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Left Turn Accommodation at Unsignalized T-Intersections: A Case Study in Kumasi Using VISSIM Modelling

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

The most common type of intersection improvement considered in Ghana is signalization. However, other actions such as the installation of left-turn lanes (left-turn accommodation) have been proven to improve the efficiency and safety of an intersection without the need for signalization in the short term. Warrants which guide the installation of such lanes in Ghana are adopted from elsewhere and as a result do not reflect local traffic flow conditions. This study investigated the use of VISSIM micro simulation tool to establish volume warrants based on a delay threshold to guide the installation of left-turn lanes at unsignalized Tintersections. The VISSIM model was calibrated using traffic flow, delay, average and maximum queue length data obtained from a two-hour video recording of the case study intersection during the morning peak period. After calibration, several scenarios covering a wide range of operational conditions were simulated. Using Level of Service (LOS) C cut off point of 25 s/veh as the maximum acceptable delay to minor road left-turning traffic, an equation has been developed which predicts the threshold minor road left-turn volume above which a minor road left-turn lane may be considered and below which a minor road left-turn lane may not be necessary for a range of major road volumes. The critical delay to major road left-turning traffic was found to be 16 s/veh. Major road left-turn lane volume warrants were determined based on this threshold delay value. The approach used in this study can serve as a procedure guide that can be used by metropolitan and municipal road engineers to assess the need for left-turn lanes.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ANOVA	Analysis of Variance
B/C	Benefit-Cost
FHWA	Federal Highway Administration
GEH	Geoffrey Havers
GHA	Ghana Highway Authority
HCM	Highway Capacity Manual
LOS	Level of Service
LTGAP	Left Turn Guidelines Analysis Package
MOEs	Measures of Effectiveness
MUTCD	Manual on Uniform Traffic Control Devices
PCE	Passenger Car Equivalent
pcu/h	Passenger car units per hour
pcu/h/ln	Passenger car units per hour per lane
s/veh	Seconds per vehicle
veh/h	Vehicles per hour

CHAPTER 1: INTRODUCTION

1.1 Background

The most conflicting manoeuvre at a priority or unsignalized T-intersection is the left turning manoeuvre and its proportion should have the most negative impact on the intersection operation and resulting measures of effectiveness. With advances in computational technology, microscopic simulation models are being widely used by traffic engineers in recent years. Micro-simulation modelling is able to simulate the movement of individual vehicles travelling within a road network through accurate replication of driver behaviour.

According to Lieberman & Rathi (2001), microscopic simulation programs compared with other traffic analysis tools are useful for the evaluation of alternative treatments and the test and visualisation of new designs amongst others. The test and visualisation of new designs are considered valuable in order to gain an understanding and insight into how a traffic network behaves under different conditions. In the local context especially, visualization can be used as a means to explain results related to alternative treatments or designs to non-technical audience who are mostly policy makers like Metropolitan and Municipal Chief Executives. Simulation models also represent a means to describe the dynamic and often complex processes of the traffic system.

1.2 Statement of Problem

The most common type of intersection improvement considered in Ghana is signalization. However, other actions may improve the safety and efficiency of an intersection without the need to signalize in the short term. Potential intersection improvements include installing a left-turn lane (left-turn accommodation). Many a time, implementing this improvement will increase the safety and efficiency of the intersection to such a degree that signalization will no longer be warranted in the short term.

In Ghana, warrants which guide the installation of intersection improvements, such as installing exclusive major and minor road left-turn lanes, are adopted from other countries. These warrants, therefore, do not capture local traffic conditions. Depending on local conditions, a priority T-intersection can experience a wide range of left-turning traffic on the minor and major roads which can have a considerable impact on the operations of the intersection.

The extent of the problem with exclusive left-turn lanes is that road agencies such as the Department of Urban Roads and the Ghana Highway Authority are frequently faced with the decision as to when to install an exclusive left turn lane, either on the major or minor road, to reduce unnecessary delay to through vehicles and also reduce crash risks. Such decisions are mostly taken without detailed engineering analysis of the situation. There is, therefore, the need to establish a local guideline or basis for determining when the need for an exclusive major road left-turn lane or minor road left-turn lane becomes critical. The guideline should also be specific for the intersection type.

This study used VISSIM micro simulation tool to determine the relationship between delay to left-turns and variables such as minor road and major road left-turn volumes. The T-intersection is modelled after calibration and warrants for determining when a major road and minor road left-turn lanes are needed based on left-turn volumes were derived.

1.3 Research Objectives

The objectives of this research were:

1. To calibrate a VISSIM simulation model of the case study T-intersection.
2. To analyze the effect of increasing minor road left-turning traffic volume on minor road delay.
3. To develop a model equation that forecasts threshold minor road left-turn volume warrants for minor road left-turn lane.
4. To develop volume warrants for exclusive major road left-turn lane at a Tintersection.

1.4 Justification

This research developed volume warrants to guide local traffic practitioners in the municipal and metropolitan road departments as well as Consultants who need to make decisions as to when the delay to minor road or major road left-turners warrant the installation of exclusive major road or minor road left-turn lanes at unsignalized T-intersections. In this research, the procedure used can also be adopted in the decision to improve unsignalized T-intersections.

1.5 Scope of Research

The research covered the following:

- The case study T-intersection had a configuration of a two lane major road (width of 7.4m) and a two lane minor road (width of 6.5m).
- Data used for the analysis was collected in the month of April. No local factors were applied for seasonal variations. The collected data was for the morning peak period.

- Projections were made within the limits of the data collected and may not be applicable in all situations.

CHAPTER 2: LITERATURE REVIEW

2.1 Warrant Criteria for Installing Left-Turn Lanes

Many warrant criteria are currently being used to determine the need for left-turn lanes. Previous works reviewed in this study used some common terms which indicated volumes that were used to determine the need for left-turn lanes at unsignalized intersections. These movements are shown in Figure 2.1.

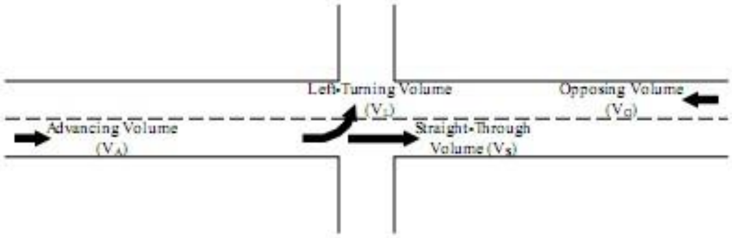

Figure 2.1. Volumes used in left-turn lane warrant methods (Fitzpatrick *et al.*, 2010)

Where,

- Advancing volume (V_A) - The total peak hourly volume of traffic on the major road approaching the intersection from the same direction as the left-turn movement under consideration.
- Left-turn volume (V_L) - The portion of the advancing volume that turns left at the intersection.
- Percent left-turns (P_L) - The percentage of the advancing volume that turns left; equal to the left-turn volume divided by the advancing volume expressed as a percentage.
- Opposing volume (V_O) - The total peak hourly volume of vehicles opposing the advancing volume.

The different warrant criteria for the determination of a left-turn lane are discussed below.

2.1.1 Probability Warrants

The first major study carried out towards the development of left-turn lane warrants was by Harmelink (1967). Warrants from his study are in the form of sets of different volume combinations. These combinations are specifically, the advancing volume (V_A), the percentage of left-turns in the advancing volume (P_L), and the opposing volume (V_O). The warrants were developed for the approach speeds of 40, 50 and 60 mph. The warrants developed by Harmelink (1967) tried to minimize the conflict between the left turning vehicles and through vehicles approaching from behind. To be more precise, these warrants are based on the probability that one or more through vehicles are present in the queue formed by the left-turning vehicles that is waiting for a suitable gap. Harmelink (1967) determined values for the maximum allowable probabilities based upon the judgment of a panel of traffic engineers. He then computed the combination of the three volumes (i.e. advancing, left-turn and opposing volumes) for each value of the probabilities suggested by the panel of traffic engineers he interviewed. This was done analytically on the basis of queuing theory. Harmelink's queuing system assumes that the arriving units are the through vehicles arriving behind the left-turning vehicles, and that the service is the departure of the left-turning vehicles. Harmelink (1967) formulated the arrival rate (λ) and the service rate (μ) of the queuing system as follows: $t^{w+te} \lambda = P_L(1 - L)(V_A)\ (\ _2\ )$ (2.1) $(\ )t_A\ 3$

Where,

P_L = Percentage of left-turning volume in the advancing volume V_A = Advancing Volume (veh/hour) t_w = Average time a left-turning vehicle have to wait to find a suitable gap in the opposing stream t_e = Time required for a left-turning vehicle to clear itself from the advancing queue t_A = The median headway of the advancing stream $t_{w,}$

which is the average time a left-turning vehicle must wait to find suitable gap in the opposing stream is calculated as

$$t_W = \frac{3600}{V_O}\left(e^{\frac{V_O}{3600} G_c} - \frac{V_o}{3600} G_c - 1\right) \qquad (2.2)$$

Where,

V_O = Opposing Volume (veh/hour) G_C =

Critical gap for a particular site (sec) For

the service rate, μ:

$$\mu = \frac{\text{Total unblock time}}{t_1}$$

Where, t_1 = Time taken to complete a

left-turn manoeuvre (sec)

With the arrival and service rates determined from Equations (2.1) and (2.2) above, the probability that one or more units are in the system can be calculated.

Given λ and μ, the probability of k units in the system would be:

$$P(k) = \left(\frac{\lambda}{\mu}\right)^k \left(1 - \frac{\lambda}{\mu}\right) \qquad (2.3)$$

So, the probability of no vehicles behind the left-turning vehicles would be,

$$P(0) = \left(\frac{\lambda}{\mu}\right)^0 \left(1 - \frac{\lambda}{\mu}\right) \qquad (2.4)$$

1 – $P(0)$ will therefore represent the probability of one or more through vehicles behind a left-turning vehicle in the system. The criterion for installing a left-turn lane is based on the probability that one or more units in the system will be less than a given value of ∝. Therefore,

$$1 = P(0) = \frac{\lambda}{\mu} \leq \propto$$

Where the value of $\propto$ is the probability defined in Table 2.1.

Probability values that Harmelink (1967) used to base his warrants are different for different operating speeds as shown in Table 2.1.

Table 2.1. Probability values for Different Operating Speeds for a Two-lane Highway

Approach Speed (mph)		Probability of through vehicles behind left turn vehicle
Design	Operating	
50	40	0.02
60	50	0.015
70	60	0.01

Source: Harmelink (1967)

For example, Harmelink (1967) concluded that if the probability of one or more through vehicles present behind the left-turning vehicle is greater than 0.02 for 40mph operating, an exclusive left turn lane is justified.

Since the average arrival rate is the function of left-turn volume/advancing volume and the average service rate is the function of opposing volume, the relationship between left-turn volume/advancing volume and opposing volume can also be

derived. He expressed these in a series of design charts.

AASHTO (2001) summarized the left-lane graph developed by Harmelink (1967) into a table as shown in Table 2.2.

Table 2.2. Guide for left-Turn Lanes on Two-Lane Highways

Opposing Volume(veh/h)	Advancing Volume (veh/h)			
	5% left Turn	10% Left Turn	20% Left turn	30% Left Turn
40-mph Operating Speed				
800	330	240	180	160
600	410	305	225	200
400	510	380	275	245
200	640	470	350	305
100	720	515	390	340
50-mph Operating Speed				
800	280	210	165	135
600	350	260	195	170
400	430	320	240	210
200	550	400	300	270
100	615	445	335	295

Opposing Volume (veh/h)	Advancing Volume (veh/h)			
	5% left Turn	10% Left Turn	20% Left turn	30% Left Turn
60-mph Operating Speed				
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

Source: AASHTO (2001)

For example, for a two lane highway having an operating speed of 50 mph with an advancing volume of 195 vph and an opposing volume of 600 vph, the minimum left turn volume warranting a major road left turn lane is 39 vph.

The Highway Capacity Manual (HCM) (2000) cites warrants from AASHTO (2001) as its guideline for determining when to install a left-turn lane.

Kikuchi and Chakroborty (1991) critically evaluated Harmelink's warrants and pointed out a number of problems with his model. The first flaw they pointed out in their study concerns the inconsistent definitions of the arrival and service rates. In queuing theory, both the arrival and departure rate should have the same units. This, however, is not the case in the Harmelink's model. As mentioned above, in Harmelink's model the arrival rate refers to the through vehicles behind the left turning vehicles, whereas the service rate refers to the left-turning vehicles. This inconsistency leads to error in results when more than one through vehicle is queued behind the left-turning vehicle.

The second flaw concerns the issue of residual gaps. In the Harmelink's model, the service rate is derived by dividing the sum of gaps that are greater than the critical gap by the time required for completing a left-turn manoeuvre. The problem here, as pointed by Kikuchi and Chakroborty (1991), is that the residual gaps (i.e. the remainder of individual gap after subtracting the value of the critical gap) are added up and that the sum is also considered to be part of the time available for making leftturns. This makes the service rate (μ) represent more left-turn opportunities than are actually available. For example, suppose there are a total of four gaps of seven seconds each available in the opposing traffic and the time required for completing the left turn manoeuvre is four seconds, then, according to Harmelink's equation, seven left-turning vehicles would be served in that period of time but, practically, only four vehicles will depart in that much time.

Finally, the different values of the parameters used by Harmelink's (such as critical gap headway, average time a left-turn vehicle has to wait before finding a suitable gap in the opposing traffic and time required to clear advancing lane correspond to conditions of the roads and state of vehicles decades ago (i.e. in 1967) may not be applicable to the current state of roads as well as vehicles. In addition, the warrants were developed primarily for rural areas and their application to the urban setting may, therefore, be inappropriate.

To address the two main theoretical flaws, Kikuchi and Chakroborty (1991) first suggested a more refined analytical formulation that avoids the two theoretical flaws of

Harmelink's model. The newly developed equations by Kikuchi and Chakraborty (1991) used arrival and departure rates which have consistent units and also make sure that the residual gaps are not added up leading to erroneous results. According to the newer formulation, the arrival rate (λ) is the number of arriving units per unit time. One arriving unit is a left turning vehicle followed by one or more through vehicles. The departure rate (μ) is the departure of the arriving units per unit time. Using this newer formulation, they revised the volume warrants based on the probability values suggested by Harmelink (1967) as shown in Table 2.3.

