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The effect of left-turning traffic on the performance of through traffic at a signalized intersection: case study of Anloga intersection in Kumasi, Ghana

By

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DECLARATION

I hereby declare that this submission is my own work towards the MSc and that, to the best of my knowledge, it contains no material previously published by another person nor material which has been accepted for the award of any other degree of the University, except where due acknowledgement has been made in the text.

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ABSTRACT

Rapid vehicular population growth in urban centres in developing countries such as Ghana has resulted in increased volumes contributing to long delays at signalized intersections. At intersections where there is high volume of left-turning traffic, spill back and blocking of the through traffic has much effect on saturation flows. In many situations, a change of layout of intersection or even grade separation may be the option for improving delay and LOS but uneconomical to most developing countries due to lack of funds or inadequate right-of-way or both. The objective of this thesis was to evaluate the effect of left turning traffic on delay to through traffic at signalized intersection, evaluate the effect of left turning traffic on saturation flow for through traffic at a signalized intersection and propose a left-turn storage lane length that will improve the saturation flow and reduce delay for through traffic at a signalized intersectionusing micro simulation tool (VISSIM).An intersection characterized by long queue and delays during peak periods was chosen as the study site. Two hours video data collection was undertaken on a typical morning peak from which the traffic demand and turning movements were extracted. VISSIM was used for simulation analysis of the intersection. The model was calibrated with the traffic flow, queue length and delay data for the approaches and the result validated. After conducting a sensitivity analysis by increasing the approach volume from 1000pcu/h to 3000pcu/h and varying the left turn traffic percentage and left turn storage lane lengths, it was revealed that as the approach volume of signalizes intersection increases with a corresponding increase in left-turning traffic volume proportions, delay to through traffic movement increases and its saturation flow reduces. It was also found out that, for approach volumes of 1000pcu/h to 3000pcu/h and corresponding left turn traffic proportion 10% to 20%, 150m optimal storage lane is required to reduce delay and improve on saturation flow. For an approach volume of 1000pcu/h to 3000pcu/h with

30% to 50% left turn traffic proportion, 175m storage lane is required to reduce delay and improve on saturation.

KEYWORDS: Saturation flow, delay, signalized intersection, left-turning traffic, storage lane, micro simulation.

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DEDICATION

This thesis is dedicated to my beloved wife, Margaret Agyeiwaa Boateng and children, Rebecca Nana Akua Antwi and Antwi k. Francis Jnr. for their endless love and encouragement. They have been the source of my strength in difficult times throughout my graduate studies.

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LIST OF ABBREVIATIONS

ANOVA	-	Analysis of Variance
DVLA	-	Driver Vehicle and Licensing Authority
FHWA	-	Federal Highway Authority (2004)
GEH	-	Geoffrey E. Havers
HCM	-	Highway Capacity Manual (2000)
KNUST	-	Kwame Nkrumah University of Science and Technology
LOS	-	Level of Service
PCE	-	Passenger Car Equivalent
PCU	-	Passenger Car Unit
PHF	-	Peak Hour Factor
PHV	-	Peak Hour Volume
WB	-	West Bound
WBT	-	West Bound Through
WBL	-	West Bound Left
WBR	-	West Bound Right

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Rise in the mobility demand of traffic within urban and suburban areas has resulted in rapid increase in growth rate of traffic in Ghana. A report published by the Driver Vehicle and Licensing Authority (DVLA) in 2012 indicates that the vehicle population in Ghana is close to a million, with much concentration in the two biggest cities, Accra and Kumasi. Research has shown that traffic movement within the streets of these cities is therefore becoming critical. One of these streets is the Osei Tutu II Boulevard in Kumasi.

It is an east-west major arterial of about 5.4 km length from KNUST junction to the Asafo Interchange traffic light (Nyantakyi, et al, 2013). The road is a 2-lane dual carriage and serves as the main route that leads into the Kumasi metropolis from the southern parts of Ghana. There are three signalized intersections on the road, namely, Anloga, Stadium and Amakom signalized intersections, (Nyantakyi, et al, 2013). There is a high demand at these intersections with regards to capacity especially during the peak periods making them fail in operation as traffic overflow normally occur.

Research conducted by Nyantakyi, et al, (2013), indicated that these intersections exhibit similar traffic characteristics and would reach critical traffic condition in five years.

1.1.1 Intersections

Intersection is any place where two or more roads intersect or meet each other at grade. A highway intersection is required to control conflicting and merging streams of traffic so that delay is minimized and this is achieved through choice of geometric characteristics and regulation that control and regulate the vehicle paths through the intersection. These determine priority so that all movements take place with safety.

1.1.2 Types of intersection

The three main types of intersections are:

1. Priority intersections, either simple T-junctions, staggered T-junctions or crossroads
2. Signalized intersections
3. Roundabouts.

All these are aimed at providing vehicle drivers with a road layout that will minimize confusion. The need for flexibility dictates the choice of most suitable junction type. The selection process requires the economic, environmental and operational effects of each proposed option to be evaluated.

The designer needs to assess the projected traffic flow at the location in question, termed the design reference flow. The range within which this figure falls will indicate a junction design which is both economically and operationally efficient rather than one where there is either gross over or under provision. Test should be conducted on different combinations of turning movements in order to check the performance characteristics of each junction option under consideration. The volume of traffic incident on the intersection together with the various turning, merging and conflicting movements involved are the main determinants of the intersection design.

1.1.3 Left- turn lane

A direct left-turn egress from a driveway must yield to all other movements since it has the lowest priority and it is the most likely movement to be delayed, especially under heavy traffic volume conditions, (Mahmoud Ahmed Taha, 2015). Left turns are a significant hindrance to the smooth flow of traffic in networks involving at-grade intersections. Therefore, separate turn phases are often used on the approach legs of the signalized intersections with heavy left turns, but this wastes capacity on the approach because some of its lanes cannot discharge during its green phases since left turns are a significant hindrance to the smooth flow of traffic in networks involving at-grade intersections, (Yiguang, et al., 2010). Left-turn lanes are therefore incorporated to separate the left turn traffic from the through traffic and this can lead to an improvement in the signalized intersection capacity.

1.2 PROBLEM STATEMENT

In Ghana, most urban areas such as Accra and Kumasi are constantly experiencing traffic growth. This development always results in undue delays at signalized intersections because of spillback of left-turning traffic blocking the movement of through traffic. The intersection capacity would be reduced due to inefficient utilization of the through movement green time, and the vehicle delay would increase because the blocked through vehicles spent more time in the traffic queues. Fuel consumption and emissions would also be increased since the queued vehicles would spend more idle time in the queues

1.3 PROJECT OBJECTIVES

The objectives of the project are to:

- Evaluate the effect of left turning traffic on delay to through traffic at signalized intersection.
- Evaluate the effect of left turning traffic on saturation flow for through traffic at a signalized intersection.
- Propose a left-turn lane length that will improve the saturation flow and reduce delay for through traffic at a signalized intersection.

1.4 JUSTIFICATION

To reduce traffic delays, most research by road and transportation engineers have suggested solutions such as addition of more lanes, grade separation (interchange), etc. However, such solutions are not always feasible for many at grade signalized intersections as a result of limited right-of-way or budgetary constraints. One of the possible solutions to reduce the overall delay and improve capacity of a signalized intersection is using exclusive left-turn lane which separates left-turning traffic from the through traffic.

The research has become necessary because of the following reasons:

- Most of the arterial roads in the urban areas are connected by signalized intersections therefore their effectiveness in terms of saturation flow, capacity and level of service is of great concern especially as the proportion of left turner increase.
- Ghana has on its drawing board designs to develop some of these intersections into interchanges at a very high cost. Since there are usually budgetary constraints, a better understanding of the effect of left turners will help to improve performance in the medium term.
- The cost in implementing appropriate left turning storage lane to improve the performance of the signalized intersection is generally lower than changing the entire intersection design if left turning traffic is the main problem affecting the measures of effectiveness.

1.5 SCOPE OF WORK

The study was conducted to evaluate the effect of left turn traffic on the performance of through traffic for a signalized intersection. The signalized intersection’s performance indicators that were considered under this study were delay and saturation flow to through traffic.

- *General purpose*: To find a simple inexpensive means other than an interchange and addition of more lanes for the reduction of delay and improving saturation flow.
- *Subject matter*: Effect of left turn traffic on the performance of through traffic; traffic delays and saturation flow of the through traffic were evaluated based on increase in

approach volume and increase in left turn traffic percentage with variation of the left turn storage lane length.

- *Locale of the study*: Anloga Intersection in Kumasi, Ghana.
- *Period of the study*. The data collection was undertaken on15th March, 2016.

CHAPTER 2: LITERATURE REVIEW

2.1 INTERSECTION

Intersection is of greatest importance in highway design because of their effect on the movements and safety of vehicular traffic flow.

2.2 OPERATIONAL PERFORMANCE

To assess the operational performance of an intersection, the following parameters must be born in mind (HCM, 2000):

2.2.1 Geometric conditions: The geometry of a signalized intersection is normally in the form of a diagram, comprising of the number of lanes, width of lanes, exclusive left or right turn lane if it exist, parking facility etc. (HCM, 2000).

A study conduct by Nyantakyi et al., (2013) revealed that geometric improvement such as addition of one lane and two lanes would lead to improvement in the level of service, reduce delay and subsequently increase in capacity, however, there are instances where due to limited space additional lanes cannot be the option.

2.2.2 Traffic conditions: For each movement on each approach the traffic volumes (over saturated condition, demand) must be specified. According to the HCM, vehicle type distribution must be quantified as a percentage of heavy vehicles in each movement for the various approaches. To complete the operational analysis, the vehicle arrival time type 1, 2…6 must be computed. This is a parameter that shows the quality of progression for each lane group, i.e. the density of a platoon that contains a certain percentage of a volume of traffic for a lane group, arriving at the start, throughout and in the middle of a red or a green phase or exceptional progressions (HCM, 2000).

2.2.3 Signalization conditions: Every information about the signalization is needed for the analysis, such as the phase plan, the circle length, the green time, and-change-and clearance interval (HCM, 2000).

2.3 LEVEL OF SERVICE

LOS is a qualitative measure of the highway's operating conditions under a given demand within a traffic stream and their perception by motorists and/or passengers (HCM, 2000).

The parameters selected to define LOS which is also called measures of effectiveness (MOE) are travel times, speeds, total delay, probability of delay, comfort, and safety, (HCM, 2000).

The overall LOS of the road system is significantly affected by the level of service (LOS) at intersections (Vijayendra et al., 2004). The LOS at an intersection can also be affected by frequently allowing left-turning vehicles to block through traffic *(*Vijayendra et al., 2004), it is therefore more important to separate left-turning vehicles from through traffic for an effective traffic operation of intersections (Vijayendra et al., 2004). According to Zhao Yang et al, the capacity of left-turning lanes depend largely on, a)start up lost time of the first four vehicles behind the stop line, b)saturation headway and c)queue formation and discharge pattern.

2.4 LEFT TURNING LANES

Left turns are a significant hindrance to the smooth flow of traffic in networks involving at-grade intersections (Yiguanget al., 2010). If left turns are significant in number, separate left turn phases have to be introduced at signalized intersections to handle the flow (Yiguang et al., 2010). Unfortunately according to Yiguang et al., separate left turn phases reduce the fraction of time that the signal can devote to through movements, thereby reducing capacity and this reduction can increase delay if through traffic is heavy.

According to the HCM, when an approach with more than one lane includes a lane that may be used by left turning vehicles and through vehicles, existence of equilibrium condition must be determined or the lane must act as an exclusive left turn lane (de facto left-turn lane) when there are so many left-turning vehicles. This can be identified when the proportion of left-turning vehicle is 1.0 (i.e., 100%) of the shared lane (HCM, 2000).

According to IHRB (Iowa Highway Research Board), left turning lane is an auxiliary lane for storing left turning vehicle, thus clearing the way for through traffic.

Zhao, et al., (2012), studied the operational impact of left-turn waiting areas at signalized intersections. The arrival and departure processes of left-turning vehicles at exclusive left-turn lanes with and without waiting areas by using Cross-sectional analysis data collected from 12 approaches at nine signalized intersections to compare the start-up lost time and saturation headways of left-turn passenger cars for four scenarios**.** The results indicated that left-turn waiting areas increase the capacity of exclusive left-turn lanes and that the capacity gains would increase with an increase in the storage capacity of the left-turn waiting area.

Vijayendra, et al., (2004) on the other hand proposed new left-turn guidelines for both unsignalized and signalized intersections on the basis of well-validated event-based simulation programs. Sites that had large opposing volumes and large delays for left-turning traffic were selected and data collected with STV (state-of the –art-van) for two hours. In the data reduction process Vijayendra, et al., (2004) extracted the following data: *Geometric and Traffic Data* inputted in the event-based simulation program developed.

- number of approaches
- number of lanes on each approach
- turn attribute of each lane (left only, right only, etc.)
- volumes on each approach

- percentage of turns on each approach
- operating speed

Critical Gap Data The critical gap of the drivers plays an important role in the delays experienced by the left-turning vehicles and therefore will determine whether the shared lane will be blocked or not. This dictates the need for the separate left-turn lane. This is to examine left-turn behaviour.

Measures of Effectiveness Data this validated the event-based simulation program by comparing the simulated and the one seen on the field. The MOE data included the queue length and delay of left turning vehicles.

The study suggested that separate lane for left turners on the major street do not guarantee that the through traffic is completely unimpeded. There might be cases where the queue of left turners is exceeding the length of the left turning pocket. The study therefore suggested that a method is needed to quantify this effect on the intersection capacity. But Pei-Sung Lin (2004) focused on the storage blockage problem for left-turn lanes at signalized intersections and came out with potential strategies to alleviate traffic congestion and safety problems caused by inadequate left-turn lane storage lengths at signalized intersections. The study used and compared the following options: a. Apply a lead-lag left-turn operation for a coordinated signal, b. Use split phasing c. Provide a better signal coordination, d. Use a shorter cycle length and longer left-turn green time, e. Apply fixed green times for specific movements in Central Business District areas f. Increase left-turn queue storage length**.**

The study concluded that when left-turn lanes carrying heavy traffic volumes are inadequate in length, and the signalized intersection is near or exceeds its capacity, transportation professionals can consider to reduce left-turn traffic demand by providing an advanced left-turn lane before the signalized intersection, or eliminate the left-turn phases with proper diversion of left-turn traffic to its intended destination. Asaithambi & Ramaswamy (2008) however presented the development of a simulation model to imitate the flow of heterogeneous traffic through a signalized intersection. Their model examined the effects of left turn channelization on vehicle waiting times and after conducting a sensitivity analysis it was estimated that vehicle waiting times were reduced if a channelization was provided for a high traffic volume and certain proportions of left turn vehicles in the intersection approach. The length of channelization therefore has marginal impacts on vehicle waiting, (Asaithambi & Ramaswamy, 2008). Again, Haddad and Geroliminis, (2012) model the effects of queue spillbacks at the left-turn bay on the capacity of the intersection. Their Case study examples examine the effect of intersection characteristics (e.g. left-turn percentage, green durations, and storage capacity) on the arterial capacity drop. They develop a new probabilistic model to address the queue spillback effects on the capacity of the intersection. They concluded that queue spillback affects the main flow to move smoothly through all intersections in the arterial, thus decreasing the bandwidth and the capacity and increasing delays. One can prevent queue spillback by preventing the left-turn at intersections with short left-turn bay and limited capacity storage (Haddad and Geroliminis, 2012).

