0.503866859	-0.12879
	PM	0.45167418	-0.24398

C.5.2 Australian Road Capacity Guide method

Table C-24 Australian Road Capacity Guide model calculations and results

ID	s (veh/hr)	g (sec)	ft reqd.	d_1 (ft)	d_p	F	s_{ATL}	p_{ATL}	Residuals
V2W	1358	41	371	404	-	-		1	0.58225

AM									
V2W PM	1359	43	390	404	-	-		1	0.51187
V9S AM	1793	15	179	200	-	-		1	0.36734
V9S PM	1794	24	287	200	0.69676	0.03	1266	0.7058	0.01021

C.5.3 Australian Road Research method

Table C-25 Australian Road Research model calculations and results

ID	Period	n	q_{ATL}	q_C	p_{ATL}	Residuals
V2W	AM	4	0.125	0.2917	0.4286	0.0108
	PM	4	0.125	0.2917	0.4286	-0.0595
V9S	AM	2	0.25	0.7500	0.3333	-0.2993
	PM	2	0.25	0.7500	0.3333	-0.3623

C.5.4 SIDRA method

Table C-26 SIDRA model calculations and results

ID	Period	d_L (m)	d_{full} (m)	d_{min} (m)	Criteria?	p_{dmin}	p_{ATL}	Residuals
V2W	AM	135	200	30	dmin<dL<dfull	20	0.6487	0.2310
	PM	135	200	30	dmin<dL<dfull	20	0.6487	0.1606
V9S	AM	104	200	30	dmin<dL<dfull	20	0.4949	-0.1378
	PM	104	200	30	dmin<dL<dfull	20	0.4949	-0.2008

C.5.5 Transport Research Board method

Table C-27 Transport Research Board model calculations and results

ID	Period	X_T	q_T (veh/hr)	q_{ATL} (veh/hr)	CTLs (no.)	q_c (veh/hr)	p_{ATL}	Residuals
V2W	AM	0.8026	1397	394.930	3	334.023	1.1823	0.7646
	PM	0.8157	1444	418.701	3	341.766	1.2251	0.7370
V9S	AM	0.3864	160	36.661	1	123.339	0.2972	-0.3354
	PM	0.2355	156	28.776	1	127.224	0.2262	-0.4695