Table 2.3. Modified volume warrants by Kikuchi and Chakroborty (1991) based on Harmelink's study

Opposing Volume(veh/h)	Advancing Volume (veh/h)			
	5% left Turn	10% Left Turn	20% Left turn	30% Left Turn
40-mph Operating Speed				
800	434	300	219	189
600	542	375	272	134
400	682	472	343	293
200	863	600	435	375
100	946	679	493	424
50-mph Operating Speed				
800	366	257	185	162
600	460	320	234	202
400	577	403	294	255
200	735	513	373	324
100	830	576	424	365
60-mph Operating Speed				
800	294	207	154	146
600	365	259	187	165
400	461	324	238	206
200	586	414	303	263
100	663	468	344	297

Source: Kikuchi and Chakroborty (1991)

Lakkundi *et al*. (2004) of the Virginia Transportation Research Council developed new left-turn guidelines for both signalized and unsignalized intersections. Their warrants were developed on the basis of a well-validated, event-based simulation programs

“LTGAP” which the authors developed themselves and calibrated based upon field data collected at a number of intersections from the Commonwealth of Virginia. One advantage of their study over Harmelink (1967) and Kikuchi and Chakroborty (1991) was that they used more accurate modeling techniques which incorporated a stochastic gap acceptance module. They calibrated the models based on the number of left-turning vehicles stopped on the subject link.

For unsignalized intersections, their study developed left-turn lane volume warrants based on the probability criteria as suggested by Harmelink. They also developed warrants for signalized intersections. They reckoned that, if it is decided that an exclusive left-turn lane at a particular intersection should be provided, the length of the lane also needs to be determined. So their study also recommended the length of the proposed left turn lane. Since the purpose of installing a left turn lane is to prevent left-turn overflows, the probability of left-turn lane overflows for varying left-turn lane lengths was investigated, which was later used to recommend the left-turn lane length for the candidate intersections. In addition to the general guidelines, Lakkundi *et al*. (2004) also developed a prioritization tool that can be used to prioritize candidate intersections to accounts for both operational and safety aspects.

Despite extensive improvements over the previous attempts at developing left turn lane warrants, the warrants for their study was still based on the probability criterion suggested by Harmelink. As pointed out by Kikuchi and Chakroborty (1991), this practice is somehow flawed.

2.1.2 Benefit-Cost Warrant

Fitzpatrick *et al.* (2010), as published in the National Cooperative Highway Research Program, developed Left-Turn Lane Warrants for Unsignalized Intersections. Their study used benefit-cost (B/C) approach to justify the installation of a left-turn lane.

They indicated that the two major benefits in installing a left-turn lane are benefit in crash reduction (safety benefit) and operational benefit (benefit in reduction in delay and improved left-turn capacity when stopped vehicles making left-turns are removed from the main travel lane). They evaluated benefits in crash reduction by predicting average crash frequency for base conditions i.e. without a left-turn lane and with a left-turn lane. In order to assess the benefits in terms of delay savings, computer simulation was used to evaluate the benefits at a case study intersection when a left- turn lane was provided for left turners. The average delay to left-turners, when a left- turn lane was installed, was deducted from the average delay to left-turners when a left-turn lane was absent. The difference represented the average delay savings per vehicle. The research proposed an average value of $250,000 as a typical cost of constructing a left-turn lane. Using the following equation, the benefit-cost ratio over a specified period was calculated:

$$\frac{B}{C} = \frac{\text{Delay reduction} + \text{Safety Improvements}}{\text{Construction Costs}} \qquad (2.5)$$

An economic criteria for installing a left-turn lane was established by determining the level of peak hour major road volume and peak hour conflicting left-turn volume that will result in a Benefit-Cost ratio equal to or greater than 1.0. Their study suggested that a left-turn lane is considered economically justified when the Benefit-Cost ratio of installing a left turn lane is equal to or greater than 1.0 because the benefits are greater than the cost.

The developed left-turn lane warrants using a benefit-cost ratio of 1.0 from the research of Fitzpatrick *et al.* (2010) for an urban and suburban three leg intersection is shown in Figure 2.2.

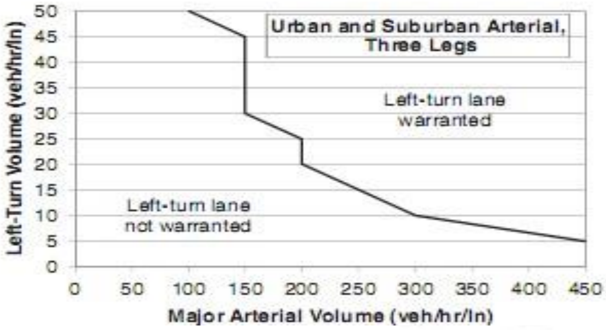

Figure 2.2. Recommended left-turn lane warrant for an urban unsignalized three leg intersection (Fitzpatrick *et al.*, 2010).

2.1.3 Accident Warrant

Agent (1982) developed a warrant for left-turn lane based on accident experience. He collected data for five years at intersections in Lexington, Kentucky. Accident rates at locations with and without left-turn lanes were calculated. This was done using volume counts at intersections for a 12-hour period (7 a.m. to 7 p.m.). With an assumption that 80 percent of the volume occurred in this 12-hour period, he then multiplied these volumes by 1.25 to obtain the 24-hour volume. Using the collected data base, accident rates (left-turn accidents per million left turning vehicles) were calculated for intersections with and without left-turn lanes.

Accidents were based on the following definitions: (a) when a left-turning vehicle turned into the path of an oncoming vehicle, (b) when a left-turning vehicle was struck in the rear while waiting to turn, or (c) when a vehicle weaving around a vehicle stopped to turn left was involved in an accident. His study revealed that, lefttum accident rate dropped significantly for intersections with left-turn lanes. For unsignalized intersections, the left turn accident rate was 77 percent lower. The rate was 54 percent lower at signalized intersections. At signalized intersections, the rate dropped even lower when left-tum phasing was added. Using the same data base, the average number of left-turn accidents for the approaches with no left-turn lanes was calculated. Using the average number of left-tum accidents, he determined the critical number of accidents. For unsignalized intersections, the study found the average number of accidents to be 0.8 left-tum accidents per approach per year.

He derived Equation (2.6) below to determine the critical number of left-turn accidents warranting a left turn lane.

$$Nc = Na + K\sqrt{Na} + 0.5 \qquad (2.6)$$

Where,

N_C = critical number of accidents

Na = average number of accidents and

K = constant related to level of statistical significance selected (for $P = 0.95$, $K = 1.645$; for $P = 0.995$, $K = 2.576$).

He concluded that for $P = 0.995$, the critical number of left-turn accidents in one year necessary to warrant installation of a left-turn lane is four at an unsignalized intersection.

2.1.4 Delay Warrants

Kikuchi and Chakroborty (1991), in addition to modifying the work of Harmelink (1967), developed a set of volume warrants based on delay to through vehicles caused by left-turn traffic and degradation of Level of Service from A to B as warrant criteria. In order to develop a delay warrant, the researchers first developed their own simulation model, because commercial models that time had several limitations regarding modeling unsignalized intersections as well as computing the different delay values. They then developed warrants from the model's output. The measures of effectiveness used for the calibration of the model were average delay to left turning vehicles and number of vehicles caught behind the left turning vehicles. A minimum delay threshold of 14 seconds was used even though the justification behind it was not stated in their report.

For degradation of level of service as a warrant criterion, the developed warrants were based on the degradation of Level of Service from A to B based on different volume combinations of advancing, opposing and left-turn volumes. Ivan *et al.* (2009) however, disagree to this and argue that this criterion may not seem to be reasonable for traffic engineers, as in the field, a level of service of C is considered acceptable.

Ivan *et al.* (2009) used CORSIM simulation software to develop Warrants for Exclusive Left-Turn Lanes at Unsignalized Intersections and Driveways based on control delay and number of stops during the peak hour. They varied the opposing and advancing volumes between 100 and 800 vehicles per hour per lane in increments of 100 per hour per lane. For left-turning percentages, values of 5%, 10%, 20% and 30% and for operating speeds values of 30 mph, 40 mph and 50 mph were used.

In setting up the thresholds for the control delay and number of stops per hour, they considered the following points. First, they found it necessary to look at the rate of change in the delay and number of stops with respect to the opposing, advancing, left turning volumes and operating speeds. For that, they plotted total delay and the total number of stops on the subject link against the various combinations of advancing, opposing, left-turning volumes and different operating speeds. They kept the thresholds selected constant regardless of the volumes and category (e.g. urban twolane and rural two-lane categories had same thresholds), but varied the operating speeds. For example, volume combinations for 30 mph speed were higher than the volume combinations for 40 mph speed as the thresholds are higher for the former.

The thresholds were set just below the point at which the curves for delay and the number of stops rose sharply for the relatively high opposing volumes. A sample graph showing how they determined the threshold for delay is shown in Figure 2.3. This graph is for the case of an operating speed of 50 mph and a left-turning percentage of 30%.

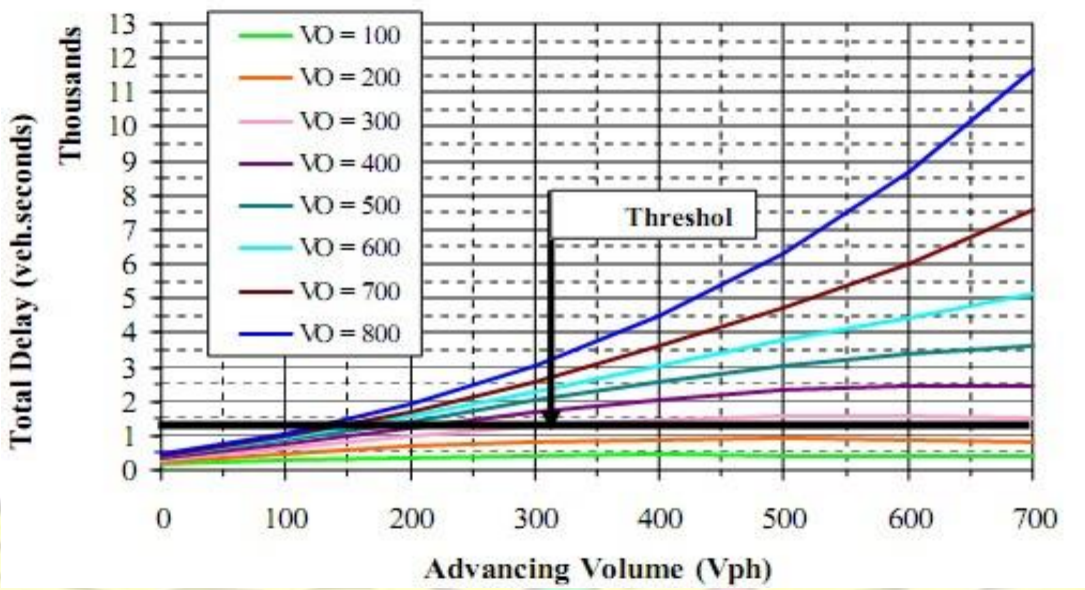

Figure 2.3. Delay plotted for urban two lane category (Ivan *et al.*, 2009).

Using the thresholds values, warrants were developed for urban two-lane category based on total delay (sec/hour) for operating speeds of 30 mph, 40 mph and 50 mph respectively.

Agent (1982) also used delay to determine guidelines on when to install a left-turn lane on a four-lane highway and two-lane highway unsignalized intersections. He considered delay to left-turning vehicles and suggested that the critical delay is the delay at which left-turn delay started to increase drastically and represents the delay at which a left-turn lane should be considered. He then determined the minimum sum of peak-hour left-turn and opposing volumes which resulted in the critical delay.

Agent's procedure was based on simulation. In this procedure, the computer input specified that 100 percent of the volume on the left-turn approach turned left while 100 percent of the opposing volume went straight through. Delay to the left-turn vehicles was determined. Figure 2.4 is a graph showing the results of Agent's simulation experiment.

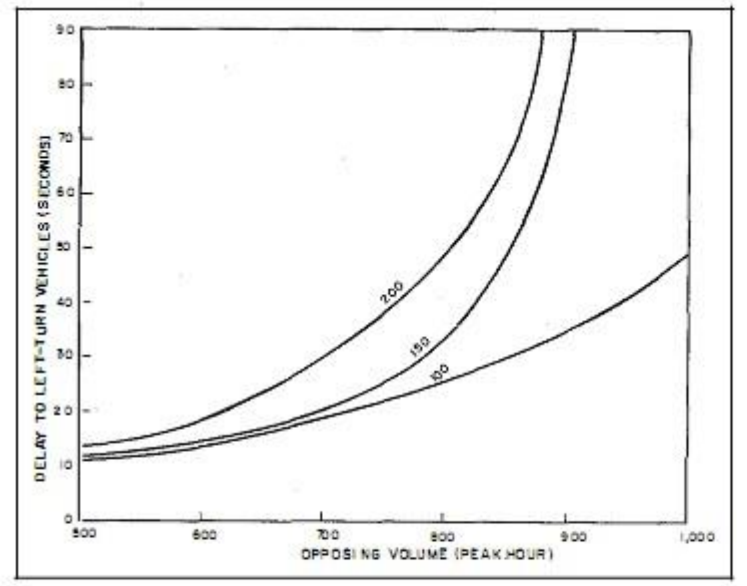

Figure 2.4. Left-turn delay as a function of opposing and left-turning volume for two-lane unsignalized intersection (Agent, 1982).

He took 20s/veh as the delay point at which left-turn delay started to increase drastically and, therefore, the critical delay. From Figure 2.3, the points at which the opposing and left-turning volume resulted in a critical delay of 20s/veh were taken from the graph and is presented in the Table 2.4.

Table 2.4. Volume warrants for major road left-turn lane

Opposing Volume (veh/h)	Major road left-turns (veh/h)
600	200
700	150

800	100

Source: Agent (1982)

Agent (1982) determined the minimum sum of peak hour left-turn and opposing volumes which resulted in a critical left-turn. His result is presented in Table 2.5

Table 2.5. Critical Sum of left –turn and Opposing Volumes during the Peak Hour for creating a left –turn Delay Problem

Unsignalized Intersection		
Delay Criterion	Two-Lane Highway	Four-Lane Highway
20 seconds	800	900

Source: Agent (1982)

2.1.5 Summary and Comparison of Different Warrants from literature review

A summary and comparison of different warrant criteria for installing a major road left-turn lane reviewed in literature is presented in Table 2.6.