Meanwhile Yiguang, et al., (2010) said Separate turn phases are often used on the approach legs to intersections with heavy left turns wastes capacity on the approach because some of its lanes cannot discharge during its green phases. They therefore proposed that this problem can be eliminated by reorganizing traffic on all the lanes upstream of an intersection using a mid-block pre-signal. The study stated that if drivers behave deterministically, the capacity that can be achieved is the same as if there were no left turns, however, the reorganization is so drastic that it may be counterintuitive to drivers. This can be remedied by reorganizing traffic on fewer lanes (Yiguang, et al., 2010). Furthermore, Ryan,Wen Cheng &Loera-

Gutierrez, evaluate the performance of dual left-turn phases using the VISSIM microscopic traffic simulation. They collected real-world traffic data for 14 signalized intersections during morning and afternoon peak periods and simulated based on the field data repeatedly for different cycle lengths and varied phase lengths for both single and dual left-turn phases in a cycle and relative performance gauged against MOEs. Ryan,Wen Cheng & Loera-Gutierrezconcluded that dual left-turn phases in a cycle can be considered as an effective method for improving the efficiency of signalized intersections in some situations.

Also, Peng, Hideki and Miho (2011), investigated saturation flow rate analysis for shared left-turn lane at signalized intersections in Japan. The study compared methods used by Capacity Manual 2000 (HCM, 2000) and Japan Society of Traffic Engineering (JSTE) to estimate the performed saturation flow rate in shared left-turn lane and established that both methods overestimated. The study therefore used binary logistic regression model and it was found out that under the same pedestrian and bicycle volume, the lane blockage probability increases with the rising left-turn proportion. it stated that the trend becomes more pronounced when left-turn proportion exceeding 0.5 and in an extreme cases, when left-turn proportion equals to 1, the shared lane becomes a de facto left-turn lane which result to even minimal pedestrian volume would causing so high a lane blockage probability exceeding 0.5 (Peng, Hideki and Miho, 2011).

Levinson (2004) also studied the capacity concepts for access management. Several capacity concepts that respond to this need and that reflect the uncertainties and variations of future traffic estimates were studied and the results compared with Australian, Canadian and HCM procedures for typical volume conditions. It shows that shared lanes are typically about 40 to 60per cent as effective as through lanes.

Gowri and Sivanadan (2008) used microscopic simulation to study heterogeneous traffic flow at a signalized intersection and the efficacy of the left turn lanes. He concluded that left turn lanes gives the highest reduction in total waiting times of vehicles for a total volume level of 3000 vh/h and left turn volume of 50%. They stated that channelized left turn lanes showed positive impacts for a higher proportion of left turn volumes and higher volums of total traffic volumes, however, for low left turning volume left turn lanes could be detrimental.

2.4.1 CAPACITY OF LEFT TURNING LANES

Capacity of a facility is the maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic, and control conditions (Highway Capacity Manual, 2000).

Capacity is the main determinant of the performance measures such as delay, queue length and stop rate and their relationship are often expressed in terms of degree of saturation (Akcelic, 2005).According to Zhao, et al, left-turn lanes without waiting areas starts to form a queue after initiation of the red signal and will be discharged during the lagging left-turn phase. The left turn queue length reaches the maximum value ($q$max2) at time t1 when the left-turn green arrow time is initiated (Zhao et al, 2012).

Zhao,et al. modelled the capacity of exclusive left-turn lanes at signalized intersections without waiting areas as $C_1 = 3{,}600/h_1$. $GY\text{-}L_{si}\text{-}L_c/T$

where

$C1$ = capacity of exclusive left-turn lanes at signalized intersections without left-turn waiting areas (passenger car unitper hour per lane),

3,600 = seconds in 1 h,

T = cycle length of target intersection (s),

G = green time for left-turn phase (s),

Y = yellow interval (s),

$Ls1$ = start-up lost time of left-turn movement (s),

Lc= clearance lost time (s), and

$h1$ = saturation headway for left-turning passenger cars (s)

2.4.2 SATURATION FLOW RATE

Saturation flow rate as defined by HCM "is the flow in vehicle per hour that can be accommodated by a lane group assuming that the green phases were displayed 100 percent of the time.

This is given by: $s = So\ N f_W f_{HV} f_g f_{bp} f_{bb}\ f_a f_{LU} f_{Lt} f_{Rt} f_{LPb} f_{RPb}$

Where:

S = saturation flow rate for subject lane group, expressed as the total for all lanes in lane group (VEH/H);

So =Base saturation flow per lane (pc/h/l);

N = number of lanes in a lane group;

f_W = adjustment factor for lane width;

f_{HV} = adjustment factor for heavy vehicle in the traffic stream;

f_g = adjustment factor for approach grade;

f_p = adjustment factor for the existence of parking lane and parking activities adjacent to the lane group;

f_{bb} = adjustment factor for blocking effect of buses that stop within the intersection area;

f_a = adjustment factor for area type;

f_{LU} = adjustment factor for lane utilization;

f_{Lt} = adjustment factor for left-turns in the lane group;

f_{Rt} = adjustment factor for right-turns in the lane group;

f_{LPb} = pedestrians adjustment factor for left-turn movements;

f_{RPb} = pedestrians-bicycle adjustment factor for right-turn movements (HCM, 2000).

Similarly the saturation flow rate at a signalized intersection can also be given as; s=3600/h

Where s = saturation flow rate (veh/h)

h = saturation headway (s)

2.5 SATURATION HEADWAY AND START-UP LOST TIME

The discharge headway method has been widely used for estimating the saturation headways and start-up lost time for a lane or a lane group at a signalized intersection (Zhao, et al, 2012). According to the HCM, the discharge headways between the first few successive vehicles are initially large. Immediately after the fourth vehicle passes the stop line, the headways will converge to a relatively constant state (Zhao, et al., 2012). The constant headway is defined as the saturation headway, which can be measured in the field by averaging the discharge headways after the fourth or fifth discharged vehicle (Zhao, et al, 2012). The start-up lost time includes the reaction time of the first driver responding to the change in signal phase and the acceleration of the first few vehicles before a saturation state is achieved (Zhao, et al., 2012). The start-up lost time is usually estimated with the total elapsed time for the first four vehicles and the saturation headway (Highway Capacity Manual, 2000). According to HCM, the start-up lost time is given

$L s = T4 - 4 \times h$

where

Ls=start-up lost time (s),

*T*4 =elapsed time from initiation of green signal until fourth

vehicle crosses stop bar (s), and

h=saturation headway (s).

2.6 DELAYS

Delays at intersections are the major contributing factor to arterial delays and to estimate it requires sampled travel times between two consecutive locations on arterial streets, one upstream and the other downstream of a signalized intersection, without the need to know signal timing or traffic flow information (Xuegang et al., 2009). for a signalized intersection, a combination with heavy left turning ratios (30, 60, 10) being the percentage of left turning, through and right turning traffic volumes respectively performed the worst giving relatively higher delays (Prakash et al.,2014).

According to the HCM, delay can be estimated thus: d2 = 900T[(x-1) + √(x-1)2 + 8klx/CT]

Where

d2=incremental delay to account for effect of random and oversaturation queues, adjusted for duration of the analysis period type of signal control (s/veh); this delay assumes that there is no initial queue for the lane group at start of analysis period;

T= duration of the analysis period (h);

K= incremental delay factor that is dependent on control settings;

l = upstream filtering / metering adjustment factor;

c= lane group capacity;

X= lane group v/c ratio or degree of saturation.

Thorsten et al, established that there are two different types of delay which are mostly related to the yellow and orange areas in Figure 2 below. On the one hand, that is the waiting time emerging in front of the traffic light because of a red signal which may vary for each turning direction. It is denoted by $d^{(1)}_{SLR}$in the following depending on whether the corresponding vehicle is driving straight on (*S*), turning left (*L*) or turning right (*R*). On the other hand $d^{(2)}_{SLR}$, describes the additional delay for turning vehicles when waiting on the intersection for oncoming traffic and/or pedestrians .

That is, the travel time Δt of a vehicle passing the intersection is given by the superposition of free flow travel time and these two types of delay. With regard to given GPS data from a common floating car system, for example, the measured travel time Δt between two data points of the same vehicle – one of them upstream, the other one downstream the intersection with known distance Δx – can be written as

$$\Delta t = \frac{\Delta x}{v} + d^{(1)}_{SLR} + d^{(2)}_{SLR},$$

where v is the average free flow speed of the considered vehicle (Thorsten, et al, 2010). In other words, one obtains a linear model for the travel times provided by the floating car system. Hence, assuming a constant network resistance (i.e. a constant free flow speed v) for all vehicles, it is easy to compute $1/v$ and the average total delay

$$d_{SLR} = d^{(1)}_{SLR} + d^{(2)}_{SLR}$$

for each turning direction separately by linear regression (Thorsten, et al., 2010).

2.7 MICRO SIMULATION

Micro simulation is the modeling of individual vehicle movements on a second or sub second basis for the purpose of assessing the traffic performance of highway and street systems, transit, and pedestrians (FHWA, 2004).

The overall process for developing and applying a micro simulation model to a specific traffic analysis problem consists of Identification of Study Purpose, Scope, and Approach, Data Collection and Preparation, Base Model Development, Error Checking, Calibration, Alternatives Analysis, Final Report and Technical Documentation (FHWA, 2004).

To better match specific local conditions micro simulation software program has a set of user-adjustable parameters that enable the practitioner to calibrate the software. In this study, VISSIM 7.0 version is chosen as the simulation tool. VISSIM was developed by Planning

Transport Verkehr (PTV) and the model presents vehicles interactions within the network based on “psycho-physical” driver behaviour where the speed of the vehicle depends on the driver’s perception of the preceding vehicle’s speed (Chatterjee, 2008).

2.7.1 CALIBRATION

In order to mimic the real-world situation models must be calibrated.

According to HTE as cited by Middleton and Cooner (1999) model calibration is the process of quantifying model parameters using real-world data in the model logic so that the model can realistically represent the traffic environment being analyzed.

The calibration process accounts for the impact of “unmodeled” site-specific factors through the adjustment of the calibration parameters included in the software for a specific purpose (FHWA, 2004).

According to HTE, user experimental conditions from simulation results are compared with observed data from the real network (fields’ data). If the error between simulation results and the observed data is small enough then it can be said to be accurate. Thus, model parameters should be optimized to match (possibly site-specific) observed settings. However, finding the model parameters requires a decision on a data set and a decision on an objective function that can quantify the closeness of the simulation to observed data set. Generally, calibration uses optimization techniques, such as generalized least square, to minimize the deviation between observed and corresponding simulated measurements. Sensitivity analysis should be conducted to test the model robustness as well as to study the impacts of changes in the model parameters.

The literature review has revealed that though there have been several researches into left-turn lanes and how it can improve capacity of a signalized intersection, however, none of them have been able to come out with how long the left-turn lanes should be given a particular traffic condition in Africa or Ghana for that matter.

CHAPTER 3: METHODOLOGY

3.1 Data Collection

3.1.1 Site selection and Visit

Anloga intersection in Kumasi was selected because of the level of congestion and delays experienced by drivers during morning, afternoon and evening peak periods. Prior to the day of the data collection, the site was visited for observation of challenges that may be confronted on the day of the data collection and the way out. Locations of video cameras were identified and queue counters marked. Fig 3.1 shows the geometry of Anloga intersection with the left-turning storage lanes.

Left-Turn Lane

Figure 3.1 Geometry of Anloga Intersection. Source: Google earth.

3.1.2 Videotaping of Traffic at Anloga Intersection

The Anloga intersection was filmed for an hour, from 07:00 a.m to 8:00 a.m on Tuesday, the 8th of March, 2016 during the morning peak conditions and again on Saturday, 12th March, 2016 same time for same duration. A digital camera was mounted on a tripod stand on top of a building at Anloga intersection to record traffic volumes and signal head. This positioning of the camera could capture the entire intersection.

3.1.3 Calibration Data

3.1.3.1 Queue Measurement

For calibration and validation of the simulation software (VISSIM) queue length of the traffic was a major parameter. Prior to the day of the data collection, the research team visited the site and marked queue counters at ten meters interval. The measurement was conducted by recording the length of the queue from the queue counters every 30 seconds.

3.1.3.2 Travel time and Delay measurement

During the period of the study (peak period) vehicle travel time study was conducted by timing a vehicle in the traffic stream without the driver’s knowledge from a reference point of known distance till it crosses the stop line. This was done by two (2) survey assistants with synchronized stop watch. At every red signal the survey assistant at the upstream identified and timed the car at the reference point and communicated the number plate, the colour and time of departure to the survey assistant downstream who also recorded the time the vehicle crossed the stop line.

During off peak period, a study vehicle traversed the same distance several times from the same reference point and the time of travel recorded. The average delay was found to be 161 seconds.

3.1.4 Intersection Geometry Measurement

This was done with the aid of a measuring tapes. The width of each lane on each approach was measured. The storage lane length was also measured. This measurement took place during off peak period when the road was virtually with few vehicles.

3.1.5 Evaluation of Signal phasing and timing at Anloga intersection.

The Cycle length is the time required for one complete sequence of signal phases measured in seconds. Cycle lengths and phases for the approaches were recorded using stopwatch and the cycle length was found to be 205 seconds.

3.3 Intersection Simulation

A microscopic, time step and behaviour-based simulation tool, VISSIM 7.0 was used to analyse the traffic data.

3.3.1 Model building of the intersection in VISSIM 7.0

Satellite image of the intersection from Google earth was imported into VISSIM software and scaled. VISSIM uses a tool called 'Link' to trace the outline of the roadway from the satellite image and connect the directional movement from one approach to the other with a tool called 'Connecters'. Additionally data on traffic controls and link operations are then inputted. Travel demand and traveller behaviour data were then added to the basic network.

3.3.2 Error Checking

As stated by the FHWA Guidelines, the error-checking task was conducted to identify and correct model coding errors so that they do not interfere with the model calibration task. Coding errors can distort the model calibration process and bring out incorrect values for the calibration parameters (FHWA Guidelines, 2004).

3.3.3 Model calibration

To better match the field local conditions, the model was calibrated with selected parameters such as queue length, delay and saturation flow. These data were obtained from the field.