Table 2.6. Summary and Comparison of Different Warrant Criteria

Studies	Major Criteria	Basic Assumptions	Influencing Factors
Harmelink (1967) & AASHTO (2001)	Volume-based warrants for unsignalized intersections: Opposing, advancing and left-turn traffic volume	The probability of more than one/two left-turning vehicles waiting for making a left turn should be lower than a specific level. One vehicle is the threshold no. for two-lane highways.	Traffic volume: opposing, left-turn, advancing • Speed • Number of lanes
Kikuchi and Chakroborty (1991)	Volume-based warrants for unsignalized intersections: Opposing, advancing and left-turn traffic volume	Probability criteria as suggested by Harmelink (1967)	Traffic volume: opposing, left-turn, advancing • Speed • Number of lanes
	Volume-based warrants for unsignalized intersections: Opposing, advancing and left-turn traffic volume	Left-turn lanes should be installed if intersection delay is more than critical delay value.	Traffic volume: opposing, left-turn, advancing Speed Number of lanes
	Volume-based warrants for unsignalized intersections: Opposing, advancing and left-turn traffic volume	Degradation of Level of Service from A to B as warrant criteria	Traffic volume: opposing, left-turn, advancing Speed Number of lanes
Lakkundi *et al*. (2004)	Volume-based warrants: Opposing, advancing and left-turn traffic volume	Probability criteria as suggested by Harmelink (1967)	• Traffic volume: opposing, left-turn, advancing • Number of lanes
Fitzpatrick *et al*. (2010)	Volume-based warrants: Opposing, advancing and left-turn traffic volume	Benefit- Cost (B/C) ratio of installing a left-turn lane should be equal to or greater than 1.0	• Traffic volume: opposing, left-turn, advancing • Number of lanes
Ivan *et al*. (2009)	Volume-based warrants: Opposing, advancing and left-turn traffic volume	Control delay and number of stops per hour should be lower than a critical value.	Traffic volume: opposing, left-turn, advancing Number of lanes

Agent (1982)	Accident-based warrant: historical rates of the leftturn related accidents	Left-turn lanes should be installed if the critical number of left-turn related accidents had occurred.	Historical rates of the left-turn-related accidents
	Volume-based warrants: Sum of opposing and advancing traffic volume	The intersection delay to left-turn traffic should be lower than a critical value.	• Traffic volume: opposing, left-turn, advancing • Number of lanes

Source: From literature review

A review of existing literature on warrants for exclusive left turn lane revealed that volume warrants based on different criteria are different. Benefits of safety improvements cannot be easily quantified in Ghana due to unreliable accident data which makes the Benefit-Cost ratio approach difficult to adopt. The probability criterion is somehow subjective. However, delay is a more easily understandable measure of traffic performance than probability. If delay is known, the warrant will be more meaningful to engineers, planners and the general public. The volume combinations based on delay criterion was considered in this study.

2.2 Effect of Minor Road Left Turning Traffic on Minor Road Delay

Few research has been conducted to evaluate the effect of minor road left-turn manoeuvres on resulting intersection measures of effectiveness such as delay at priority intersections. Cvitanic *et al.* (2004) analysed priority T-intersection operations for Manual on Uniform Traffic Control Devices (MUTCD) traffic volume threshold values using SYNCHRO software. In order for them to compare the functionality of priority intersections, generic traffic and geometry charateristics of intersections were developed. On T-intersections, the proportions of 10 and 50 percent of left-turners on the higher volume minor road were analysed for different volume combinations on the major road. Their crietria for chosing optimal traffic signal scheme was based on average overall delay of vehicles.

Their study indicated that:

- The proportions of left-turn manoeuvres on the minor and major road in addition to the total volume on the major road exert the most significant impact on the quality of the intersection.
- Choosing average overall delay of vehicles on all approaches as a criterion for optimal intersection control type can be misleading because a situation exists when the overall average delay at a priority intersection is within acceptable limits even though the minor road vehicles suffer unacceptable delays.

Based on their results, the authors suggested possible improvements of traffic signal warrants by including composition of minor road left turning manoeuvres and the need for separate threshold values for T-intersections.

Zhu *et al.* (2010) in their study applied microscopic simulation to evaluate critical volumes used in the four-hour warrant of the Manual on Uniform Traffic Control Devices (FHWA, 2000). Their research compared minor road control delay for three turning percentages ranging from low to high left and right-turn vehicles on the minor road. They found that the higher the left-turn percentages, the higher the average delay experienced by the minor road vehicles. Also, the difference in delay on minor road vehicles becomes increasingly larger with higher major road volumes.

For these reasons, they suggested a possible need to revise the critical design values of the current four-hour volume warrant by considering turning percentages on the minor road.

2.3 Traffic Simulation Software

According to Merritt (2003), traffic simulation models can be categorised in three groups based on the level of detail that each model represents. He classified simulation software tools into macroscopic models, mesoscopic models and microscopic models.

He explained that macroscopic models describe system entities and their interactions at a low-level of detail; mesoscopic models describe dynamic system entities at a relatively high-level of detail than macroscopic models but at a lower level of detail as compared to microscopic models. Microscopic models describe entities and activities at a high-level of detail. He further explained that microscopic models can represent elementary behaviours such as: car-following, gapacceptance, and lane-changing as well as variation in road-user behaviour and vehicle performance.

Some background on features of common simulation software is necessary to understand the differences in capabilities and performance. This review of the models is developed primarily from manuals available from the developers of the packages.

2.3.1 CORSIM

CORSIM is a comprehensive traffic simulation package which models surface streets, freeway systems, and combined networks having simple or complex control conditions. The strengths of the model lie in its ability to simulate a wide variety of traffic conditions from signalized arterial corridors and freeway corridors to stop-controlled intersections (McTrans, 2016).

2.3.2 SIM TRAFFIC

SIM TRAFFIC is a microscopic simulation package that uses the outputs of SYNCHRO program to model street networks. Jones *et al.* (2004), in comparing different traffic simulation models, argue that the greatest difference between CORSIM and SIM TRAFFIC lies in the car-following algorithm. The study stated that CORSIM tries to maintain a fixed headway between vehicles, one that varies based on driver type but averages around 1 second for all speeds and driving conditions. SIM TRAFFIC also attempts to maintain a fixed headway between vehicles, but those headways vary based on speed, driver type, and link geometry. In general, this leads to SIM TRAFFIC generating saturation flow rates (and therefore roadway capacities) lower than those found in CORSIM. Practically, what this means is that, for a given segment with fixed traffic volumes, CORSIM will tend to generate higher link capacities (and therefore less congestion) than SIM TRAFFIC.

2.3.3 VISSIM

VISSIM is a discrete, stochastic, time-step based microscopic simulation tool with a broad range of simulation capabilities. Like CORSIM and SIM TRAFFIC, it can simulate surface street networks, freeways, interchanges, weaving sections, pre-timed and actuated signals, stop-controlled intersections, and roundabouts. Networks in VISSIM are represented through a series of links and connectors. Generally, when a model is created, these links and connectors are laid over a satellite image of the network being modelled. Links can be single or multi-lane roadway segments with traffic flow in only one direction. Vehicles can only travel from one link to the next over a connector. It also has features not contained in either CORSIM or SIM TRAFFIC, such as 3-D capabilities.

VISSIM software was chosen to simulate the case study intersection. This is because VISSIM is better in terms of ease of use and does not require cumbersome coding (Ratrout & Rahman, 2008). Also in a study by Kaseko (2002) in his comparison of VISSIM, CORSIM and SIM TRAFFIC for three facility types: freeways, interchanges and arterials with coordinated signals, he stated that VISSIM was the most powerful and versatile.

2.4 Theoretical Background of VISSIM

VISSIM is a microscopic, time-step oriented, and behaviour-based simulation tool for modelling urban and rural traffic as well as pedestrian flows. The traffic flow is simulated under various constraints of lane distribution, vehicle composition, signal control etc. VISSIM is based on a traffic flow model and light signal control. The traffic flow model defines a car-following model for the modeling of driving in a stream on a single lane and the lane changing model.

2.4.1 Operating Principles of Car Following Model

Vehicles move in a network using a traffic flow model. VISSIM's traffic flow model is a stochastic, time step based, microscopic model that treats driver-vehicle units as basic entities. The traffic flow model contains a psycho-physical car following model for longitudinal vehicle movement and a rule-based algorithm for lateral vehicle movement. The quality of the traffic flow model is essential for the quality of the simulation. In contrast to simpler models in which a largely constant speed and a deterministic car following logic are provided, VISSIM uses the psycho-physical perception model. The basic concept of this model is that the driver of a faster moving vehicle starts to decelerate as he reaches his individual perception threshold to a slower moving vehicle. Since he cannot exactly determine the speed of that vehicle, his speed will fall below that vehicle's speed until he starts to slightly accelerate again after reaching another perception threshold. There is a slight and steady acceleration and deceleration. The different driver behaviour is taken into consideration with distribution functions of the speed and distance behaviour. The car following model has been calibrated through multiple measurements at the Institute of Transport Studies of the Karlsruhe Institute of Technology, Germany. Recent measurements ensure that changes in driving behaviour and technical capabilities of the vehicles are accounted for.

VISSIM simulates the traffic flow by moving driver-vehicle-units through a network. Every driver with his specific behaviour characteristics is assigned to a specific vehicle. As a consequence, the driving behaviour corresponds to the technical capabilities of his vehicle. Wiedemann's traffic flow model is based on the assumption that there are basically four different driving states for a driver:

- Free driving: No influence of preceding vehicles can be observed. In this state, the driver seeks to reach and maintain his desired speed. In reality, the speed in free driving will vary due to imperfect throttle control. It will always oscillate around the desired speed.
- Approaching: Process of the driver adapting his speed to the lower speed of a preceding vehicle. While approaching, the driver decelerates, so that there is no difference in speed once he reaches the desired safety distance.
- Following: The driver follows the preceding car without consciously decelerating or accelerating. He keeps the safety distance more or less constant. However, again due to imperfect throttle control, the difference in speed oscillates around zero.
- Braking: Driver applies medium to high deceleration rates if distance to the preceding falls below the desired safety distance. This can happen if the driver of the preceding vehicle abruptly changes his speed or the driver of a third vehicle changes lanes to squeeze in between two vehicles.

For each of the four driving states, acceleration is described as a result of current speed, speed difference, distance to the preceding vehicle as well as of individual driver and vehicle characteristics. Drivers switch from one state to another as soon as they reach a certain threshold that can be described as a function of speed difference and distance. For instance, small differences in speed can only be perceived at short distances.

Whereas large differences in speed already force drivers to react at large distances. The perception of speed differences as well as the desired speed and safety distance varies across the driver population. Because the model accounts for psychological aspects as well as for physiological restrictions of drivers' perception, it is called psycho-physical car-following model.

2.4.2 VISSIM Software Calibration

In order for a traffic simulation to accurately describe reality, it must utilize a valid model and be properly calibrated. A valid model implies that the underlying simulation logic reasonably reflects real-world operations. A calibrated simulation means that the input parameters provided by the user (e.g. delay or desired speed) allow the simulation program’s model to recreate the specific network under consideration.

The FHWA (2007) guidelines for applying traffic simulation modelling give a basic strategy for calibration. Even though this strategy was developed for CORSIM micro simulation, it is also applied to VISSIM. The guide recommends that the first step in calibration is to identify the calibration Measures of effectiveness (MOEs) and thresholds where the difference between the field and model MOEs are acceptable. Once the calibration MOEs have been identified and thresholds have been established, it recommends the following three basic steps for calibration:

- Calibrate capacity at key bottlenecks: This step calibrates the capacity of key bottlenecks in the network. These bottlenecks are responsible for the majority of congestion (and thus delays and queuing) in the model. The guidelines revealed that a model must match the capacity at a bottleneck of the case study network otherwise it will be nearly impossible to calibrate the system performance MOEs. It recommends that the capacity should be estimated by measuring the maximum throughput at the bottleneck location and that the throughput should only be collected when a queue is continually present upstream of the bottleneck, as this is the only time when throughput can be used to determine the capacity of a given location.

- Calibrate traffic volumes: This is needed to ensure that the throughput volumes of the model match those in the field.
- Calibrate system performance: This step calibrates the model's Measures of Effectiveness (MOEs) of interest that were used to measure system performance to ensure that the model as a whole reasonably matches local conditions. The guide recommends that the selection of MOEs for calibration should be limited to those that can be practically collected in the field and that even if the measure of effectiveness is a critical measure of effectiveness and cannot be measured in the field, then it should not be included as a calibration measure of effectiveness. It states typical system performances MOEs as speed, density, travel time, delay, and queue length. Under these steps, MOEs measured in the field are compared to the MOEs estimated by the software and, if they do not meet the established targets, then the model parameters are systematically adjusted until an acceptable match is found. This process continues through each of the three basic steps until all calibration targets have been met.

Oketch *et al.* (2004) in an evaluation of the performance of modern roundabouts using PARAMICS micro simulation model used the GEH test statistic which is a modified Chi-squared statistic that incorporates both relative and absolute differences in comparing modelled and simulated flows for calibration. The study advocated the use of GEH statistic in comparing hourly traffic volumes only.

It is represented by the equation:

$$GEH = \sqrt{2(E-V)^2/(E+V)} \tag{2.7}$$

Where E = Model Simulated Flow

V = field Count

GEH< 5 – flows can be considered a good fit

5<GEH<10- flows may require future investigation

10<GEH – flows cannot be considered a good fit

2.5 Field Measurements of Delay at Intersections.

According to FHWA (2007) the best source of travel time data is "floating car runs." In this method, one or more vehicles are driven the length of the facility several times during the analytical period and the mean travel time is computed. The number of vehicle runs required to establish a mean travel time within a 95-percent confidence level depends on the variability of the travel times measured in the field. Free-flow conditions may require as few as three runs to establish a reliable mean travel time. Congested conditions may require 10 or more runs.