3.4 Running the model

The simulation run was performed for 10 minutes for the existing condition. Because the simulation starts with zero vehicles within the network and takes time to fill the network completely with vehicles, a warm up period of 5 minutes was included otherwise the data collected before saturation is attained will not reflect the field condition. Data collected within the first 5 minutes out of simulation was excluded from the analysis. Overall, 10 runs were simulated with different random seed numbers and the average measured data such as delay, queue lengths, travel time and headway were recorded.

3.4.1 Experiment design

3.4.1.1 Scenario one (existing condition): the model would be run for the existing condition with the existing left turn volume and the existing left turn storage lane. We will then look at the effect of the existing left turn traffic volume and the existing left turn storage length on performance indicators such as delays, saturation flow, capacity and level of service.

3.4.1.2 Scenario two: in the second scenario of the experiment, the Approach traffic volume shall be varied from 1000pcu/h to 3000pcu/h in increments of 500 whilst the left turn traffic volume is varied from 10% to 50% in increments of 10%. We will then look at the performance indicators, i.e. delays, saturation flow.

3.5 Processing the data

For the purposes of verifying whether the model was adequately calibrated, average saturation flow and average delay were computed for the simulation and the field condition using Microsoft excel for a statistics test to be carried out using GEH and ANOVA statistics to compare the match between the simulated and the field condition. For hourly throughput volumes, the GEH formula is given by:

$$\text{GEH} = \sqrt{\frac{2(m-c)^2}{m+c}}$$

where, m = output traffic throughput volumes from the simulation model (veh/h)

c = traffic throughput volumes based on field data (veh/h). Table 3.1 gives a summary of match between the simulated and the field data.

Table 3.1 GEH Statistics match.

GEH Statistic	Criteria
<5.0	A good match
5.0 to 10.0	May warrant investigation
>10.0	Possible model error bad data

Source: Colon and MacCarthy, 2010.

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Queue length data

The queue measurement was done throughout the study period and the average queue length found to be 510.9 m and the maximum queue length was 600 m for the KNUST approach. The fig. below shows the trend in queue formation and dissipation with respect to time.

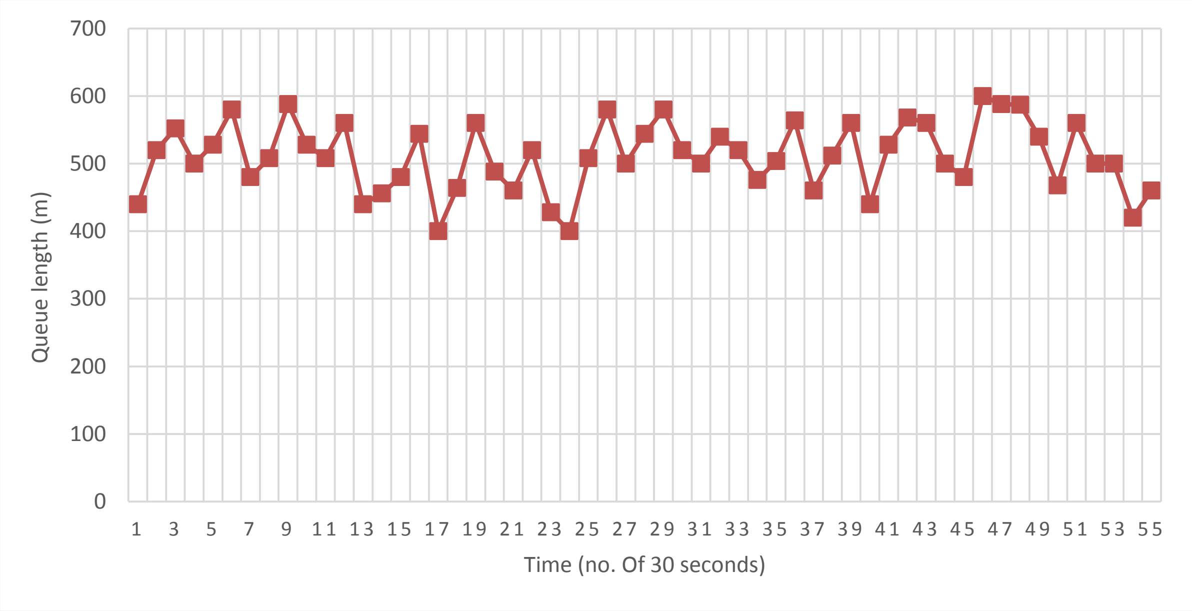

Figure 4.1 Queue formation and dissipation of queue with time

4.2 Intersection Geometry

Anloga intersection has four (4) approaches. The KNUST approach, the Airport approach, the Adum approach and the Asokwa approach. All the approaches except Asokwa approach have an exclusive left turning lane. Table 3.1 shows a summary of the intersection geometry

Table 4.1 summary of the intersection geometry

APPROACHES	LEFT TURN STORAGE (m)	RIGHT TURN STORAGE (m)	STORAGE LANE WIDTH (m)	THROUGH LANE WIDTH (m)
KNUST	75	75	3	3.5
ADUM	100	100	3	3.5
AIRPORT	75	SHARED	3.5	3.5
ASOKWA	continues		3.5	3.5

Source: from study

4.3 Evaluation of the Signal Phasing

The through movement from west and eat bounds were denoted with (1) and have 70 seconds green time, 4 seconds amber and 131 red time. Table two (2) gives the summary of the phase sequence of the four (4) approaches and fig 4.2 and 4.3 shows the traffic turning movement and signal time allocation of the intersection.

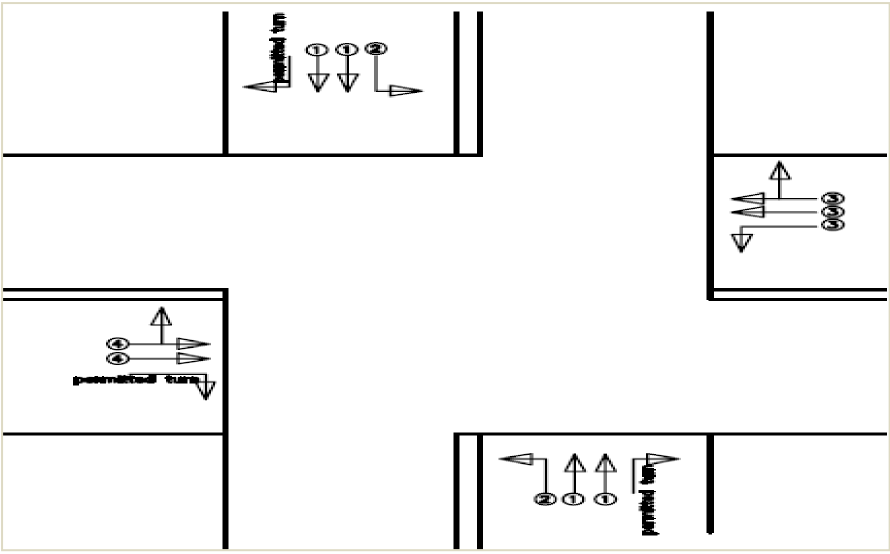

Figure 4.2 traffic turning movement. Source: from study.

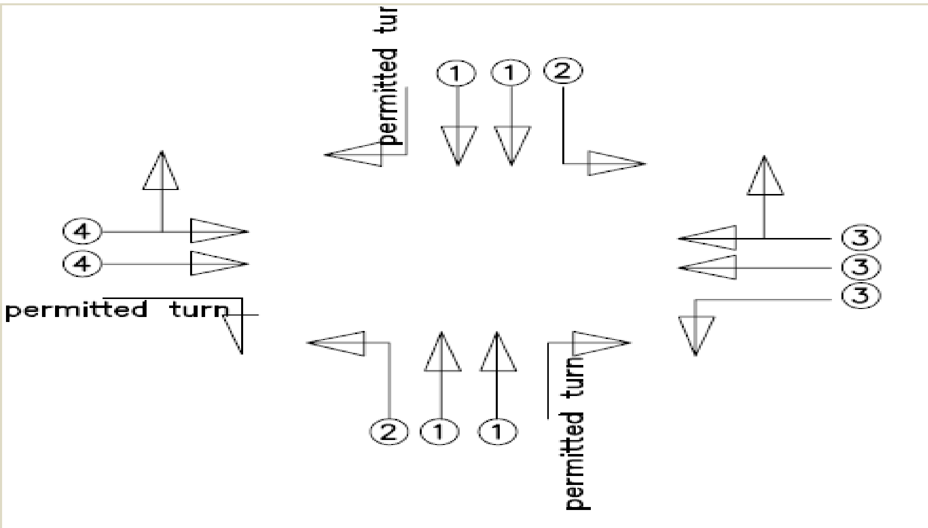

Figure 4.3 signal timing allocation. Source: from the study

Table 4.2: signal phasing plan

cycle length sec.	Green time (1)	green time (2) sec.	green time (3) sec.	green time (4) sec.
205	70	35	45	45
	Amber (1)	Amber (2)	Amber (3)	Amber (4)
	4	4	4	4
	Red time (1)	red time (2) sec.	red time(3) sec	red time (4)
	131	166	156	156

From observation, the signal phasing was largely dependent on the treatment of left-turn movements because it was a 'Protected left turns', where the left turn has exclusive right-of-way (no conflict with through movement). The phasing control for the four-leg intersection was a four-phase operation in which two phases allocate right-of-way to north-south through traffic and left turn traffic and the other phases allocates right of way to east-west traffic through traffic and left turn traffic. A separate phases to protect left-turn movements was found to be a contributing factor to increased stop and delay to the approaches because of the longer cycle lengths. Nevertheless, other considerations, such as the intersection geometry and high volume of east-west traffic might also be contributing factors.

4.4 Traffic demand data

The collected data was processed to a form that the simulation software (VISSIM) can accept.

The vehicles in medium and truck categories were converted to passenger car units by adopting the passenger car equivalent (PCE) values as studied and established by Adams and Obiri-Yeboah (2008). The PCE values are: 1 for Cars, 1.7 for Medium and 2.5 for large vehicles.

The table 4.4 below shows traffic demand data for the video recording.

Table 4.4 Turning movement for KNUST (WEST BOUND) Approach.

Direction	PCU (WEST BOUND) KNUST APPROACH			
Time	WBT	WBR	WBTL	TOTAL
7:00-7:15	408	39	40	486
7:15-7:30	275	43	22	340
7:30-7:45	354	45	31	429
7:45-8:00	253	56	34	343
Entry Flow	**1631**	**155**	**160**	**1946**
% Flow	0.84	0.08	0.08	1.00

Table 4.5 Approach Demand (Pcu/H) For KNUST (West Bound) Approach.

Direction	Westbound			Eastbound	Northbound	Southbound
Time	**Entering (turning volume)**	**Queue**	**WB total**	**Entering (turning volume)**	**Entering (turning volume)**	**Entering (turning volume)**
7:00-7:15	486	231	717	344	209	181
7:15-7:30	340	212	553	278	253	239
7:30-7:45	429	255	684	376	228	202
7:45-8:00	343	195	538	296	271	216
DEMAND(PCU)		**2870**		**1505**	**1083**	**957**
PHV	**2492**			**1295**	**960**	**838**
PHF	**0.87**			**0.86**	**0.89**	**0.88**

4.5 Calibration

The existing intersection was calibrated using the throughput, queue length, delay and average speed measurements taken at the site. The Westbound approach (i.e. from KNUST) was chosen as the subject approach. This approach experience long queue and delay during both morning Am peak and evening Pm peak. In part of this chapter, the results of calibration and scenario analysis and results from the sensitivity are presented to determine the effect of left-turning traffic on the delays and saturations flow of the signalized intersection approaching through traffic.

4.6 Scenario One: Existing Condition

The simulation exercise was carried out with data that from the existing site conditions. Measured data such as the existing geometry, throughput, turning volumes and classification, delay, and queue length and signal phase sequence were used. Altogether ten simulation runs were conducted for the calibration. BothGeoffrey E. Havers (GEH) and Analysis of Varian (ANOVA) statistics tests were conducted using average delay and average saturation flow from the field and simulation results to determine the significance of the calibration.

4.6.1 Delay

The ten simulation runs resulted in average delay of 152 seconds while the measurement at the site yielded 161 seconds. In order to verify the accuracy of the model in predicting performance parameters after the calibration, a single factor ANOVA Tables was as presented in table 4.2 performed at 95% confidence level for the delay results. The results shows that $F_{test}(0.19) < F_{crit}(4.5)$. It can therefore be conclude that the observed values are close to the simulated values to an acceptable level and thus the delay is considered satisfactorily calibrated. Table 4.7 gives summary of the ANOVA test results.

Table 4.6 field and simulation delays data.

	Delay (s/veh)	
	Delay segment	
	Simulated	**Field**
Run 1	158	131
Run 2	134	131
Run 3	146	131
Run 4	152	191
Run 5	162	131
Run 6	154	131
Run 7	143	311
Run 8	167	131
Run 9	149	131
Run 10	155	131
Average	**152**	**161**

Table 4.7 ANOVA test results

Anova: Single Factor

SUMMARY

Groups	*Count*	*Sum*	*Average*	*Variance*
Column 1	10	1520.315	152.0315	90.53861
Column 2	8	1288	161	4114.286

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	357.4863	1	357.4863	0.193139	0.666195	4.493998
Within Groups	29614.85	16	1850.928			
Total	29972.33	17				

4.6.2 Saturation Flow

According to Traffic Modelling Guidelines (2014), 'goodness of fit' between field and modelled flows is best determined by the Geoffrey E. Havers (GEH) statistic which unlike comparing flows using simple percentage difference, places more emphasis on larger flows than on smaller flows**.** A GEH value of 0.3 was recorded for the westbound approach. All other approaches except Adum (Eastbound) also obtained GEH values less than 5.When all flows were analysed, the overall approaches recorded 1.3 which is an indication of a good match of the field and the simulated saturation flow data, hence, the model may be accepted as calibrated. Table 4.8 presents a summary of the GEH test results the various approaches.

Table 4.8 GEH test result of the approaches.

	Simulation	**Field**	**GEH**
Approach	**Sat. flow/(pcu/h)**	**Sat. flow/(pcu/h)**	
Knust	1582	1593	0.3
Asokwa	1793	1733	1.4
Adum	1323	1514	5.1
Airport	1406	1369	1.0
All combined	6105	6209	1.3

4.6.3 Queue Length

The Westbound approach (i.e. from KNUST to Anloga intersection) which experiences very long queue had maximum average queue length of 600m. In the simulation however a maximum queue length 0f 609m was obtained for the westbound target approach.

Using proportion, we can conclude that the model may be accepted as calibrated.