The minimum number of floating car runs needed to determine the mean travel time within a desired 95-percent confidence interval depends on the width of the interval that is acceptable to the analyst. If the analyst wishes to calibrate the model to a very tight tolerance, then a very small interval will be desirable and a large number of floating car runs will be required.

CHAPTER 3: METHODOLOGY

3.1 Site Selection and Description

Several priority T-intersections in Kumasi were visited. The intention was to identify a suitable site for data collection purposes. After careful site observations, selection of the site to be studied was based on the following criteria:

(a) Good access and safety for the enumerators and equipment during the data collection process,

(b) Reasonable traffic volumes on both major and minor approaches so that good quality data is obtained, and

(c) Good sight distances (to ensure that the sight distances do not influence the interactions between drivers).

(d) Saturated queues during peak conditions

Based on the above criteria, the study intersection was selected. The study intersection is a priority T-intersection on the Southern Bypass located at Dakwodwom (Dr. Osei Kufuor/Obei Nkwantabisa Intersection). Figure 3.1 shows the location of the intersection.

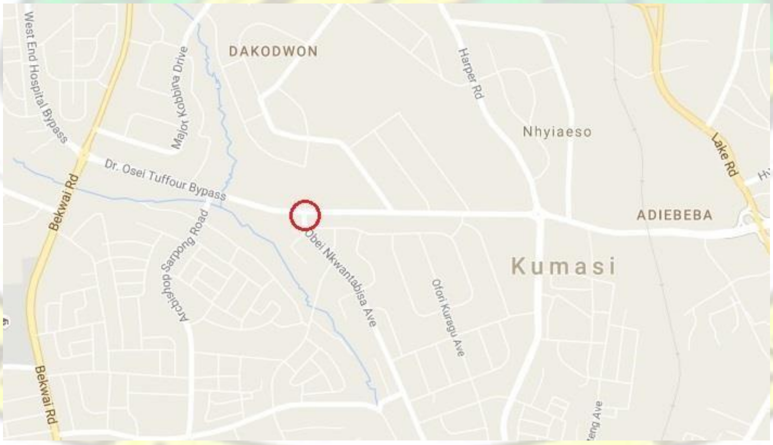

Figure 3.1. Google map showing the location of case study intersection (circled)

For study purposes, the approaches are labelled west approach, east approach and south approach. The east approach (from Ahodwo) and west approaches (from Santasi) will be referred to as the major road in this study and the south approach (from Adiembra) will be referred to as the minor road. Both are two lane single carriageways. Figure 3.2 shows the intersection configuration and layout.

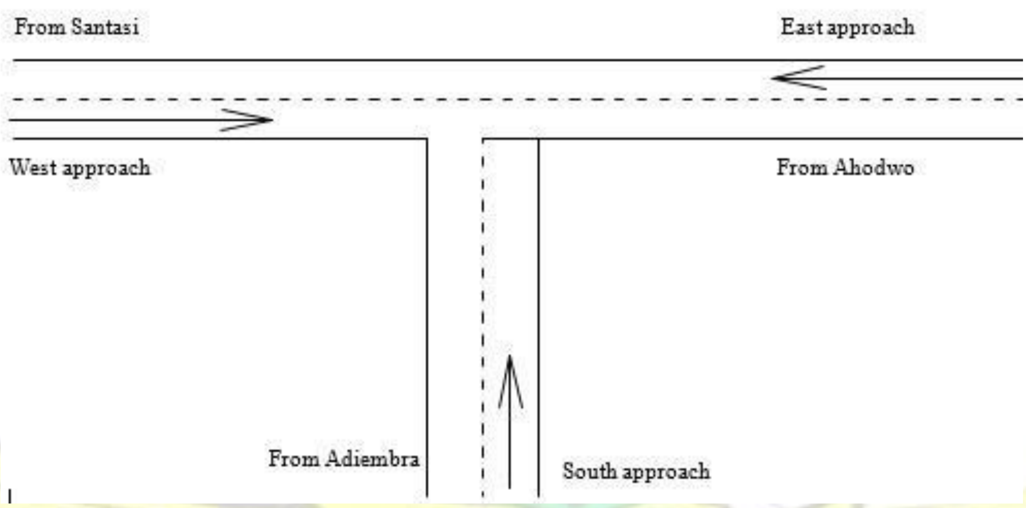

Figure 3.2. Case Study Intersection Configuration and layout

3.2 Data Collection

VISSIM microscopic simulation model has many model parameters. In order to build a VISSIM simulation model for the selected intersection and to calibrate it for the local traffic conditions, two types of data are required. The first type is the basic input data which include data on network geometry, traffic volume, turning movements and vehicle composition. The second type is the observation data employed for the calibration of simulation model parameters such as delay and queue length using standard procedures.

3.2.1 Geometric Data

Geometric data of the T-intersection was measured on site using a tape measure.

Table 3.1 shows the basic geometric element of the T-intersection.

Table 3.1. Basic geometric parameters of the case study T-intersection

	APPROACH		
	West	East	South
Approach Width	3.7m	3.7m	3.25m

3.2.2 Traffic Demand Data

In this study, traffic volumes on the major and minor roads were collected using a digital video camera. The traffic volume was recorded for two hours during the morning peak period from 8 a.m. to 10 a.m. A video camera was mounted on a tripod in such a way as to obtain a good view of all the three approaches of the intersection. The video recording was played back on a computer and analysed manually by trained observers. Traffic composition and turning movement counts were made for all three approaches from the recording using suitable forms. The traffic compositions were grouped into cars: Taxis, Pickups & Saloon Cars; medium vehicles: small bus, "tro-tros'' or minibus and large vehicles: long buses and heavy goods vehicles. The recorded volume of vehicles per 15-minute time periods were converted into passenger car units. The passenger car equivalent (PCE) values of 1.0 for cars, 1.7 for medium vehicles and 2.5 for large vehicles developed by Adams & Obiri-Yeboah (2008) from traffic studies done in Kumasi were used. Because there was no queue on the major road approaches, the demand was taken as equivalent to the traffic volume (HCM, 2000). The minor road approach, however, had queue so in order to get a measure of the true demand on the minor road approach, queue length study and the turning movement were done on the minor road approach for every 15-minute time period. The recorded queue length on the minor road approach every 15-minutes was divided by 6 m to get the equivalent number of passenger car units in the queue. This was then added to the entry flow to get the demand on the minor road approach. The 6 m was obtained by adding the length of a standard small vehicle which was taken as 4.7m according to the Ghana Highway Authority Road Design Guide (GHA, 1991) and an assumed clearance of 1.3 m between queued vehicles on the minor road approach. Tables 3.2 and 3.3 show the demand data and peak hour turning movement data during the peak morning hours respectively.

Table 3.2. Approach demand data in pcu/h

MORNING PEAK PERIOD	SOUTH APPROACH			EAST APPROACH	WEST APPROACH	ALL MOVEMENTS
TIME	ENTERING VOLUME	QUEUE	SOUTH APPROACH TOTAL	ENTERING VOLUME	ENTERING VOLUME	
8:00AM-9:00AM	574	79	653	590	618	1861
8:15AM-9:15AM	570	75	645	627	596	1868
8:30AM-9:30AM	551	49	600	615	602	1816
8:45AM-9:45AM	537	49	586	625	628	1838
9:00AM-10:00AM	533	50	583	659	635	1877

NB: The shaded portion represents the morning peak hour period

Table 3.3. Turning movement data during the morning peak hour

INTERSECTION DESIGN TRAFFIC DATA (9:00am-10:00am)						
Intersection: Dr. Osei Tuffour Bypass/Osei Nkwantabisa Avenue (Dakwadwom)						
MOVEMENT	DIRECTION	PCU/h	% of Approach volume	Approach volume	% of Total Intersection volume	Total Intersection Peak Volume
East Approach	Left	173	26	659	35	1877
	Through	487	74			
West Approach	Through	411	65	635	34	
	Right	225	35			
South Approach	Left	309	53	583	31	

	Right	274	47			

The west and south approaches did not have persistent queues and that the entering volumes were considered equivalent to the true demand.

3.2.3 Delay Data

Delay considered in this study is the average travel time difference it takes vehicles to travel between a marked control point upstream of the queue in a lane and the stop line in queued condition and free flow condition. Junction delay is a measure of junction performance, usually presented in the form of average delay per vehicle. A pilot survey was done on the survey link (minor road approach). This was to familiarise the survey team with the method and route.

At the actual day of survey, a survey car was driven along the minor road, at typical speed of other cars during the peak morning period and free flow period i.e. when there was no queue. Surveyors in the car recorded the time it took the vehicle to travel between two pre-determined control points. In addition, the distance between the control points was measured on the road using a distance measuring wheel (precimeter). For the two-hour period, four runs of the vehicle were done. The average of runs for free flow condition was then deducted from the average of the runs for queue condition to obtain the average delay in seconds per vehicle.

3.2.4 Queue Length Data

For the purpose of this study, the queue length is defined as the distance of the rear end of the furthest stopped vehicle from the stop line. Figure 3.3 shows vehicles in a queue on the minor road at the study intersection.

Figure 3.3. Vehicles in queue on minor road approach

The number of stopped vehicles queueing on the minor road was counted at fixed intervals, every minute over a period of two hours. Preliminary site visits revealed the queue length limit during the morning peak period. The Minor road was divided into smaller sections. The distance between the stop line and the normal queue limit upstream was marked at 10 m interval using red paint and a distance measuring wheel (precimeter) as shown in Figure 3.4.

Figure 3.4. Minor road approach being marked at 10m interval

A trained observer then recorded the distance of the furthest stopped vehicle from the stop line in order to determine the queue length as shown in Figure 3.5.

Figure 3.5. Layout of Queue Markers along minor road approach

3.3 VISSIM Model Development

The existing T- intersection was modelled in VISSIM by using an aerial image of the site as shown in Figure 3.6 below.

Figure 3.6. Existing intersection modelled in VISSIM

The intersection was modelled by importing the aerial image into the VISSIM program and scaling it. The links which represent road segments that carry through traffic and the general curvature of the roadway were drawn over the scaled image. These links were joined by connectors. A connector is a type of link used to join two areas of a single link or to join two areas of two links to allow for a continuous traffic flow. Figure 3.7 shows the modelled intersection in wireframe display mode. In wireframe view, the intersection is represented by blue and pink lines showing the links and connectors respectively.

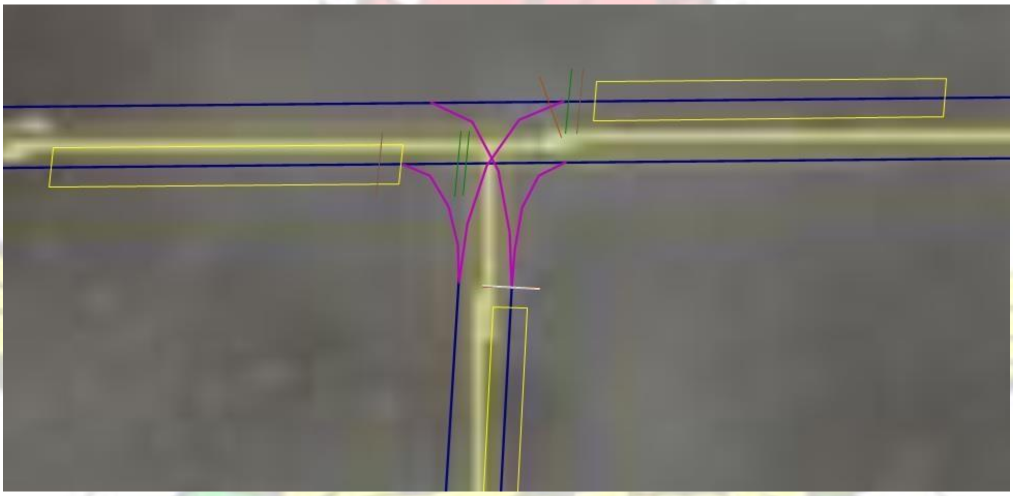

Figure 3.7. Wireframe model of existing intersection showing links (in blue) & connectors (in pink)

3.3.1 Priority Rule

The right of way at the intersection entry was modelled using the "priority rules" function in VISSIM rather than the conflict area function. The priority rules function was used because it allows for more control over input parameters such as minimum gap times, the minimum headways, and placement of where these interactions should take place thereby providing the flexibility needed to calibrate the model (Mai *et al.*, 2011). The priority rule means that priority is given to one movement over another and only the non-priority movement sees the priority rule and is required to stop. Therefore, in the case of a T- intersection, if a vehicle on the major road encounters a green line (conflict markers), the minor road vehicle will have to yield at the stop line (red line) to observe the priority rule. Figure 3.8 shows the positions of priority rules at the T-intersection and Table 3.4 details the minimum headways and gap times used for simulation respectively.