Using the calibrated model, the performance of the intersection approach was estimated as shown in Table 4.9

Table 4.9 Scenario 1, Summary of Simulation Result Using Existing Left-Turning Traffic Flow

Storage lane length (m)	saturation flow (through) pcu/h	Delay (sec/veh)	Maximum queue length (m)	HCM level of service
75	3192	152	609	F

The existing condition at the site is experiencing Level of Service F, vehicles experience excessive delay in excess of 152 sec/veh and maximum queue of 600m. The existing storage for exclusive left turn is 75m long. This may not seem to be sufficient for the high left turning proportion usually experienced at some signalized intersections.

To explore how increasing left turning vehicles proportion may affect the saturation flows and delays to through traffic, a sensitivity analysis was performed.

4.7 Evaluation of delay and Saturation flow

4.7.1.1 Evaluation of delay at 10% left turn traffic volume.

For approach demand from 1000pcu/h to 3000pcu/h, the delay was investigated for 10% left turn and a varying storage length. The results are presented in figure 4.4.the figure shows very little difference in average delay when the storage length was changed from100m to 150m except for 75m. Generally as the approach volume increase, the delay rises. In the case of 75m storage, there is an initial steady rise in delay which gradually flattens beyond 2000pcu/h flows.

There is a point where the average delays for all storage length produces similar values of delay at 2500pcu/h. at approach volume of 1000pcu/h and 10% left turning, it seems the performance of 75m storage for left turn traffic is adequate. Beyond this value 100m length of storage for left turning traffic will be adequate as the difference in delay between 100m and 150m length of storage is very marginal (i.e. less than 4 sec/h)

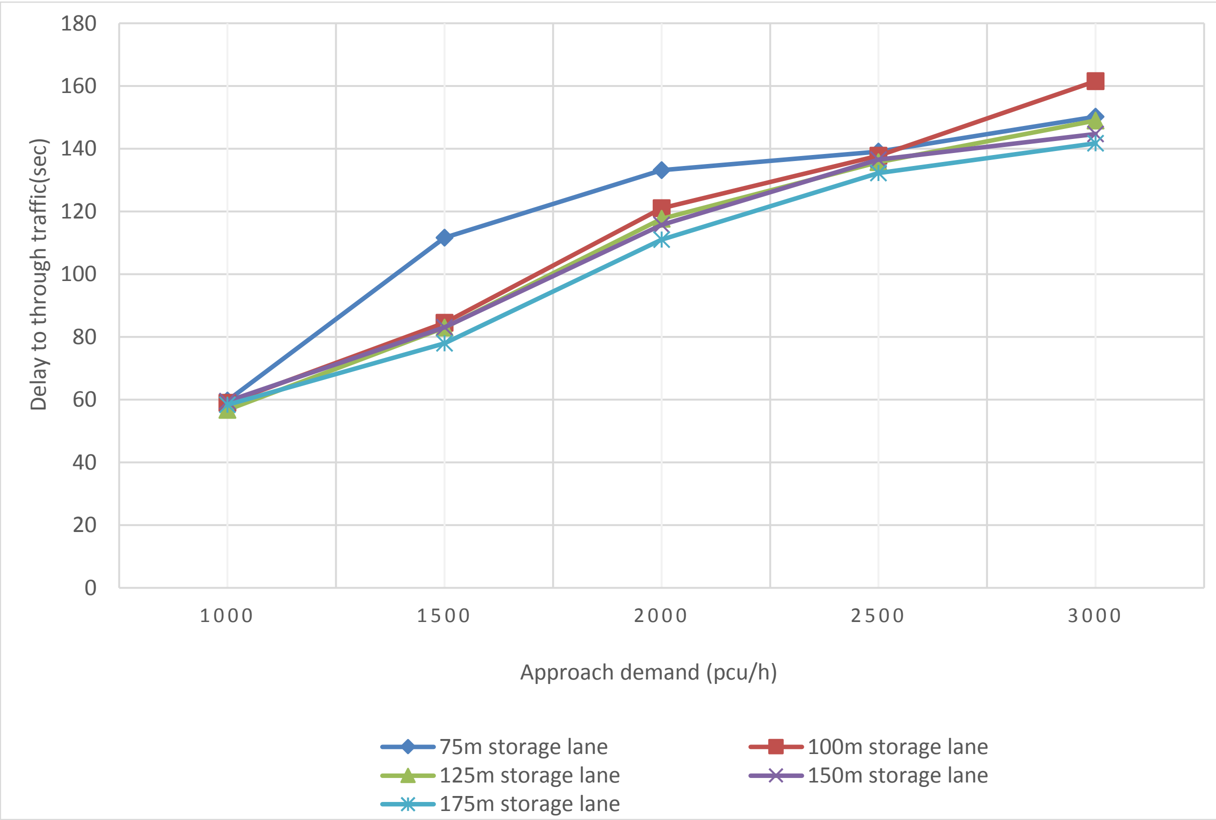

Figure 4.4 Delay to through traffic for approach demand of 1000pcu/h to 3000pcu/h with 10% left turn.

Based on this discussion, we can conclude that for approaches which have approach flows (1000-2000) and low percentage of left turn (i.e. up to 10%), provision of left turn storage length of 125m may not reduce delay to through traffic.

Also for flows lower than 1000pcu/h, on 75m storage length will be adequate for as low as the proportion of left turners is less than 10%.

4.7.1.2 Evaluation of Saturation flow at 10% left turn traffic volume.

Based on the scenario discussed above, the effect of 10% left turning traffic was investigate this could block the lanes and prevent through traffic from adequately utilizing the green phase of the signal. The saturation flow for various length of left turn accommodation is presented in figure 4.5. As approach flows increase from 1000pcu/h to 1500pcu/h, there is a rise in saturation flow and then very gradual increase from 1500 to 2000pcu/h. Beyond 2000pcu/h, saturation flows remained relatively constant. Generally, all the storage lane lengths have marginal difference between them. However, 150m storage lane seems to be the optimal storage lane length.

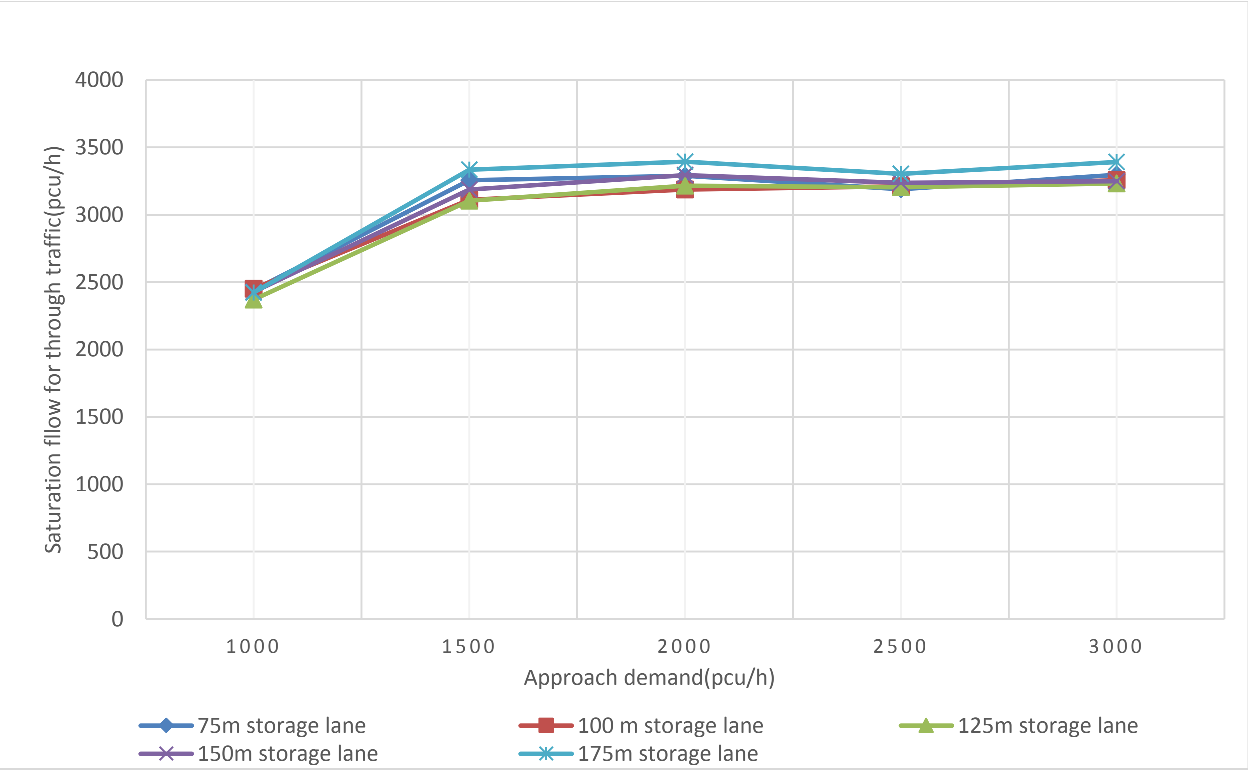

Figure 4.5 Saturation flow for through traffic for 10% left turn traffic proportion of 1000 to 3000 pcu/h approach demand.

4.7.1.3 Evaluation of delay at 20% left turn traffic volume.

The 75m storage lane recorded the highest delays of 62, 87, 131, 150, and 155 seconds for 1000pcu/h, 1500pcu/h, 2000pcu/h, 2500pcu/h and 3000pcu/h approach volumes respectively. Generally, as approach flows increase, delays increase but not as rapidly as the case when the left turn proportion was 10%. This is evident in the difference in values of delays recorded for both 10% and 20% left turn proportion. Apart from the 75m storage option, all other storage lengths recorded a reduction in delay relative to 75m storage according to figure 4.6. The longer the storage, the lower the delay, though the differences were very marginal.This means that the 75m storage is not perfoming better as compared to the others. Due to the marginal difference between the 100m, 125m, 150m and 175m storage lanes, it implies however that provision of a 100m storage lane can perform optimally better in reducing delays to the through traffic since there is no significant gain in reduction in delay beyond 100m storage lane.

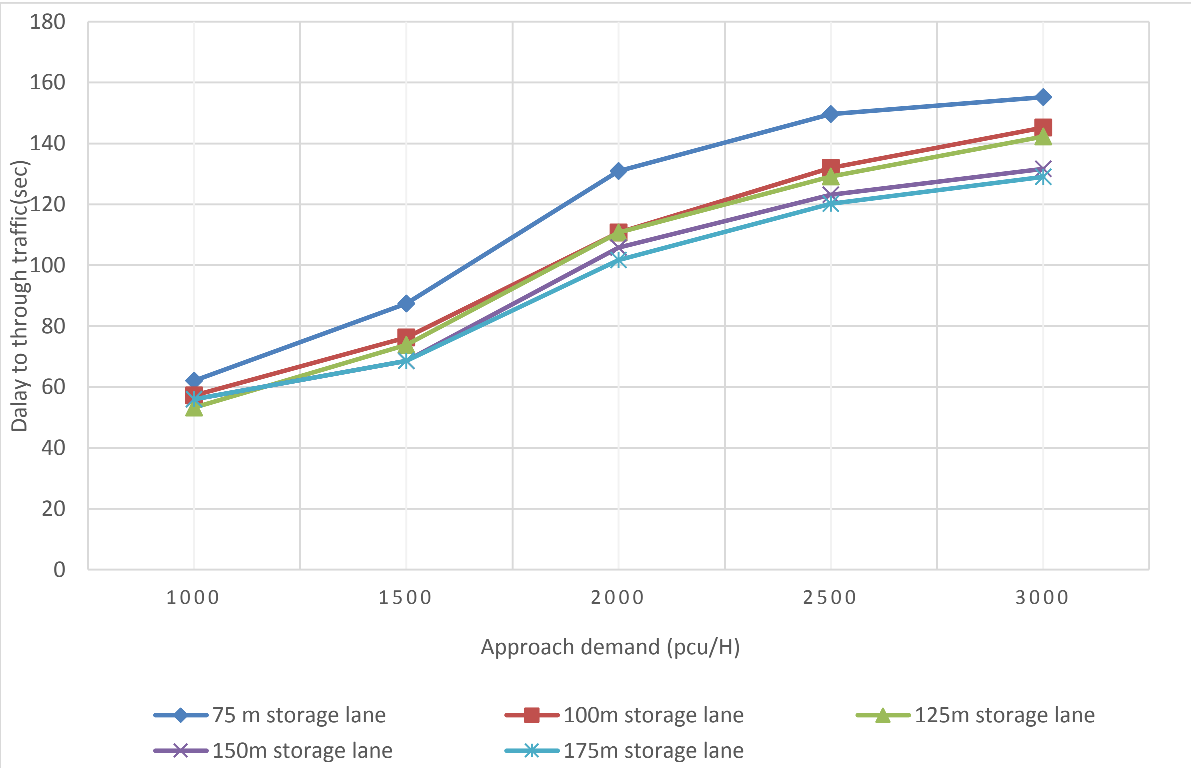

Figure 4.6 Delay to through traffic for left turn traffic volume of 20%

It may be observed from the graph that a 100m left turn storage has the same delay profile as a 125m length. A similar observation may be made for 150m 175m storage lengths. This suggest that it may be worthwhile to increase from 75m to 100m and 150m if the objective is to reduce delay.

4.7.1.4 Evaluation of Saturation flow at 20% left turn traffic volume.

The scenario discused above has revealed that there is an effect of 20% left turning traffic on delay increase as the approach volume increase. As shown in figure 4.7, saturation flow at 1000pcu/h approach demand for all the storage lanes were almost at pal, but for 1500pcu/h approach volume marginal difference occurred. figure 4.7 shows that 175m storage lane began recording a higher saturation flow, followed by 150m storage lane with a little margin with others. Based on the trend,it can be concluded that above 1500pcu/h approach volume, more than one lane may be required to optimize the utilization of the green time and thereby increase saturation flow since the increase in storage lane lengths is not seing a significant increase in saturation flows.