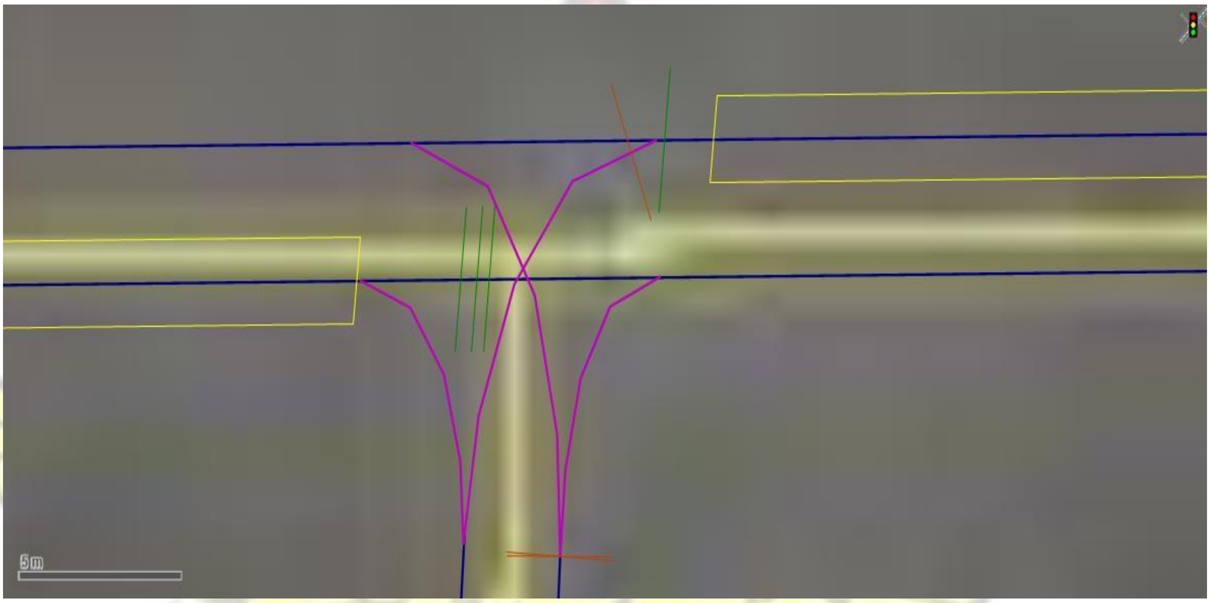

Figure 3.8. Priority rules at the case study intersection Table 3.4. Priority rules for simulation

Approach Movement	Minimum Headway	Minimum Gap
South Approach Left	3.5m	2.5s
South Approach Right	3.0m	3.0s
East Approach Left	3.0m	2.5s

3.3.2 Desired Speed Distribution

In order for VISSIM model to best reproduce field measured local traffic conditions, the model parameters for desired speed distribution and reduced speed area were adjusted until an acceptable match between predicted and filed measures of effectiveness were obtained. A desired speed distribution of 80km/h with a lower limit of 75km/h and an upper limit of 110km/h was used to model the west and east approaches (major road) whiles a desired speed distribution of 70km/h with a lower limit of 68km/h and an upper limit of 78km/h was used to model the south approach (minor road) for vehicular speeds as soon as vehicles enter the network.

Reduced speed areas of length 20m on the west and east approaches and 15m on the south approach were placed on each approach at the entrance to the junction. Vehicles within the reduced speed areas were assigned a desired speed of 40km/h with a lower bound of 40km/h and an upper bound of 45km/h.

3.4 VISSIM Run Consideration

Simulation runs are normally done for 3,600 seconds (1 hour) time period. However, the VISSIM version used for this study is the student's version. This version had a limitation of a maximum of 600 simulation seconds. The simulation runs were, therefore, performed for 600 seconds (10minutes). This includes a warm up period of 300 seconds (5 minutes). The warm up period is necessary to build the simulation to saturation flow (peak conditions) before any data is taken in order to mimic the peak condition in the field. The flow rates of the model was therefore taken for 5 minutes. In order to convert them to hourly flow rates, the flow rates per 5 minutes were multiplied by 12 to convert them to 60 minutes (1 hour) equivalent.

Due to the stochastic (random) nature of simulation models including VISSIM, a minimum of 10 simulation runs were performed with different random seed numbers to ensure that the values reported is a true statistical representation of the average (Mai *et al*., 2011). As a result, measured data like delay, maximum queue length, and minimum queue length and entry flow were recorded and averaged over 10 simulation runs. Fig 3.9 shows a snapshot of VISSIM simulation in 3D.

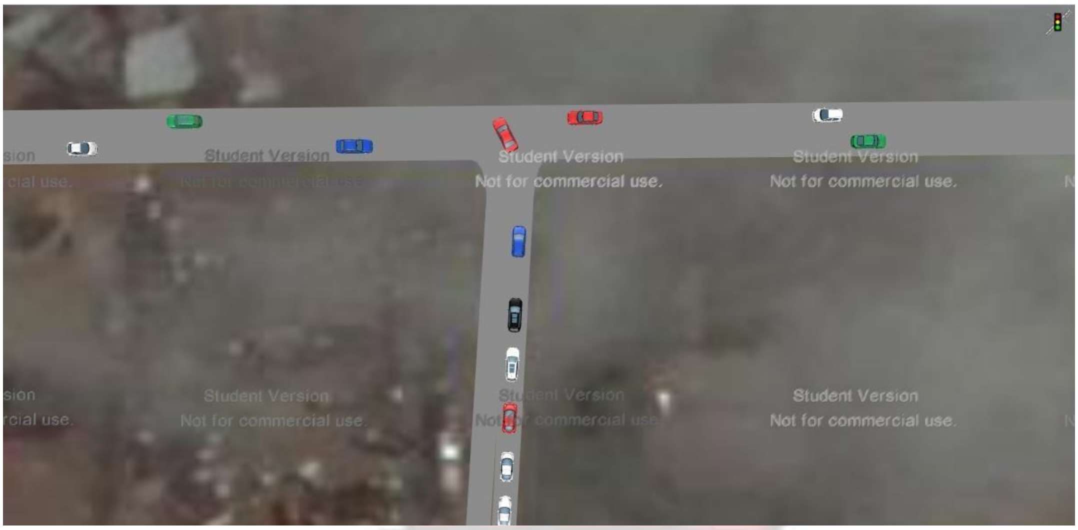

Figure 3.9. Snapshot of VISSIM Simulation in 3D

3.5 VISSIM Calibration

For calibration of VISSIM, calibrated guidelines as stated in FHWA (2007) were followed. The measures of effectiveness (MOEs) used in calibration were entry flow, average queue length and maximum queue length. Parameters within the model including minimum gap time, minimum headway and speed were adjusted such that the model's output (i.e. average of 10 runs) were compared against field measurements of entry flow on all approaches at the intersection, delay, average queue length and maximum queue length on the minor road.

3.5.1 Entry Flow Measurement

According to Mai *et al*. (2011), the first proof of calibration is how closely field volumes match simulation output volume. The measurement of simulation entry flow was made by using the "data collection point" tool in VISSIM. Data collection points are similar to counting boards that are attached to roadway tracks for recording of traffic volume

(Planung Transport Verkehr, 2014). Data collection points were used to count the simulated entry flow of vehicles at the intersection from all three approaches. After that, field and simulated flows were compared using the GEH test statistic to determine whether they were a good fit.

3.5.2 Delay and Queue Length Measurement

For delay measurements, a travel time section was created on the minor road which was used to measure the delay of every vehicle that passed between the start and end of the section. The travel time section was 205 metres long starting from the stop line of the minor road. The "floating car method" was used to measure the travel time within the same section length in the field during free flow and saturated conditions. The delay for each run for a total of four runs was calculated as the difference between the travel time measured during the saturated and free flow conditions. The average of the four runs was used for analysis. Figure 3.10 shows a travel time section used to measure delay on the minor road approach.

Figure 3.10. Travel time section used to measure delay

The maximum and average queue lengths were determined by using the queue counter tool in the VISSIM model. This tool was drawn on the minor road stop line to determine the maximum queue length and average queue length. VISSIM defines the maximum queue length as the maximum of the current queue length measured upstream every time step whiles the average queue length is computed as the arithmetic average of the current queue length measured upstream every time step (Planung Transport Verkehr, 2014). The field and simulated average queue lengths and maximum queue lengths were compared to determine whether the difference was statistically significant using single factor Analysis of Variance test.

3.6 Experimental Observations

After calibration, the VISSIM tool was used to perform several experimental observations. Various scenarios were investigated. The first scenario investigated the effect of minor road left-turning traffic on minor road delay. Other scenarios investigated include volume warrants necessitating a minor road left-turn lane and a major road left-turn lane based on delay.

A delay criterion was used rather than a probability or benefit-cost criteria. This was because delay is a more easily understandable measure of traffic performance than probability and benefits of safety improvements which is needed as an input in the benefit-cost criterion but cannot be easily quantified in Ghana due to unreliable accident data.

The simulation approach for this study followed that of Agent (1982). Agent’s approach differs from those of other studies (Ivan *et al.*, 2009; Kikuchi and Chakroborty, 1991) in that he considered delay to left-turning traffic in the development of volume warrants while they considered delay to advancing traffic caused by left-turning traffic. The warrants developed by these studies are applicable for only particular (fixed) percentages of left-turns of advancing volumes they investigated. For example, Kikuchi and Chakroborty (1991) developed warrants for 10%, 15% and 30% left turnings of advancing volume. This implies that the warrants cannot be used to determine the volume combinations at which an intersection with 40% left turnings of the advancing volume may warrant a left-turn lane.

3.6.1 Effect of left-turning traffic on minor road delay

In order to analyse the effect of left-turning traffic on minor road delay, the following experimental set up was constructed:

- 50/50 volume split per direction was assumed on the major road.
- Proportions of 25, 50 and 75 percent of left turning vehicles on the minor road approach were analysed.
- The major road volumes were varied between 1000 and 1600 pcu/h at increment of 200 pcu/h.

- The minor road volumes were varied between 200 and 800 pcu/h/ln at increments of 100 pcu/h/ln.
- A total of 48 operational scenarios were generated involving 480 simulation runs.

3.6.2 Threshold minor road left-turn volume installation warrant.

This test was performed to determine threshold minor road left-turn volume for a given total major road volume that will result in a critical delay and, therefore will warrant a minor road left-turn lane. Figure 3.11 graphically illustrates this experimental setup.

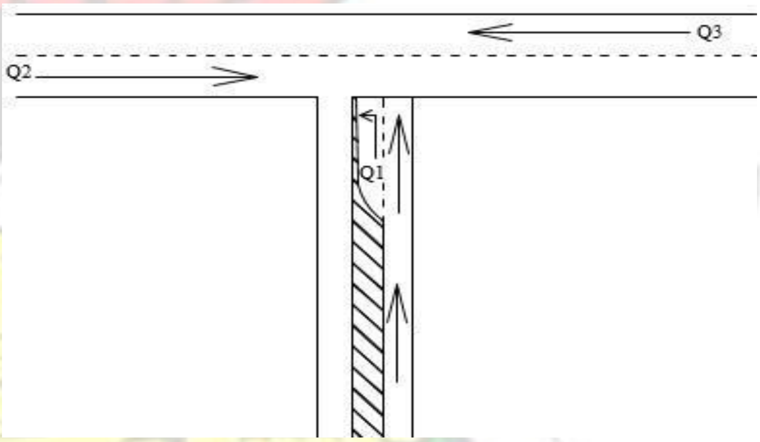

Figure 3.11. Volumes used in minor road left-turn lane warrant determination.

The experimental setup was as follows:

- In this setup, the delay measured was delay to minor road left-turning traffic. Minor road left-turn volumes (Q_1) were varied between 100 and 800 pcu/h/ln at increments of 100 pcu/h/ln.
- The total major road volume (Q_2 +Q_3) was varied between 1000 and 2000 pcu/h at increment of 200 pcu/h.

- A total of 48 operational scenarios were generated involving 480 simulation runs. Delay to minor road vehicles were plotted against minor road left turning volume and major road volume.

3.6.3 Major road left-turn lane installation volume warrant.

This test was performed to determine the threshold major road left-turn volume and opposing volume that will warrant a major road left-turn lane. For purposes of the test, a travel time section of 205 metres was created to measure delay to major road left-turning traffic. Figure 3.12 graphically illustrates this experimental setup.

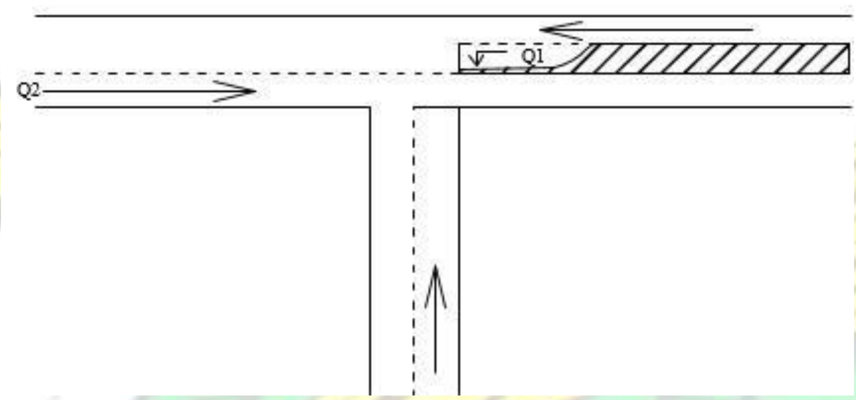

Figure 3.12. Volumes used in major road left-turn lane warrant determination.

The experimental setup was as follows:

- Major road left-turn volumes (Q_1) were held constant whiles the opposing volumes (Q_2) were varied.
- Hundred percent of the volume on the major road (Q_1) turned left whiles 100 percent of the opposing volume (Q_2) went straight through the intersection.

- Major road left-turn volumes were varied from 100 to 500 pcu/h/ln at increments of 100 pcu/h/ln.
- Opposing volumes were varied between 600 and 1600 pcu/h/ln at increments of 200 pcu/h/ln.
- A total of 30 operational scenarios were generated involving 300 simulation runs. A graph of major road left turn delay against opposing volumes was plotted.

CHAPTER 4: RESULTS AND DISCUSSIONS

4.1 Calibration Results

The study intersection was calibrated using entry flow and three measures of effectiveness namely delay, maximum queue length and average queue length. The minor road was the target approach chosen for calibration. The minor road was chosen as the target approach because of persistent delay and queue during morning peak periods.

4.1.1 Calibration using Delay

After four runs of the "floating car" to obtain the average field delay, the average delay experienced by minor road traffic was found to be 56 s/veh. After VISSIM calibration, the field and simulated delay results were compared. Table 4.1 compares the simulated and field delay results.

Table 4.1. Comparison of Field and Simulated Delay

No. of Simulations Runs	Simulation (s/veh)	Field (s/veh)
Run 1	102	59
Run 2	5	60
Run 3	20	55
Run 4	28	50
Run 5	12	-
Run 6	34	-
Run 7	20	-
Run 8	50	-
Run 9	78	-
Run 10	47	-
Average	40	56

The simulated delay were compared with field delay using single factor Analysis of Variance (ANOVA) test to determine whether the difference was statistically significant. The statistical analysis of the data was performed at 95% confidence level. The following null and alternate hypothesis were used.

H_o: There is no significant difference between the simulated and field delay

H_1: There is a significant difference between the simulated and field delay If

$F < F_{critical}$ and $P > 0.05$, we accept the null hypothesis.

The result of the ANOVA test is presented in Table 4.2 below.

Table 4.2. ANOVA test results

Summary

Groups	*Count*	*Sum*	*Average*	*Variance*
Simulated Delay	10	395.3382	39.53382	936.3672767
Field Delay	4	224	56	20.66666667

ANOVA

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	774.6713	1	774.6713	1.0950314	0.31598	4.747225347
Within Groups	8489.305	12	707.4421			
Total	9263.977	13				

Since F (1.095) < $F_{critical}$ (4.747) and the P-value (0.316) > 0.05, we accept the null hypothesis and conclude that there was no significant difference between the simulated and field delays.

4.1.2 Calibration using Entry flow

Results of the simulated and field entry flows during the peak morning hour on the three approaches to the intersection are presented in Table 4.3 below.

Table 4.3. Simulated and field entry flows

Approach	Simulation	Field
	Entry flow (pcu/hr)	Entry flow (pcu/hr)
West	590	635
East	624	659
South	517	583

Using Equation (2.7), a GEH test statistic was conducted to compare the simulated and field entry flow. An interpretation of the GEH test result is as follows: A GEH value < 5 is an indication that the simulated and field flows can be considered a good fit; GEH values between 5 and 10 indicates that the simulated and field flows may require future investigation and a GEH value > 10 is an indication that the simulated and field flows cannot be considered a good fit.

A GEH test value of 5 or lower was obtained for all the three approaches. This is an indication that the simulated and field flows can be considered an acceptable fit. Table 4.4 shows the GEH results for all the intersection approaches.

Table 4.4. GEH statistics results

Approach	GEH Test Result
West	1.8
East	1.4
South	2.8

4.1.3 Calibration using Maximum and Average Queue Length

Field average and maximum queue lengths were also compared with simulated average and maximum queue lengths using single factor Analysis of Variance (ANOVA) test to determine whether the difference was statistically significant.

Figure 4.1 shows the field and simulated maximum and average queue lengths.

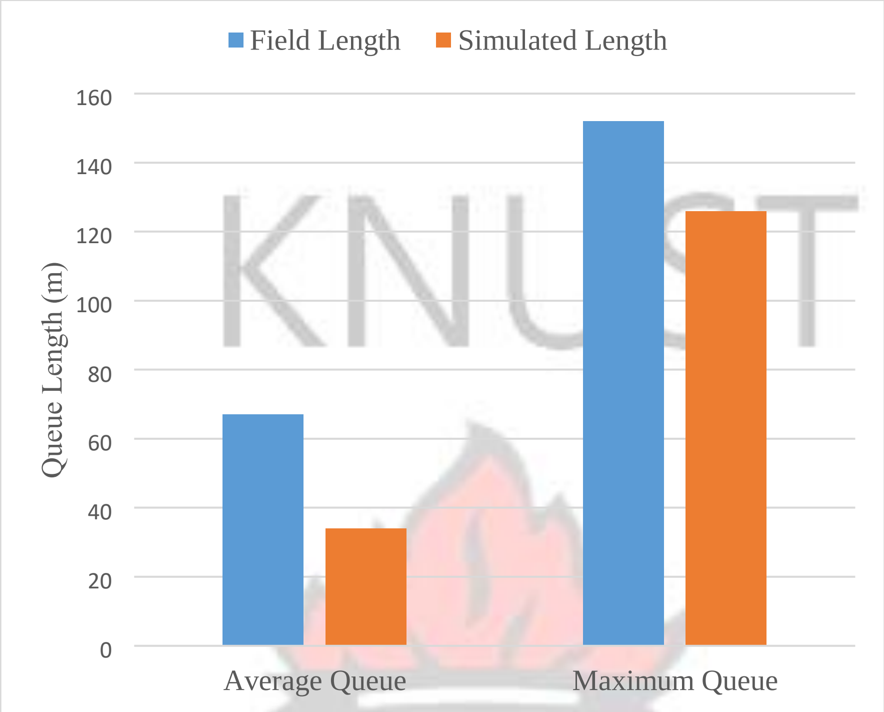

Figure 4.1. Field and Simulated Queue lengths

A single factor ANOVA performed on the queue length results showed that the difference between simulated and field values of the average and maximum lengths of queue was not statistically significant because F (17.75) < $F_{critical}$ (18.512) and P (0.0519) > 0.05. The result is shown in Table 4.5.

Table 4.5. Queue length ANOVA results

Summary				
Groups	*Count*	*Sum*	*Average*	*Variance*
Average	2	101	50.5	544.5
Maximum	2	278	139	338

ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	7832.25	1	7832.25	17.75014	0.051984	18.51282
Within Groups	882.5	2	441.25			
Total	8714.75	3				

Long queues on the minor road is due to the fact that vehicles on the minor road tend to wait for sufficient gaps in the major road traffic before joining the major road. At higher traffic volumes, vehicles on the minor road tend to wait for a longer duration.

The longer the waiting time, the longer the queue length on the minor road.

4.2 Effect of Minor road left-turning traffic volume on minor road delay

In trying to analyse the effect of different left-turning traffic volume on minor road delay, graphs of average delay on minor road for different minor road left turning percentages and major road volumes are presented in Figures 4.2 through to 4.5.

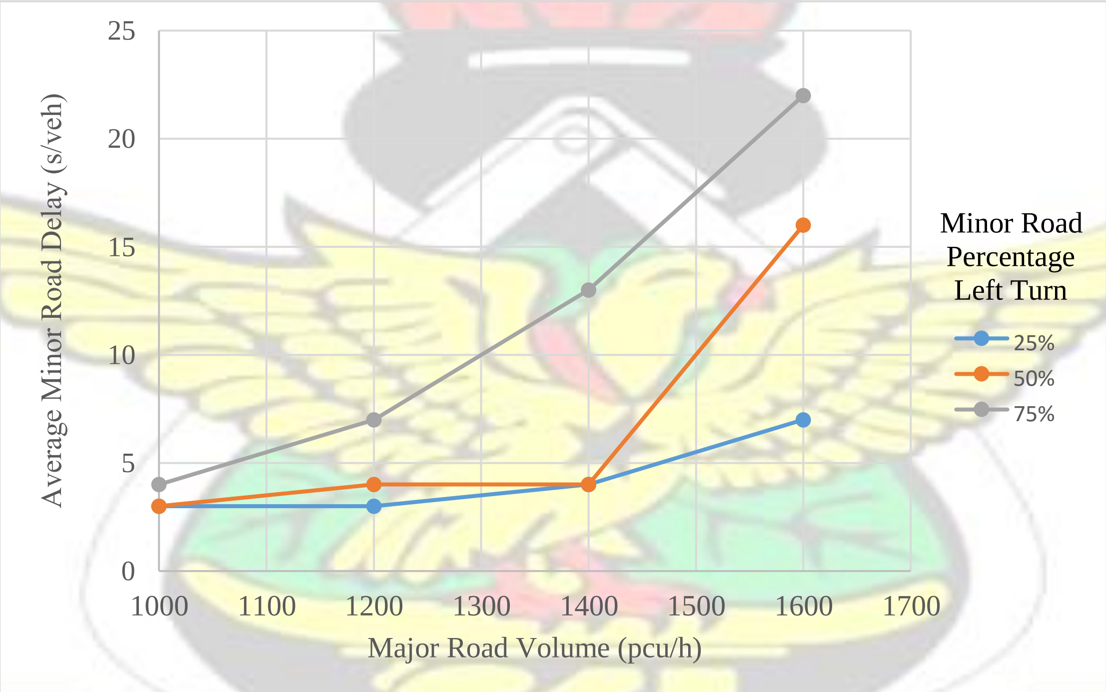

Figure 4.2. Delay comparison for a minor road volume of 200 pcu/h/ln at different turning percentages.

From Figure 4.2, minor road volumes of up to 200 pcu/h/ln and major road volumes of up to 1400 pcu/h, experienced no significant change in delay on the minor road with the maximum delay being 4 s/veh for minor road left-turns of up to 50%.

There was a sharp increase in delay for 50% minor road left-turns as compared to 25% left-turns when the major road volume exceeded 1400 pcu/h. Generally, minor road left-turns of 75% experience a higher delay than left-turns of 25% and 50% but delay to minor road left-turns of 75% was significantly higher with major road volumes of more than 1200 pcu/h.

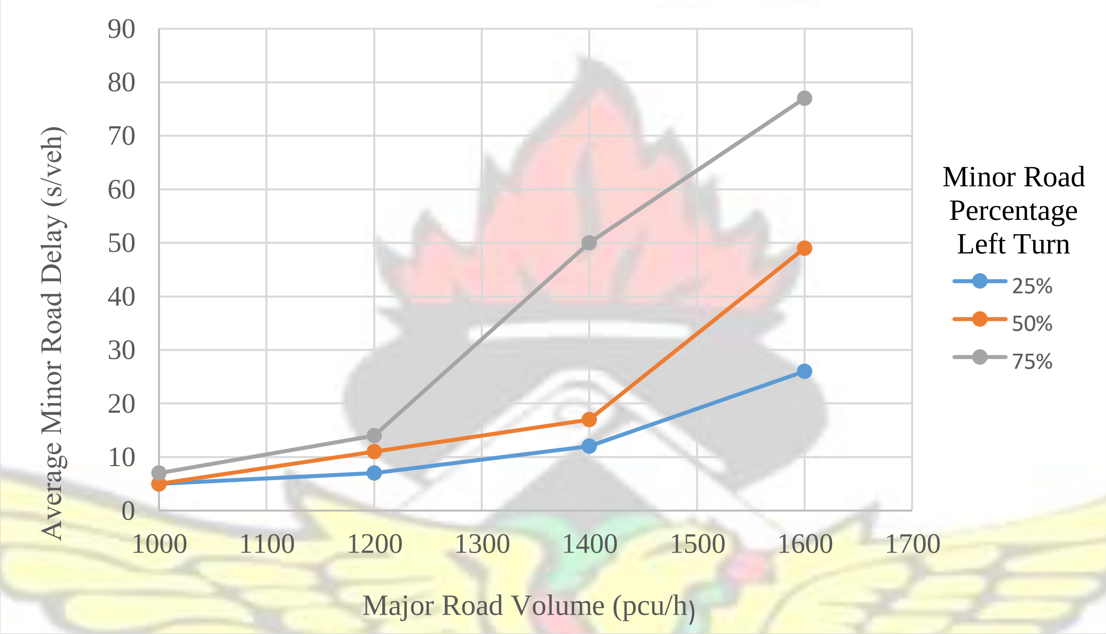

Figure 4.3. Delay comparison for a minor road volume of 400 pcu/h/ln at different turning percentages.

From Figure 4.3 above, at a major road volume of 1000 pcu/h, different left turning percentages for a minor road volume of 400 pcu/h/ln resulted in small changes in minor road delay. Beyond a major road volume of 1000 pcu/h/ln, an increase in minor road left-turns led to an increase in delay on the minor road. The significant increase in delay for 75% minor road left-turns occurred beyond a major road volume of 1200 pcu/h. For minor road left-turns of 25% and 50%, the significance increase in delay occurred when the major road volume was more than 1400 pcu/h.

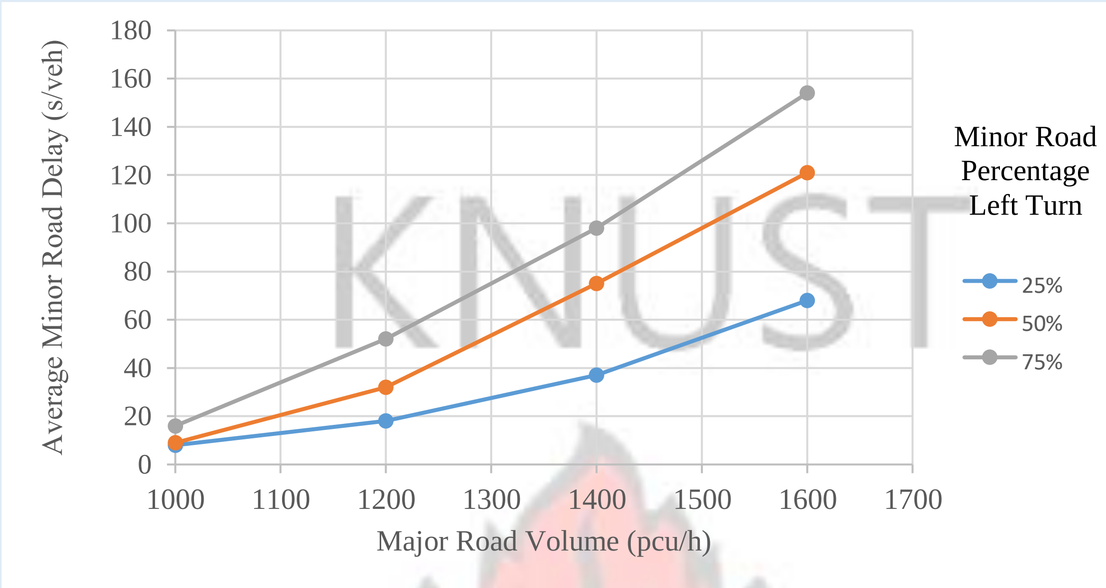

Figure 4.4. Delay comparison for a minor road volume of 600 pcu/h/ln at different turning percentages

From Figure 4.4, there was a general rise in delay with a corresponding increase in major road volume and minor road left-turns. There was a steady rise in delay for 50% and 75% minor road left-turns beyond a major road volume of 1200 pcu/h.