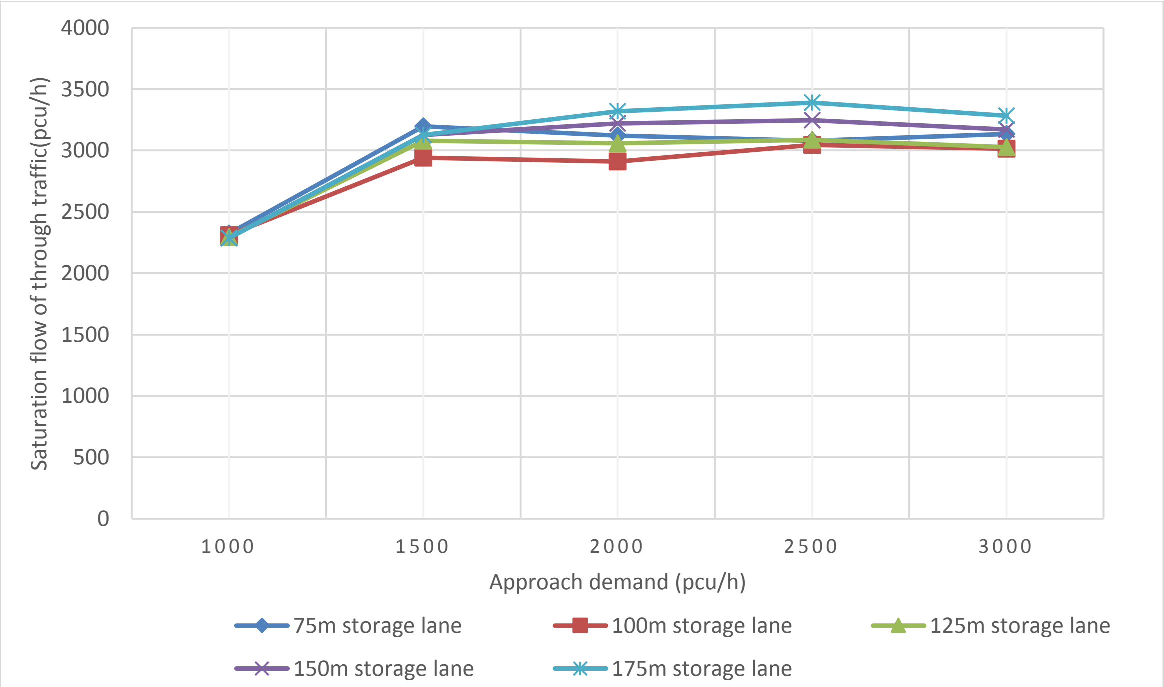

Figure 4.7 Saturation flow for through traffic at a left turn traffic of 20%.

Beyond 1500pcu/h, there is generally no increase in saturation flow to through traffic with increasing in the length of left turn storage except for 175m length which resulted in marginal rise in saturation flows up to 2500pcu/h.

4.7.1.5 Evaluation of delay at 30% left turn traffic volume.

After investigating the delays for 30% left turn traffic for approach demand from 1000pcu/h to 3000pcu/h, it was revealed that as the approach volume increase with increase in left turn traffic proportion, there is more increase in delays especially with shorter storage lane. This might be because of the increasing number of vehicles turning left, as it has the propensity of spilling back into the lanes of the through traffic, thereby denying the through traffic of full utilization of the green time. As shown in figure 4.8, there is a consistent trend in increase delays as the volume of the approach increases with a smaller storage lane length. At approach volume of 1000pcu/h, all but 75m storage lanes had almost the same value for delays, making100m storage the optimal lane required for this approach volume. Meanwhile for 1500pcu/h approach volume, 175m storage lane perform better though the difference with the others apart from 75m was marginal. Generally, 175m storage lane perform better right from 1500pcu/h to 3000pcu/h. in fact, the difference in reduction in delays between the 75m and the 175m is so significant that as 75m storage lane further inclined in delays, there was a sign of declination in delays after 2500pcu/h approach volume according to figure 4.8. this phenomenon is due to the fact that as the storage lane increases, more of the left turning vehicles are taking off the road space of the though traffic.

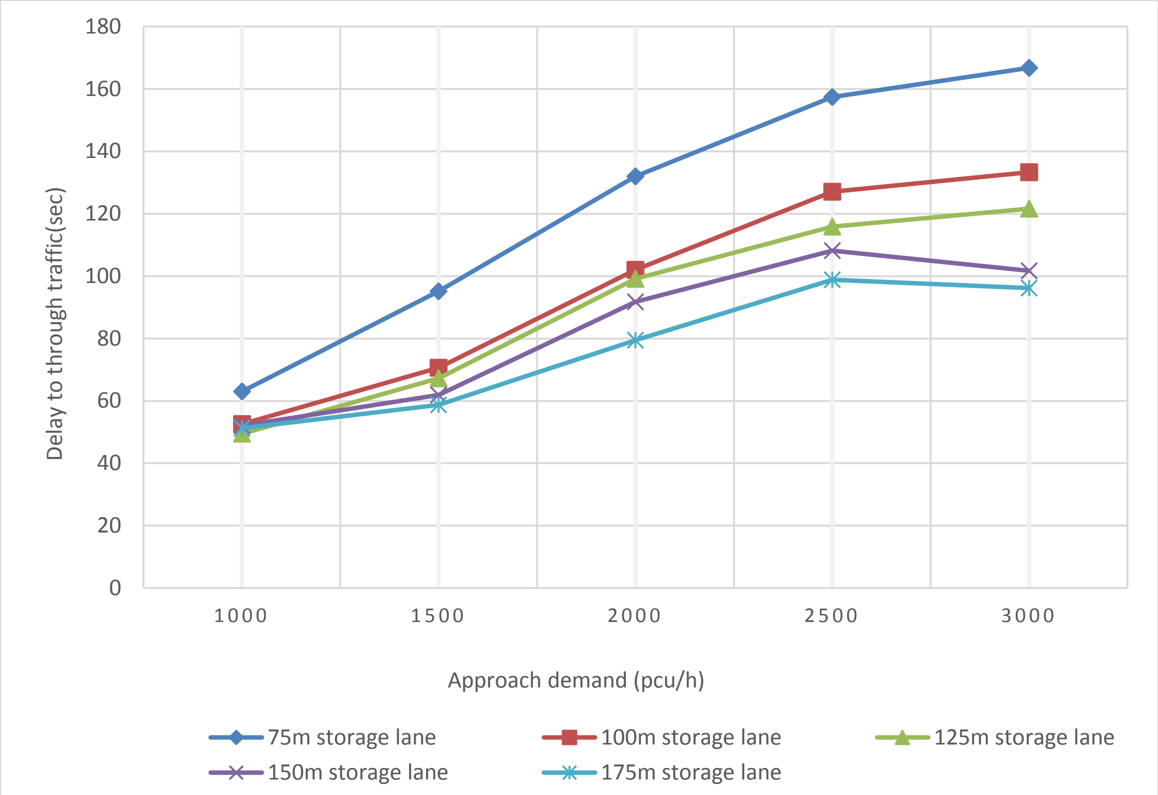

Figure 4.8 Delay to through traffic for left turn traffic of 30%

4.7.1.6 Evaluation of Saturation flow at 30% left turn traffic volume.

As demonstrated in the discussion above on 30% left turn traffic volume, there was almost equal performance in terms of saturation flow also for approach volume of 1000pcu/h. 150m and 175m storage lanes coincided from 1000pcu/h to 2000pcu/h approach volumes. However, beyond 2000pcu/h approach volume, 150m storage declined while 175m inclined slowly.

It is quite obvious from the discussion above, confirmed by figure 4.9 that 175m storage lane is the optimal required to improve both on saturation flow and delays for an approach volume of 1500pcu/h and above.

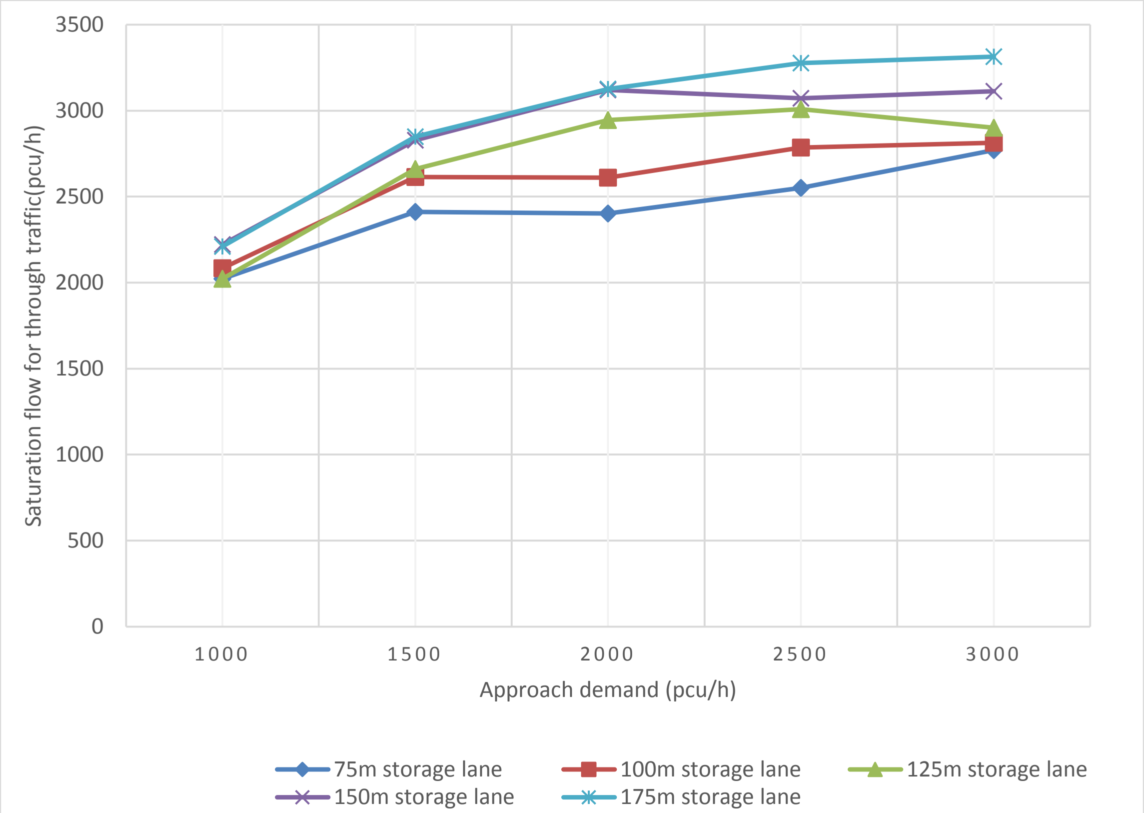

Figure 4.9 Saturation flow for through traffic for left turn traffic of 30%

4.7.1.7 Evaluation of delay for 40% left turn traffic volume

The delay was investigated for 40% left turn and a varying storage length for approach demand of 1000pcu/h to 3000pcu/h. at 1000pcu/h, 75m storage obtained the highest delay, however, the remaing storage lanes (i.e. 100m, 125m 150m and 175m) recorded almost the same values of reduced delay as compared to 75m. the reason might be because, with a lower approach volume, left turn traffic volume is taking out of the roadway of the through traffic by just a small increase in the left turn storage lane. The implication is that, at this approach volume (i.e. 1000pcu/h), we only reqiure a maximum storage lane of 100m to cause a reduction in delay. In that case, any storage lane above 100m would not make any significant difference. For an approach volume of 1500pcu/h however, 75m storage lane coincided with a 100m storage lane and the trend continued to an approach volume of 3000pcu/h as the highest delay. Figure 4.10 shows that 150m and 175m storage obtained the lowest delays with maginal diffence inbetween them at 1500pcu/h and 2500pcu/h approach volums, nevertheless, because there is significant difference in delay at approach volume of 3000pcu/h with a seemingly declined trend, we can conclude that 175m storage lane is required to reduce the delays for these approach volumes with this left turn percentage.

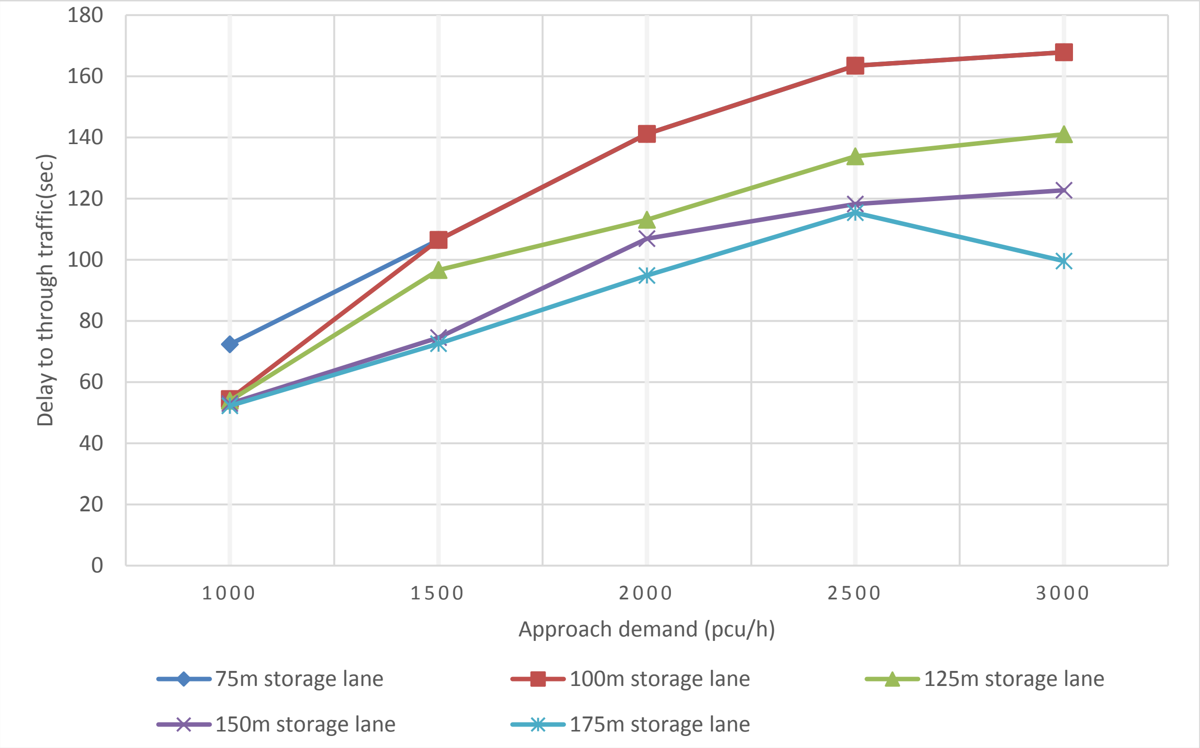

Figure 4.10 Delay to through traffic for left turn traffic of 40%

4.7.1.8 Evaluation of Saturation flow at 40% left turn traffic volume.

As the scenario discussed above, 175m storage lane obtained the highest saturation from 1000pcu/h to 3000pcu/h approach volume. However, there is a decline at 2500pcu/h and a rise afterwards. This fluctuation could be attributed to lane indiscipline by vehicle in the stream.

The worse performing lane according to figure 4.11 is 75m and 100m storage lane. This might be because of their inadequacy in length that causes a spill back of the left turners into the through traffic lanes. Based on this we can conclude that for best performance in terms of reduction in delay and improvement in saturation flows for 40% left turn traffic proportion of 1000pcu/h to 3000pcu/h approach volumes, 175m storage lane is required.

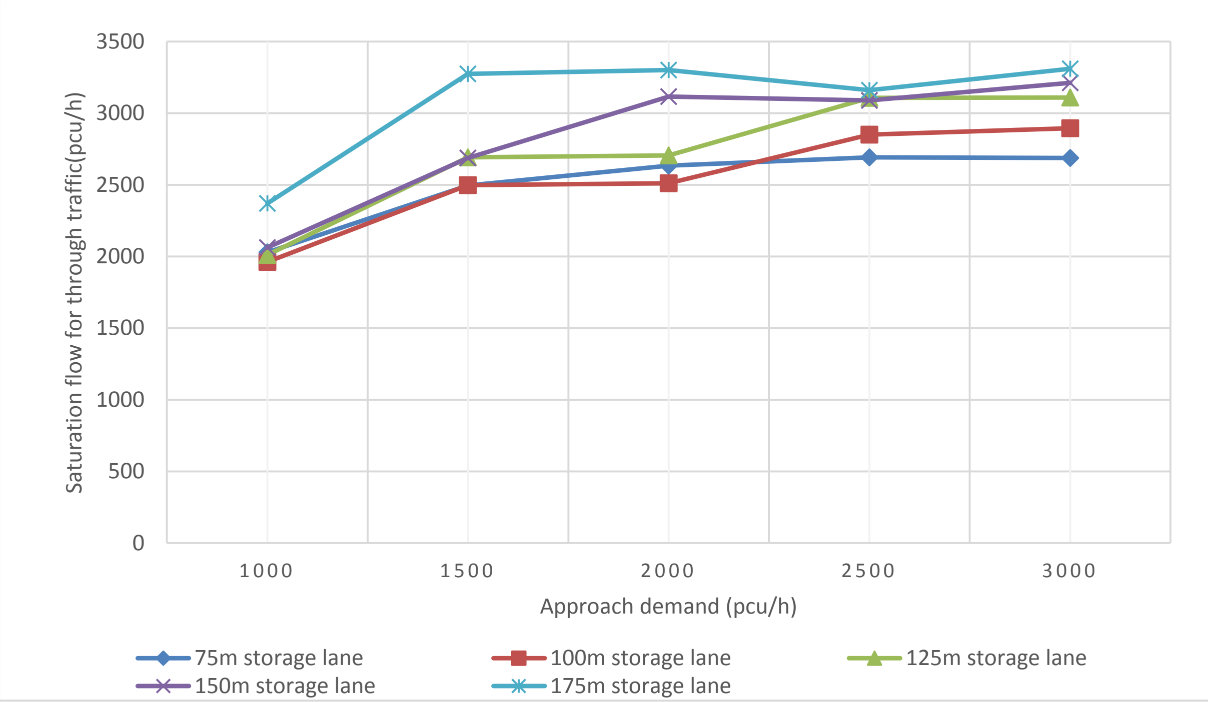

Figure 4.11 Saturation flow for through traffic for left turn traffic of 40%

4.7.1.7 Evaluation of delay for 50% left turn traffic volume

Much of the delays occurred with the 75m storage lane for 1000pcu/h to 3000pcu/h approach volume. At lower volume (i.e. 1000pcu/h) with 50% left turn as shown in fig. 4.12, delay to through traffic for 75m storage is 82 seconds and 189 seconds at a higher volume of 2500pcu/h. this is an indication of a total breakdown of the flow system characterized by queue of infinity length. Driver frustration at this point is unimaginable and unacceptable. However, for the same 1000pcu/h to 3000pcu/h approach volumes, 175m storage lane recorded delays of as low as 50 seconds at 1000pcu/h approach volume, bringing the level of service from F to D and 107 seconds at 2500pcu/h. this represents a whopping 39% and 44% reduction in delays respectively. This reduction is as a result of the long storage lane taking out most of the left turn traffic from the road space of the through traffic and thereby maximizing the utilization of the green time of the signal by the through traffic. Based on this, we can therefore conclude that 175m storage lane is required for the reduction in delays to through traffic.

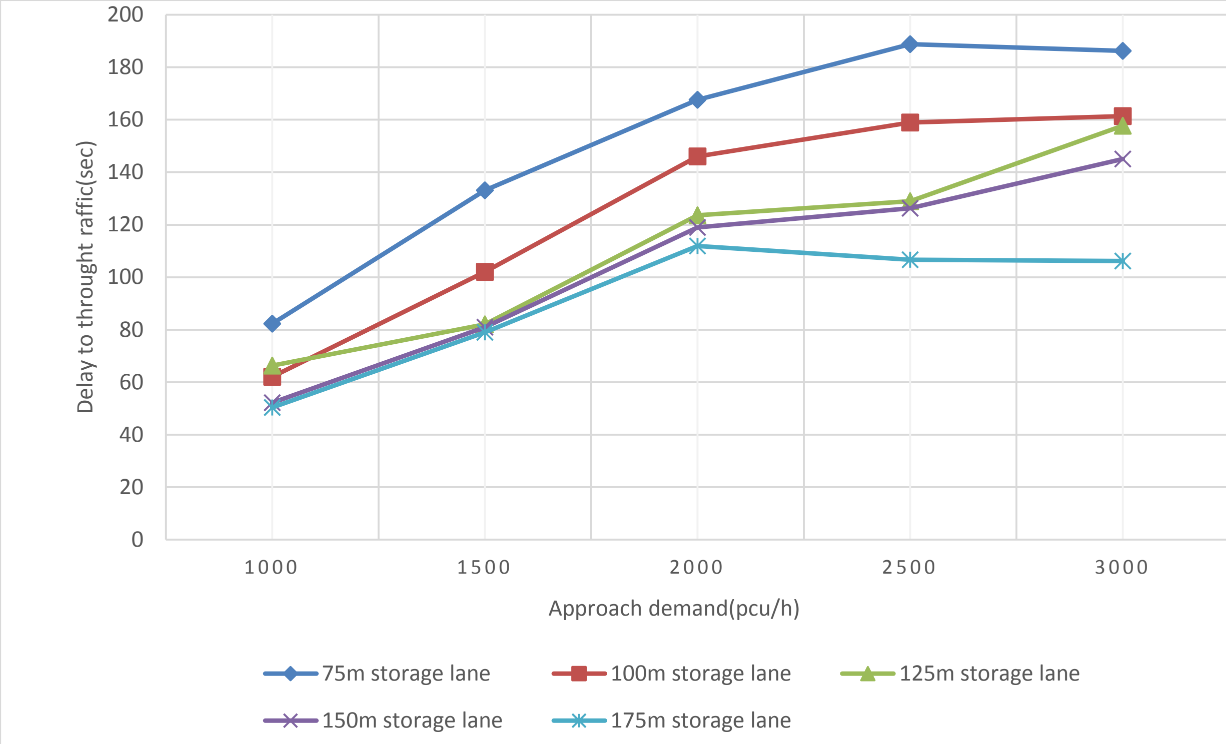

Figure 4.12 Delay to through traffic for left turn traffic of 50%

4.7.1.9 Evaluation of Saturation flow at 50% left turn traffic volume

The performance of the 175m storage lane in terms of saturation flow is in consonant with its performance in terms of delay. 175m storage lane obtained the highest saturation flow at all points as shown in fig.4.13. At 1500pcu/h, 175m and 150m storage lanes coincided, indicating that either 150m or 175m storage could be required to improve saturation flow. Beyond 1000pcu/h, 75m and 100m storage lanes obtained the lowest saturation flow as compared to 175m 150m and 125m storage lane. This confirms the indication of a total breakdown of the flow system characterized by queue of infinity length, unimaginable and unacceptable driver frustration. On this bases 175m storage lane is required.

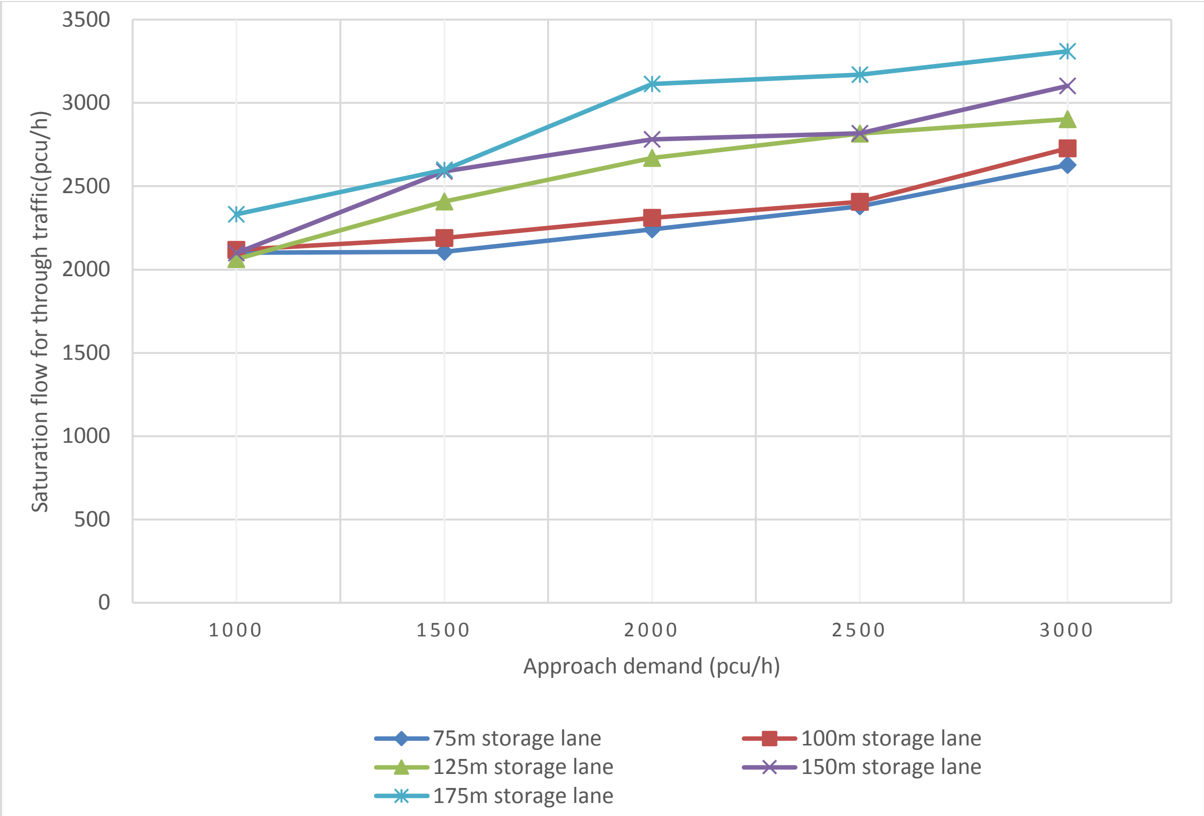

Figure 4.13 Saturation flow for through traffic for left turn traffic of 50%

CHAPTER 5: CONCLUSION AND RECOMMENDATION

5.1 Conclusions

The following conclusions have been drawn based on the results of the study conducted:

- The delays and saturation flow model have been calibrated successfully for Anlogah signalized intersections with good predictions for similar sites with similar characteristics.
- As the approach volume increased from 1000pcu/h to 3000pcu/h in increments of 500pcu/h and the left-turning traffic volume proportion also increased from 10% to 50%, delays to through traffic also increased whilst saturation flow of through traffic also deteriorated because of the spillback of left turn traffic into the lanes of the through traffic, thereby blocking the through traffic from utilizing the green time.
- For lower approach volume up to1000pcu/h and left turn traffic proportion of 10% to 20%, 75m left turn storage lane is adequate to reduce delays to through traffic.. But beyond 1000pcu/h up to 2500pcu/h approach volume and 10% left turn proportion, 100m storage lane is adequate. For saturation flows on the other hand, 75m storage lane is adequate for approach volume of 1000pcu/h with 10% to 20% left turn proportion, whilst 100m storage is adequate for the remaining approach volumes with 10% to 20% left turn proportion, though there was a declined generally in saturation flows, this is attributed to the fact that increase in length of storage lane has very little effect on the saturation flows when the percentage of left turning traffic is 20% and below. However, 150 m storage seems to be the optimum storage lane required to reduce delay at approach volume 1000pcu/h to 3000pcu/h with 10% to 20% left turn proportion
- As the left turn traffic percentage increases from 30% and beyond, storage lane of 175m may be required for the reduction in delays and improvement in saturation

flows. Generally however, under such circumstances a double left turn storage lane will be useful in reducing the delay and queues to acceptable levels.

5.2 Recommendations

Based on the discussion and conclusions above, we recommend the following:

- For an approach volume of up to1000pcu/h and 10% to 20% left turn traffic proportion, 75m left turn storage lane length may be required to reduce delay and improve saturation flow.
- For an approach volume of up to 1000pcu/h t0 2500pcu/h with 10% to 20% left turn traffic proportion, 100m storage lane is required to reduce delay and improve saturation flow.
- The use of left turning storage for high left turn volumes is a very effective way of reducing delay, queueing and improving saturation flow of through traffic.
- Further study could look at the impact of right-turning traffic flow and adjustment in the signal configuration also.
- Further studies could be conducted on several signalized intersections in Ghana and standards developed for the extensions of storage lanes as a means to reduce delays and improve saturation flow and capacity.
- Traffic engineers in Ghana can explore the method used in the calibration and simulation to find inexpensive solutions such as storage lane extension to signalized intersection congestion problems in our urban centers while waiting for funds for other interventions like grade separations (interchange) and addition of lanes.