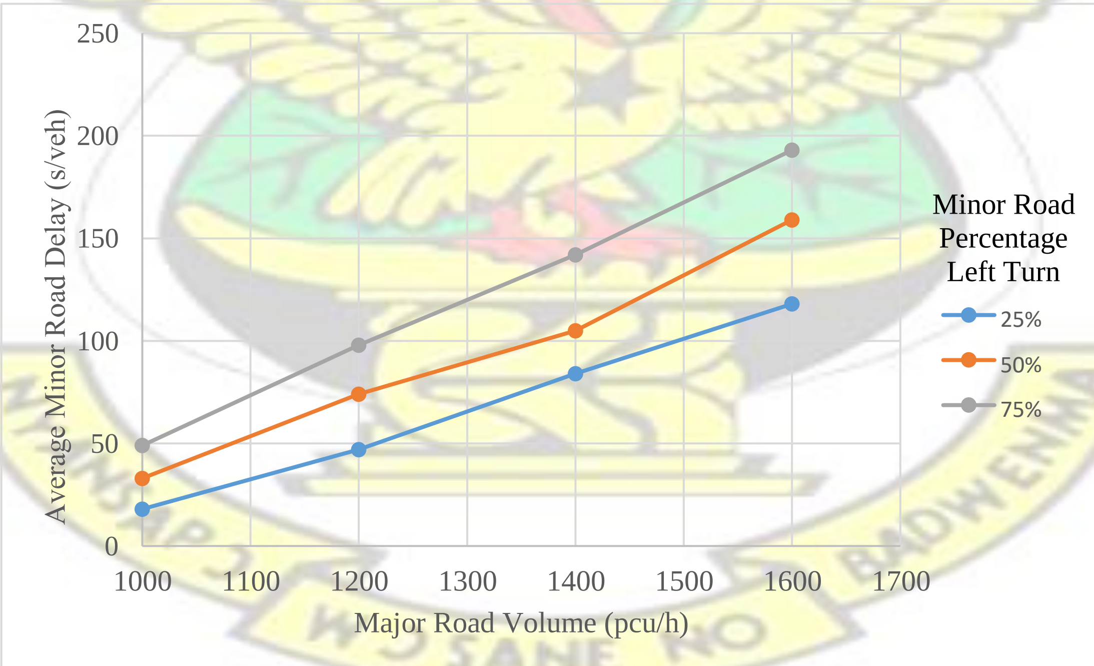

Figure 4.5. Delay comparison for a minor road volume of 800 pcu/h/ln at different turning percentages.

From Figure 4.5, the increase in delay with a corresponding increase in major road volume for the three different left-turning percentages on the minor road investigated generally appeared to be uniform when the minor road volume is 800 pcu/h/ln.

The results from Figures 4.2 through to 4.5 indicated that the proportion of left-turn manoeuvres on the minor road in addition to the volume of traffic on the major road exert significant impact on minor road delay. This agrees with a study by Cvitanic *et al.* (2004). The increase in minor road delay as left turning percentage increased can be attributed to the fact that more minor road traffic will be looking for gaps in the major road traffic stream. This leads to long queues and higher delay on the minor road.

4.3 Threshold minor road left-turn volume installation warrant.

In order to establish a relationship between minor road left-turn volume and major road volume, a graph of average delay to minor road left-turners, major road volume and minor road left-turn volume was plotted (see Figure 4.6).

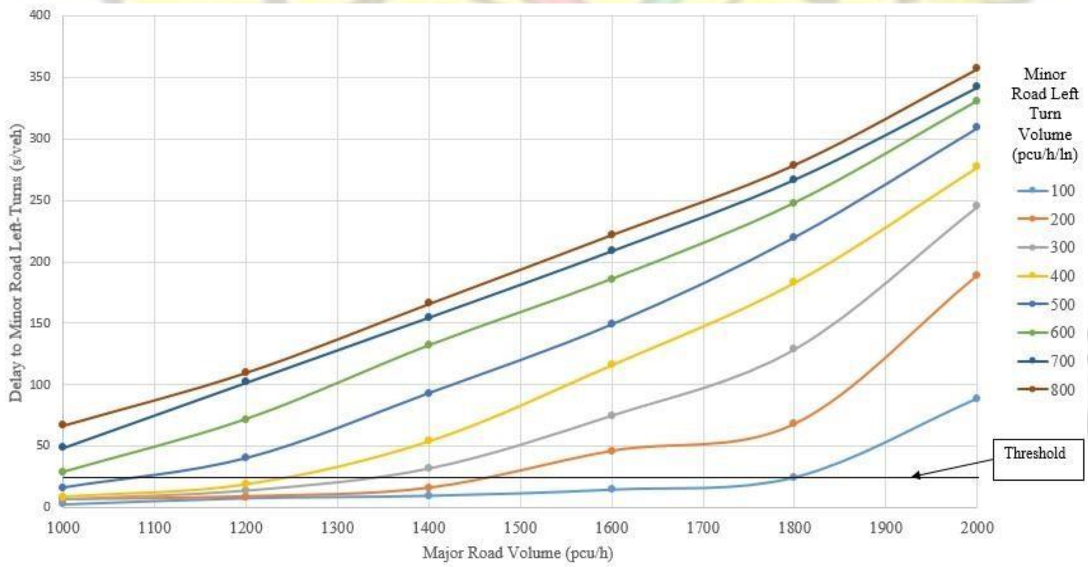

Figure 4.6. Relationship of minor road delay to major road volume and minor road left turns.

It can be seen from Figure 4.6 that the higher the minor road left-turn volume, the higher the average delay experienced by minor road left-turners. Also, the difference in delay on the minor road becomes significantly higher with higher major road volumes. This trend in results is in line with a study done by Zhu *et al.* (2010).

For major road volumes of up to 1200 pcu/h and minor road left turning volumes of up to 400 pcu/h/ln, the variation in delay corresponding to a change in minor road left-turn volume was relatively small. There was a sharp rise in delay for minor road left-turns from 500 pcu/h/ln with major road volumes under 1200 pcu/h.

The change in delay on the minor road from major road volumes of 1800 pcu/h to 2000 pcu/h was significantly very high (an average change in delay of as much as 90 s/veh). A major road volume of 2000 pcu/h indicate that very few gaps in the major road traffic stream will be available to minor road left-turners, thus causing increased delay to them.

For the development of a model equation between minor road left-turn volume and major road volume, a delay criterion of 25 s/veh, representing the cut off point for LOS C (HCM, 2000), was chosen. This cut off point was chosen because previous works seem to agree that a LOS C on the minor road is considered acceptable by engineers and that delays beyond this LOS may warrant an intersection improvement (Ivan et al., 2009; Henry *et al.,* 1982).

The points at which delay became critical were taken from the graph shown in Figure 4.6 and plotted as a line of best-fit. The relationship found is shown in Figure 4.7.

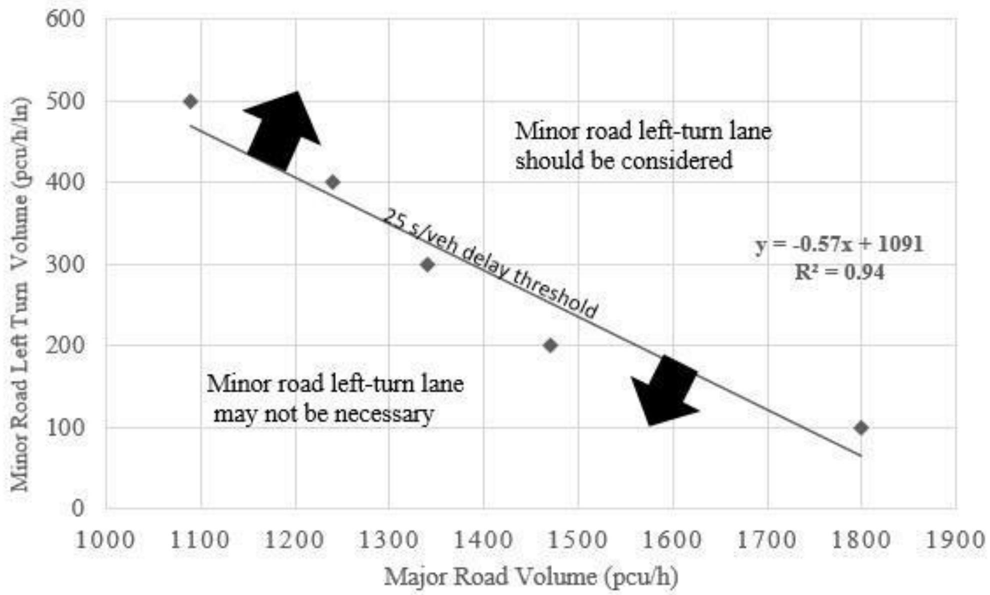

Figure 4.7. Minor road left turns when delay becomes critical

The resulting equation of the line is given as:

y = -0.57x +1091 (4.1)

Where:

y = minor road left turn volume in pcu/h/ln *x*

= major road volume in pcu/h

The model in Figure 4.7 is a good fit with an R^2 value of 0.94. Given a major road volume (both directions), the minor road left turn volume necessary to create the critical delay could be estimated. It should be noted that, the model is valid for major road volumes between 1090 pcu/h and 1800 pcu/h and $x \neq 0$.

Using Eq. (4.1), it can be predicted that, for a major road volume of say 1450 pcu/h, the minor road left turn volume necessary to achieve the critical delay is 265 pcu/h/ln. This implies that, given a total major road volume of 1450 pcu/h, minor road left turn volumes above 265 pcu/h/ln will experience delays of more than 25 s/veh whiles minor road left turn volumes below 265 pcu/h/ln will experience delays of less than 25 s/veh.

Five sets of major road volumes *(column a)* and their corresponding minor road leftturn volume thresholds *(column b)* were used to compare the delay expected from the regression model *(column c)* and that from the simulation *(column d)*. The simulated delay results of the same set of major road volumes *(column a)* and minor road leftturn volumes above the model's thresholds *(column e)* and minor road left-turn volumes below the model's thresholds *(column g)* were also compared with the delay expected from the regression model. The simulated delay results for minor road leftturn volumes above the model's thresholds *(column f)* were expected to be more than 25 s/veh whiles the simulated delay results for minor road left-turn volumes below the model's thresholds *(column h)* were expected to be less than 25 s/veh (see Table 4.6).

The model performed well for the range of input values.

Table 4.6. Comparison of expected and actual simulated delay

Major Road Volume (pcu/h) *(a)*	Minor Road left-turn threshold volume (pcu/h/ln) *(b)*	Expected simulated delay of VISSIM *(a+b)* (s/veh) *(c)*	Actual simulated delay of VISSIM *(a+b)* (s/veh) *(d)*	Minor Road left-turn volume above model threshold (pcu/h/ln) *(e)*	Simulated delay results of *(a+e)* (s/veh) *(f)*	Minor road left-turn volume below model threshold (pcu/h/ln) *(g)*	Simulated delay results of *(a+g)* (s/veh) *(h)*
1450	265	25	29	300	44	125	17
1350	322	25	26	350	27	295	23
1250	379	25	23	410	26	290	15
1150	436	25	22	550	42	420	16
1100	464	25	19	530	31	450	17

4.4 Major road left-turn lane installation volume warrant.

In trying to develop a volume warrant for an exclusive major road left-turn lane at a T-intersection, a graph of delay to major road left-turn vehicles and opposing volume was plotted and the relationship between them is as shown in Figure 4.8.

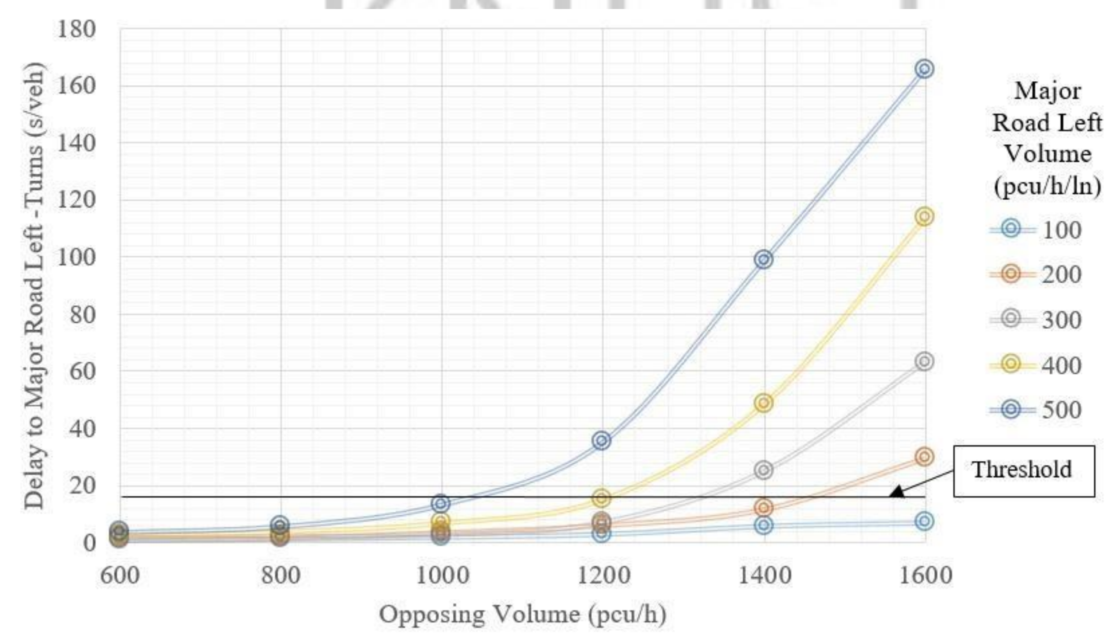

Figure. 4.8. Relationship of major road left turn delay to opposing volume

In setting up the critical delay above which a major road left-turn lane is to be considered, the threshold (*black line*) was set at the point at which the curve for delay rose sharply for relatively high opposing volumes (Agent, 1982; Ivan *et al.*, 2009). The critical delay selected for a two-lane T-intersection for developing the warrant was 16 s/veh. This value is within the magnitude of the 20 s/veh that Agent (1982) found and the 14 s/veh that Kikuchi and Chakroborty (1991) proposed in their study.

Using a critical delay threshold of 16 s/veh, the combinations of opposing volume and major road left-turn volume warranting a major road left-turn lane are shown in Table 4.7.

Table 4.7. Volume warrants for major road left-turn lane at a T-intersection

Opposing Volume (pcu/h)	Major road left-turn volume (pcu/h/ln)
500	1040
400	1200
300	1320
200	1460

The results of this study are corroborated by the Kikuchi and Chakraborty (1991) study, in which the researchers reported that the warrants based on delay as a warrant criterion tend to yield higher volume thresholds compared to those based on the probability of vehicles stopping behind the left-turning vehicles.

For purposes of comparison, the volume warrants developed by Agent was converted into passenger car units using the vehicle composition obtained from field volume studies and PCE values developed by Adams & Obiri-Yeboah (2008). The converted volume warrants were then compared with volume warrants obtained from this study. This is shown in Figure 4.9.