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APPENDICES

APPENDIX A: DEMAND DATA FOR THE APPROACHES

A1. Approach pcu

	KNUST	ASKWA	ADUM	AIRPT
RUN 1	1738	1532	1869	1245
RUN 2	1669	1389	1737	1622
RUN 3	1686	1413	1472	1578
RUN 4	1747	1452	1544	1360
RUN 5	1479	1487	1906	1393
RUN 6	1651	1567	1705	1531
RUN 7	1677	1394	1631	1538
RUN 8	1737	1422	1734	1322
RUN 9	1687	1388	1676	1203
RUN 10	1723	1568	1762	1336
AVG. SAT. FLOW pcu/h/ln	1679	1461	1704	1413

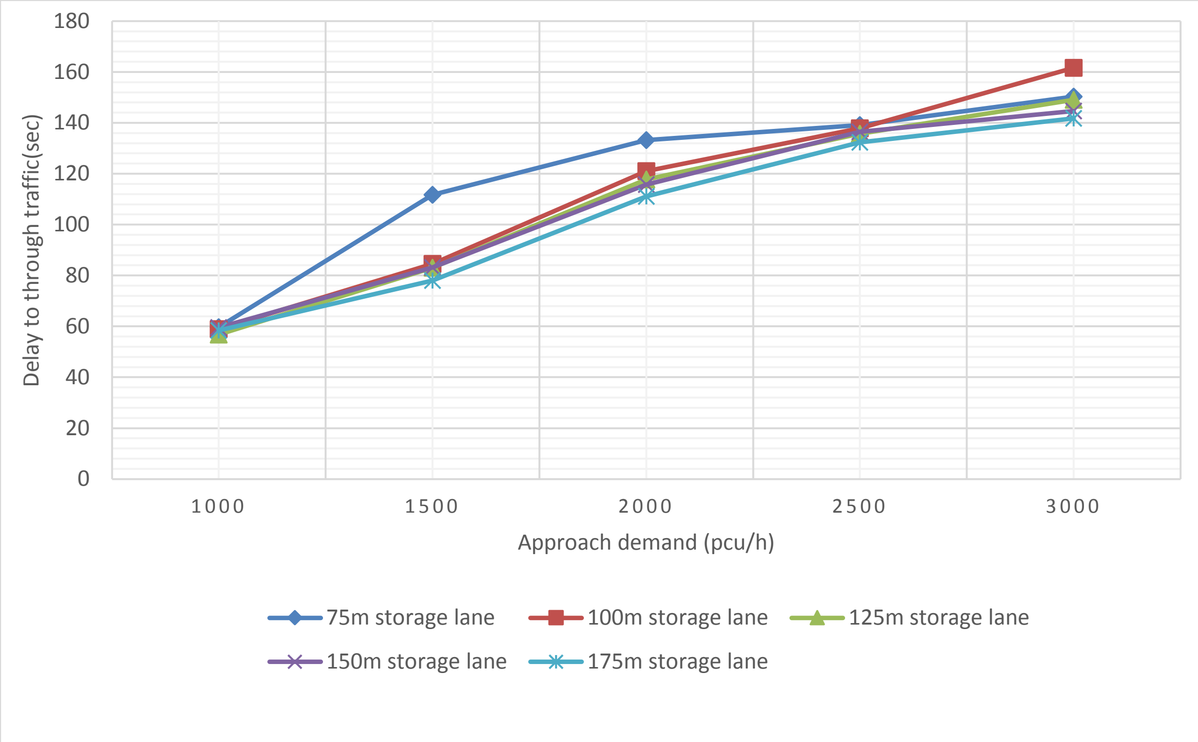
APPENDIX B: SIMULATED RESULTS

B1. Field and simulated delays for existing condition.

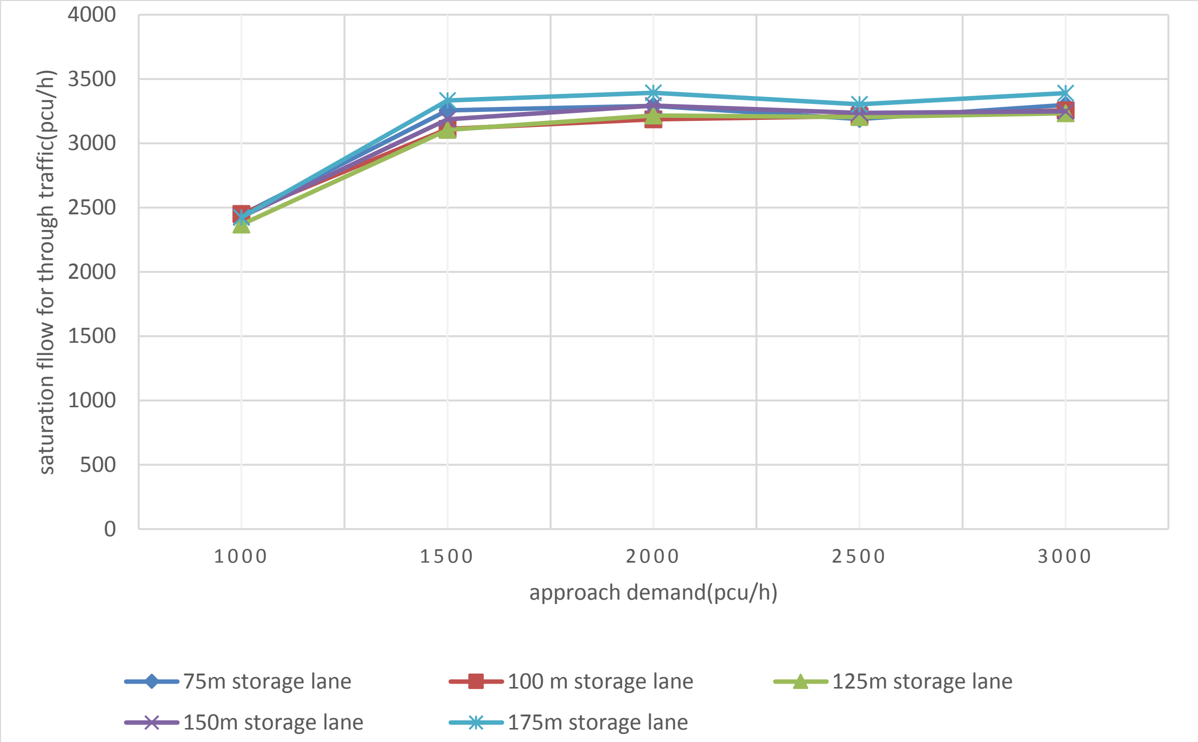
	Delay (s/veh)	
	Delay segment	
	Simulated	**Field**
Run 1	158	131
Run 2	134	131
Run 3	146	131
Run 4	152	191
Run 5	162	131
Run 6	154	131
Run 7	143	311
Run 8	167	131
Run 9	149	131
Run 10	155	131
Average	**152**	**161**

B2delay and saturation flow for through traffic for 10% left turn traffic proportion

approach demand (pcu/h)	storage lane (m)	delay to through trff. (s)
1000	75	60
1500	75	112
2000	75	133
2500	75	139
3000	75	150
1000	100	59
1500	100	85
2000	100	121
2500	100	138
3000	100	162
1000	125	57
1500	125	83
2000	125	118
2500	125	136
3000	125	149
1000	150	60
1500	150	83
2000	150	116
2500	150	137
3000	150	145
1000	175	58
1500	175	78
2000	175	111
2500	175	132
3000	175	142

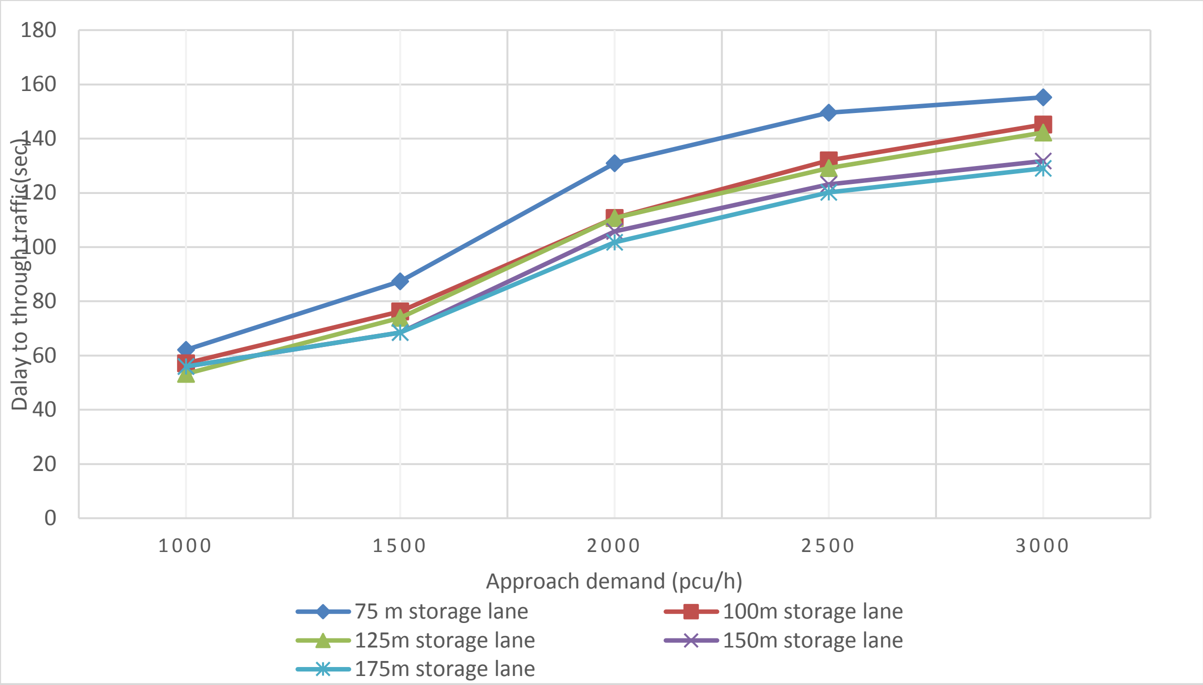


approach demand (pcu/h)	storage lane (m)	daturation flow for through trff. (pcu/h)
1000	75	2442
1500	75	3255
2000	75	3290
2500	75	3188
3000	75	3297
1000	100	2450
1500	100	3111
2000	100	3187
2500	100	3210
3000	100	3256
1000	125	2369
1500	125	3104
2000	125	3215
2500	125	3203
3000	125	3233
1000	150	2428
1500	150	3186
2000	150	3294
2500	150	3236
3000	150	3248
1000	175	2422
1500	175	3334
2000	175	3393
2500	175	3303
3000	175	3391

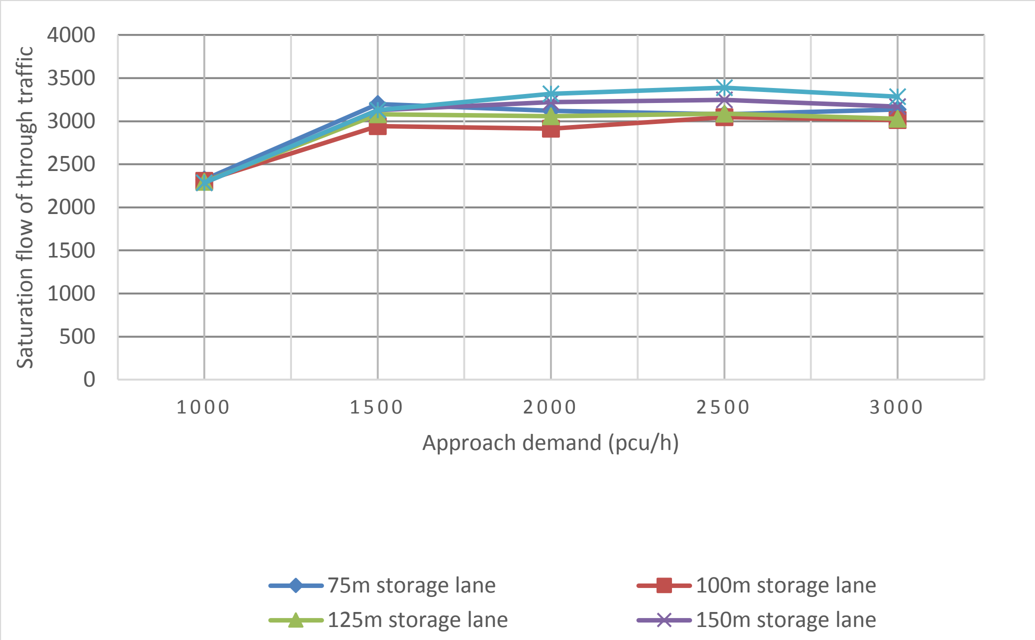


B3.delay and saturation flow for through traffic for 20% left turn traffic proportion

approach demand (pcu/h)	storage lane (m)	delay to through trff. (s)
1000	75	62
1500	75	87
2000	75	131
2500	75	150
3000	75	155
1000	100	57
1500	100	76
2000	100	111
2500	100	132
3000	100	145
1000	125	53
1500	125	74
2000	125	111
2500	125	129
3000	125	142
1000	150	56
1500	150	69
2000	150	106
2500	150	123
3000	150	132
1000	175	56
1500	175	69
2000	175	102
2500	175	120
3000	175	129

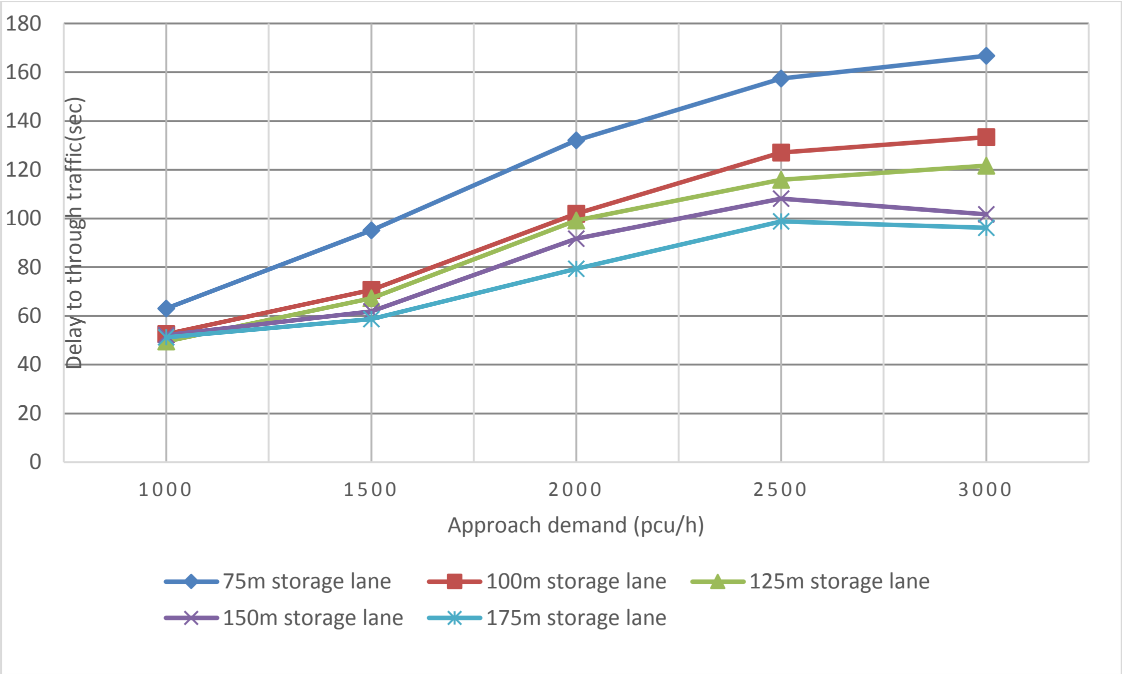


approach demand (pcu/h)	storage lane (m)	saturation flow for through trff. (pcu/h)
1000	75	2323
1500	75	3198
2000	75	3121
2500	75	3080
3000	75	3135
1000	100	2309
1500	100	2942
2000	100	2911
2500	100	3045
3000	100	3014
1000	125	2295
1500	125	3080
2000	125	3059
2500	125	3088
3000	125	3026
1000	150	2287
1500	150	3126
2000	150	3220
2500	150	3246
3000	150	3170
1000	175	2287
1500	175	3126
2000	175	3319
2500	175	3390
3000	175	3283

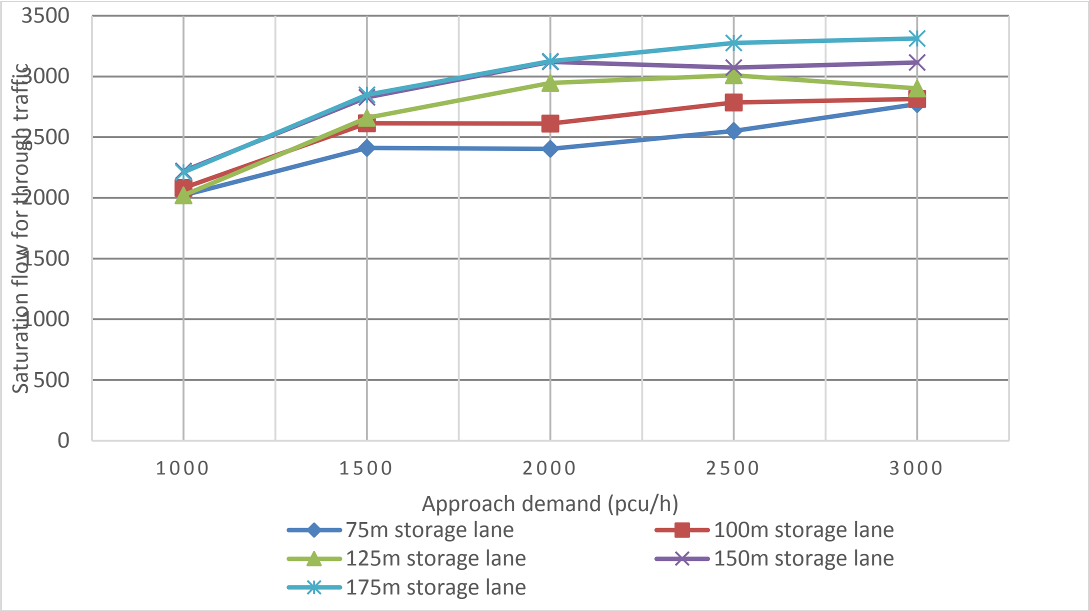


B4. Delay and saturation flow for through traffic for 30% left turn traffic proportion

approach demand (pcu/h)	storage lane (m)	delay to through trff. (s)
1000	75	63
1500	75	95
2000	75	132
2500	75	157
3000	75	167
1000	100	53
1500	100	71
2000	100	102
2500	100	127
3000	100	133
1000	125	50
1500	125	67
2000	125	99
2500	125	116
3000	125	122
1000	150	52
1500	150	62
2000	150	92
2500	150	108
3000	150	102
1000	175	51
1500	175	59
2000	175	79
2500	175	99
3000	175	96

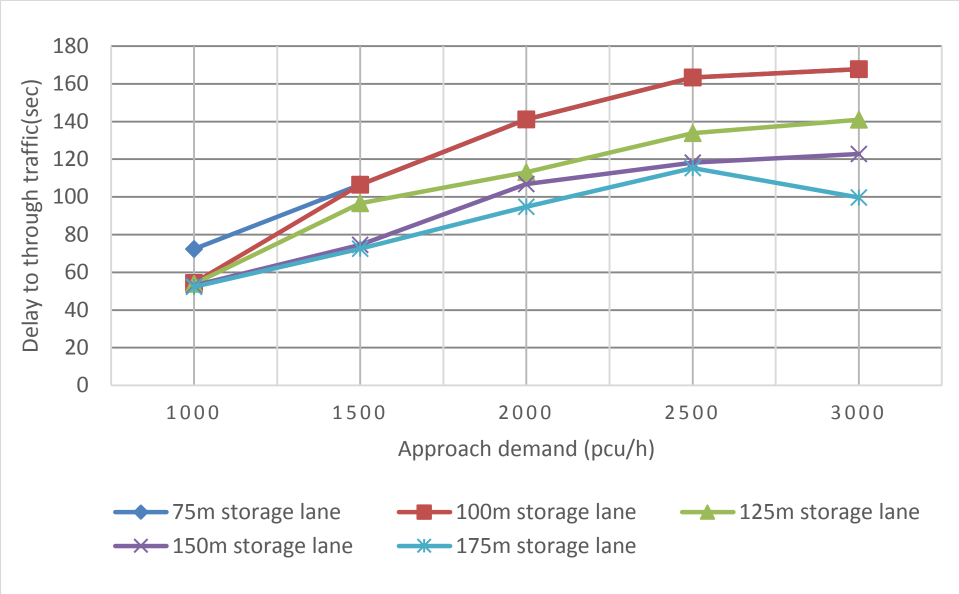


approach demand (pcu/h)	storage lane (m)	daturation flow for through trff. (pcu/h)
1000	75	2022
1500	75	2411
2000	75	2402
2500	75	2551
3000	75	2771
1000	100	2083
1500	100	2613
2000	100	2611
2500	100	2786
3000	100	2813
1000	125	2022
1500	125	2660
2000	125	2946
2500	125	3009
3000	125	2901
1000	150	2221
1500	150	2828
2000	150	3121
2500	150	3071
3000	150	3113
1000	175	2211
1500	175	2848
2000	175	3125
2500	175	3276
3000	175	3313

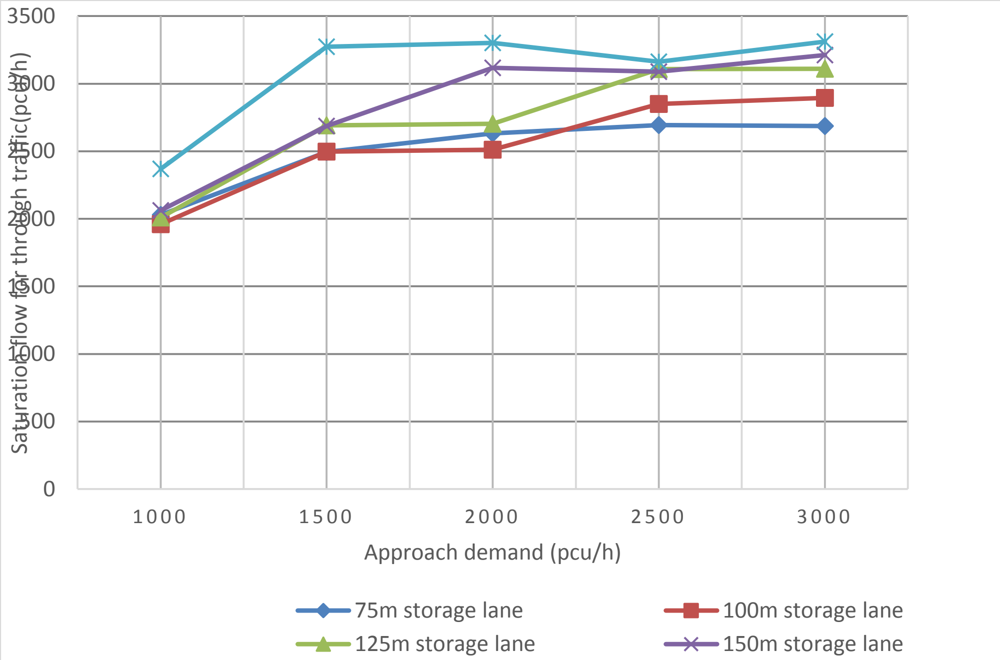


B5.delay and saturation flow for through traffic for 40% left turn traffic proportion

approach demand (pcu/h)	storage lane (m)	delay to through trff. (s)
1000	75	72
1500	75	106
2000	75	141
2500	75	163
3000	75	168
1000	100	54
1500	100	106
2000	100	141
2500	100	163
3000	100	168
1000	125	54
1500	125	97
2000	125	113
2500	125	134
3000	125	141
1000	150	53
1500	150	75
2000	150	107
2500	150	118
3000	150	123
1000	175	52
1500	175	73
2000	175	95
2500	175	115
3000	175	100

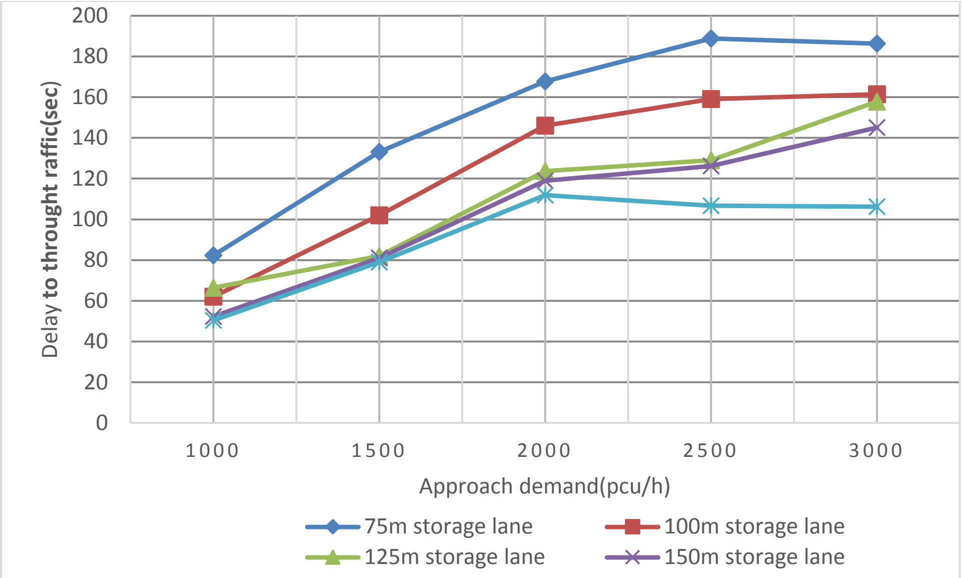


approach demand (pcu/h)	storage lane (m)	daturation flow for through trff. (pcu/h)
1000	75	2029
1500	75	2495
2000	75	2633
2500	75	2693
3000	75	2687
1000	100	1961
1500	100	2497
2000	100	2511
2500	100	2850
3000	100	2896
1000	125	2011
1500	125	2691
2000	125	2704
2500	125	3107
3000	125	3110
1000	150	2063
1500	150	2687
2000	150	3116
2500	150	3089
3000	150	3211
1000	175	2370
1500	175	3275
2000	175	3302
2500	175	3161
3000	175	3311

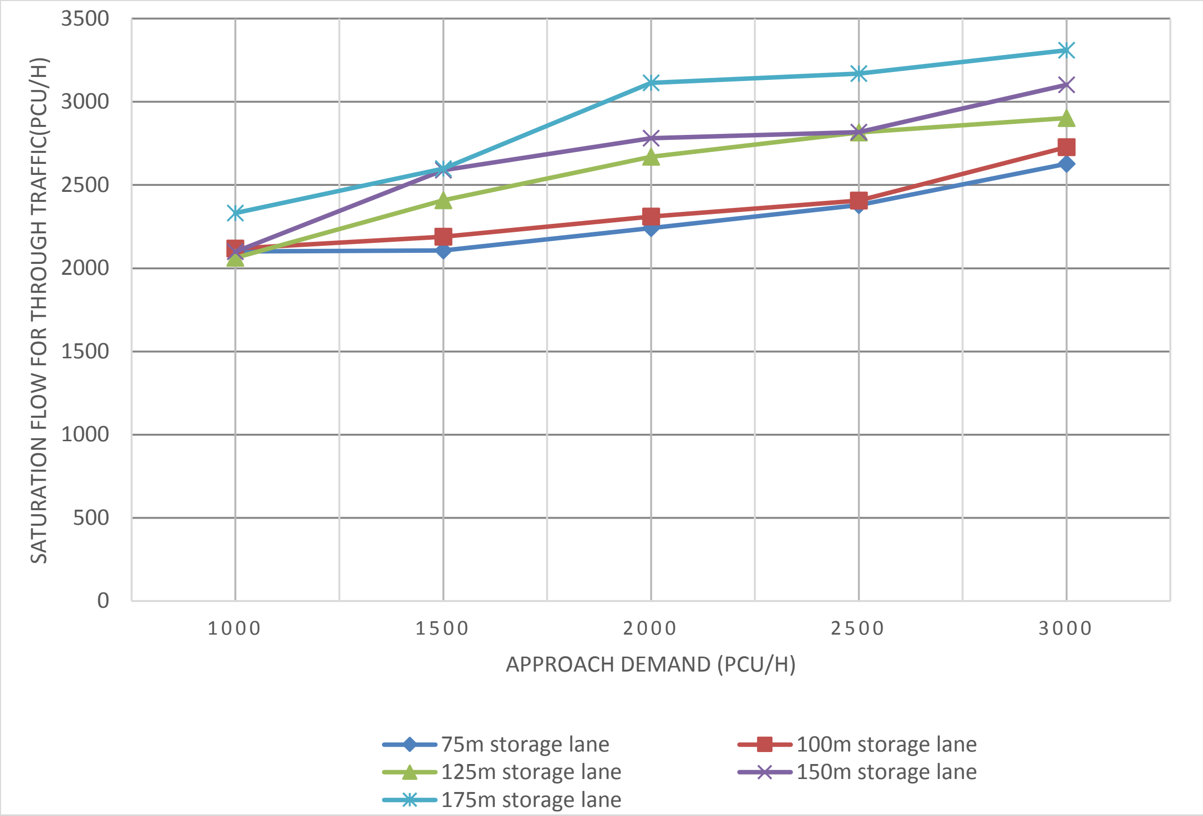


5. delay and saturation flow for through traffic for 40% left turn traffic proportion

approach demand (pcu/h)	storage lane (m)	delay to through trff. (s)
1000	75	82
1500	75	133
2000	75	168
2500	75	189
3000	75	186
1000	100	62
1500	100	102
2000	100	146
2500	100	159
3000	100	161
1000	125	66
1500	125	82
2000	125	124
2500	125	129
3000	125	158
1000	150	52
1500	150	81
2000	150	119
2500	150	126
3000	150	145
1000	175	50
1500	175	79
2000	175	112
2500	175	107
3000	175	106



approach demand (pcu/h)	storage lane (m)	saturation flow for through traffic. (pcu/h)
1000	75	2101
1500	75	2106
2000	75	2241
2500	75	2379
3000	75	2627
1000	100	2119
1500	100	2190
2000	100	2310
2500	100	2408
3000	100	2728
1000	125	2063
1500	125	2409
2000	125	2671
2500	125	2816
3000	125	2902
1000	150	2100
1500	150	2590
2000	150	2781
2500	150	2819
3000	150	3102
1000	175	2331
1500	175	2600
2000	175	3114
2500	175	3170
3000	175	3310


3500
3000
2500
2000
1500
1000
500
0
SATURATION FLOW FOR THROUGH TRAFFIC(PCU/H)
1000
1500
2000
2500
3000
APPROACH DEMAND (PCU/H)
75m storage lane
100m storage lane
125m storage lane
150m storage lane
175m storage lane