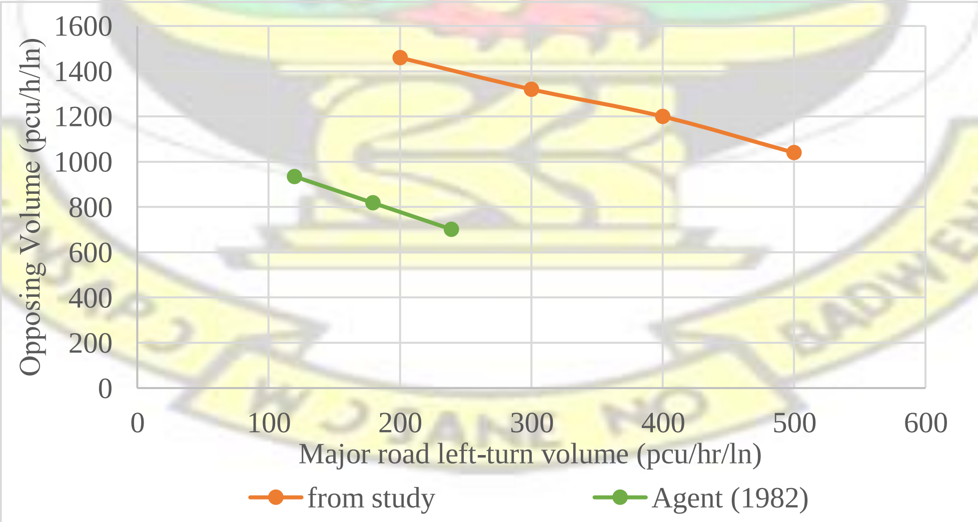

Figure 4.9. Comparison of warrants from this study & Agent (1982).

As observed from Figure 4.9, the warrants developed from this study require higher volumes than those determined by Agent (1982). This may be so because the critical gap of today's drivers is likely to be smaller than that of the 1980s when Agent developed his warrants. This is primarily due to improvements in vehicle performance. This probably shows the need to develop warrants that reflect current traffic flow conditions. Also, Agent's simulation model may have been limited when compared to current advanced microscopic simulation models.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

In this study, the researcher sought to calibrate a VISSIM model of the case study Tintersection, analyse the effect of increasing minor road left-turning traffic volume on minor road delay, develop a model equation that forecasts threshold minor road leftturn volume warrants for minor road left-turn lane and develop volume warrants for exclusive major road left-turn lane at a T-intersection. Based on the results of the study, the following conclusions have been made:

- The VISSIM intersection model of the study T-intersection has been successfully calibrated to reflect field flow conditions indicating that there was no significant difference between field and simulated results.

- For minor road volumes of up to 200 pcu/h/ln and major road volumes of up to 1400 pcu/h, there was no significant change in delay on the minor road for minor road left-turns of up to 50%.
- An equation of the form $y = -0.57x + 1091$ has been developed. This equation forecasts the threshold minor road left-turn volumes above which a minor road left turn lane may be considered and below which a minor road left turn lane may not be necessary.
- Volume warrants for a major road left-turn lane has been developed.

5.2 Recommendations

- It is recommended that the developed model and warrant be further tested for many more T-intersections before it may be applied.
- T-intersection volumes meeting threshold values found in this study is an indication that further detailed study of the intersection is required.
- The approach used in the calibration and simulation can be adopted by metropolitan road engineers to assess the need for a left-turn lane.
- Future intersection improvements should consider the impact of left-turning manoeuvres.

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APPENDICES

APPENDIX A: INTERSECTION TURNING MOVEMENT COUNT (PCU)

A.1. South Approach

Intersection: Dr. Osei Tuffour Bypass/Osei Nkwantabisa Avenue (Dakwadwom)

MORNING: Peak Period Day: Friday Date: 01-04-2016 Weather: Sunny

SOUTH APPROACH (FROM) ADIEMBRA

Time Period	CARS/TAXIS/PICKUPS/4X4			SMALL & MEDIUM BUS			LARGE BUS/TRUCKS/OTHERS			TOTAL TRAFFIC		
	L	T	R	L	T	R	L	T	R	L	T	R
8:00AM-8:15AM	55	0	55	5	0	9	10	0	3	70	0	66
8:15AM-8:30AM	51	0	74	9	0	10	0	0	3	60	0	87
8:30AM-8:45AM	63	0	66	5	0	7	8	0	5	76	0	78
8:45AM-9:00AM	62	0	57	3	0	9	3	0	5	68	0	71
9:00AM-9:15AM	57	0	60	2	0	3	8	0	3	66	0	66
9:15AM-9:30AM	49	0	57	5	0	9	3	0	5	57	0	71
9:30AM-9:45AM	49	0	48	7	0	5	28	0	3	83	0	56
9:45AM-10:00AM	59	0	54	2	0	0	15	0	5	76	0	59

Movement Code: L = Left turn, T = Through/Straight ahead, R = Right turn.

A.2. East Approach

Intersection: Dr. Osei Tuffour Bypass/Osei Nkwantabisa Avenue (Dakwadwom)

MORNING: Peak Period Day: Friday Date: 01-04-2016 Weather: Sunny

EAST APPROACH (FROM) AHODWO ROUNDABOUT

Time Period	CARS/TAXIS/PICKUPS/4X4			SMALL & MEDIUM BUS			LARGE BUS/TRUCKS/OTHERS			TOTAL TRAFFIC		
	L	T	R	L	T	R	L	T	R	L	T	R
8:00AM-8:15AM	33	67	0	12	20	0	0	5	0	45	92	0
8:15AM-8:30AM	32	70	0	2	29	0	3	23	0	36	121	0
8:30AM-8:45AM	37	78	0	2	27	0	0	23	0	39	128	0
8:45AM-9:00AM	24	60	0	2	20	0	0	23	0	26	103	0
9:00AM-9:15AM	32	80	0	7	36	0	3	18	0	41	133	0
9:15AM-9:30AM	27	67	0	5	14	0	8	25	0	40	106	0
9:30AM-9:45AM	36	77	0	3	31	0	8	23	0	47	130	0
9:45AM-10:00AM	39	83	0	3	22	0	3	13	0	45	118	0

Movement Code: L = Left turn, T = Through/Straight ahead, R = Right turn.

A.3. West Approach

Intersection: Dr. Osei Tuffour Bypass/Osei Nkwantabisa Avenue (Dakwadwom)

MORNING: Peak Period Day: Friday Date: 01-04-2016 Weather: Sunny

WEST APPROACH (FROM) SANTASI ROUNDABOUT

Time Period	CARS/TAXIS/PICKUPS/4X4			SMALL & MEDIUM BUS			LARGE BUS/TRUCKS/OTHERS			TOTAL TRAFFIC		
	L	T	R	L	T	R	L	T	R	L	T	R
8:00AM-8:15AM	0	76	47	0	9	7	0	20	0	0	105	54
8:15AM-8:30AM	0	72	42	0	9	2	0	15	0	0	96	44
8:30AM-8:45AM	0	75	53	0	5	7	0	8	5	0	88	65
8:45AM-9:00AM	0	81	45	0	7	5	0	23	8	0	110	58
9:00AM-9:15AM	0	51	45	0	5	5	0	23	8	0	79	58
9:15AM-9:30AM	0	75	34	0	9	5	0	18	5	0	101	44
9:30AM-9:45AM	0	78	55	0	14	7	0	15	10	0	107	72
9:45AM-10:00AM	0	89	38	0	15	3	0	20	10	0	124	51

Movement Code: L = Left turn, T = Through/Straight ahead, R = Right turn.

APPENDIX B: INTERSECTION HOURLY VOLUME COUNT (PCU/H)

B.1. South Approach

Intersection: Dr. Osei Tuffour Bypass/Osei Nkwantabisa Avenue (Dakwadwom)

MORNING Peak Period Day: Friday Date: 01-04-2016 Weather: Sunny

SOUTH APPROACH (FROM) ADIEMBRA

Time Period	CARS/TAXIS/PICKUPS/4X4			SMALL & MEDIUM BUS			LARGE BUS/TRUCKS/OTHERS			TOTAL TRAFFIC			ALL	NO. OF VEHICLES IN QUEUE	DEMAND
	L	T	R	L	T	R	L	T	R	L	T	R			
8:00AM-9:00AM	231	0	252	22	0	34	20	0	15	273	0	301	574	79	653
8:15AM-9:15AM	233	0	257	19	0	29	18	0	15	269	0	301	570	75	645
8:30AM-9:30AM	231	0	240	15	0	27	20	0	18	266	0	285	551	49	600
8:45AM-9:45AM	217	0	222	17	0	26	40	0	15	274	0	263	537	49	586
9:00AM-10:00AM	214	0	219	15	0	17	53	0	15	282	0	251	533	50	583
Total	**1126**	**0**	**1190**	**88**	**0**	**133**	**150**	**0**	**78**	**1364**	**0**	**1400**	2765	302	3,067

B.2. East Approach

Intersection: Dr. Osei Tuffour Bypass/Osei Nkwantabisa Avenue (Dakwadwom)

MORNING Peak Period Day: Friday Date: 01-04-2016
Weather: Sunny

Movement Code: L = Left turn, T = Through/Straight ahead, R = Right turn.

EAST APPROACH (FROM) AHODWO ROUNDABOUT

Time Period	CARS/TAXIS/PICKUPS/4X4			SMALL & MEDIUM BUS			LARGE BUS/TRUCKS/OTHERS			TOTAL TRAFFIC			ALL
	L	T	R	L	T	R	L	T	R	L	T	R	
8:00AM-9:00AM	126	275	0	17	97	0	3	73	0	146	444	0	590
8:15AM-9:15AM	125	288	0	12	112	0	5	85	0	142	485	0	627
8:30AM-9:30AM	120	285	0	15	97	0	10	88	0	145	469	0	615
8:45AM-9:45AM	119	284	0	17	100	0	18	88	0	154	472	0	625
9:00AM-10:00AM	134	307	0	19	102	0	20	78	0	173	487	0	659
Total	624	1439	0	80	508	0	55	410	0	759	2357	0	3116

B.3. West Approach

Intersection: Dr. Osei Tuffour Bypass/Osei Nkwantabisa Avenue (Dakwadwom)

MORNING Peak Period Day: Friday Date: 01-04-2016
Weather: Sunny

WEST APPROACH (FROM) SANTASI ROUNDABOUT

Time Period	CARS/TAXIS/PICKUPS/4X4			SMALL & MEDIUM BUS			LARGE BUS/TRUCKS/OTHERS			TOTAL TRAFFIC			ALL
	L	T	R	L	T	R	L	T	R	L	T	R	

8:00AM-9:00AM	0	304	187	0	29	20	0	65	13	0	398	220	618
8:15AM-9:15AM	0	279	185	0	26	19	0	68	20	0	372	224	596
8:30AM-9:30AM	0	282	177	0	26	22	0	70	25	0	378	224	602
8:45AM-9:45AM	0	285	179	0	34	22	0	78	30	0	397	231	628
9:00AM-10:00AM	0	293	172	0	43	20	0	75	33	0	411	225	635
Total	0	1443	900	0	156	104	0	355	120	0	1954	1124	3078

Movement Code: L = Left turn, T = Through/Straight ahead, R = Right turn.

*shaded rows represent morning peak hour period

APPENDIX C: QUEUE LENGTH STUDIES

C.1 Field Queue Length Data (Minor Road)

Intersection Queue Length Studies (Minor Road)

Intersection: Dr. Osei Tuffour Bypass/Osei Nkwantabisa Avenue (Dakwadwom)

Morning Peak Period **Day: Friday** **Date: 01-04-2016** **Weather: Sunny**

Time	Queue Length (m)	Time	Queue Length (m)
08:01	133	08:31	190
08:02	122	08:32	182
08:03	154	08:33	184
08:04	166	08:34	170
08:05	210	08:35	124
08:06	190	08:36	151
08:07	231	08:37	128
08:08	190	08:38	152
08:09	149	08:39	120
08:10	132	08:40	151
08:11	120	08:41	133
08:12	147	08:42	132
08:13	153	08:43	128
08:14	153	08:44	64

08:15	162	08:45	55
08:16	171	08:46	100
08:17	166	08:47	111
08:18	175	08:48	60
08:19	165	08:49	120
08:20	170	08:50	145
08:21	175	08:51	160
08:22	134	08:52	130
08:23	128	08:53	127
08:24	204	08:54	90
08:25	171	08:55	107
08:26	208	08:56	90
08:27	184	08:57	90
08:28	158	08:58	100
08:29	184	08:59	106
08:30	160	09:00	94

Time	Queue Length (m)	Time	Queue Length (m)
09:01	47	09:31	5
09:02	62	09:32	5
09:03	42	09:33	0
09:04	42	09:34	23
09:05	25	09:35	21

09:06	8	09:36	24
09:07	30	09:37	30
09:08	20	09:38	5
09:09	44	09:39	11
09:10	120	09:40	20
09:11	138	09:41	45
09:12	152	09:42	24
09:13	137	09:43	42
09:14	128	09:44	73
09:15	142	09:45	54
09:16	104	09:46	21
09:17	112	09:47	82
09:18	119	09:48	30
09:19	90	09:49	13
09:20	116	09:50	62
09:21	114	09:51	28
09:22	96	09:52	39
09:23	88	09:53	82
09:24	109	09:54	72
09:25	112	09:55	132
09:26	141	09:56	132
09:27	98	09:57	134
09:28	80	09:58	106
09:29	0	09:59	115
09:30	5	10:00	101

C.2 Simulated Queue Length Results Used In Calibration (Minor Road)

QUEUE COUNTER EVALUATION: SIMRUN	TIME INT	QUEUE COUNTER	QLENGTH	QLENGTH MAX	QSTOPS
334	0-3600	1	87.62	164.65	95
335	0-3600	1	2.88	29.44	35
336	0-3600	1	43.74	164.59	64
337	0-3600	1	21.92	91.84	86
338	0-3600	1	9.42	67.03	72
339	0-3600	1	30.14	164.59	109
340	0-3600	1	13.84	102.52	60
341	0-3600	1	38	164.56	95
342	0-3600	1	57	164.18	81
343	0-3600	1	30.7	144.16	105
AVG	0-3600	1	33.53	125.76	80
STDDEV	0-3600	1	25.09	49.74	23
MIN	0-3600	1	2.88	29.44	35
MAX	0-3600	1	87.62	164.65	109