Synchronization of Signalised Intersections: A Case Study of Three Major Intersections on the 24th February Road, Kumasi, Ghana

E. K. Nyantakyi[1], C. A. Adams[2], J. K. Borkloe[3] and D. Pobee[4]
[1]Department of Civil Engineering, Kumasi Polytechnic, P.O. Box 854, Kumasi, Ghana
[2]Department of Civil Engineering, KNUST, Kumasi, Ghana
[3]Department of Civil Engineering, Kumasi Polytechnic, P.O. Box 854, Kumasi, Ghana
[4]Department of Civil Engineering, Kumasi Polytechnic, P.O. Box 854, Kumasi, Ghana

ABSTRACT

Signalized intersections are critical elements of an urban road transportation system and maintaining these control systems at their optimal performance for different demand conditions has been the primary concern of the traffic engineers. Traffic simulation models have been widely used in both transportation operations and traffic analyses because simulation is safer, less expensive, and faster than field implementation and testing. This study evaluated performance of selected signalized intersections using micro simulation models in Synchro/SimTraffic. Traffic, geometric and signal control data including key parameters with the greatest impact on the calibration process were collected on the field. At 5% significant level, the Chi square test and t-test analyses revealed that headway had a strong correlation with saturation flow compared to speed for both field and simulated conditions. It was concluded that changes in phasing plan with geometric improvement would improve upon the selected signalized intersection's level of service. An interchange therefore should be constructed at Anloga intersection to allow free movement of vehicles thereby minimizing congestion and accident occurrences. Stadium and Amakom signalized intersections should be coordinated to allow as many vehicles as possible to traverse those intersections without any delay.

***Key words*:** Performance measures, Sensitivity Analysis, Signalized intersections, Synchro/ Sim Traffic, S

I. INTRODUCTION

In general, "simulation is defined as dynamic representation of some part of the real world achieved by building a computer model and moving it through time" [1]. Computer models are widely used in traffic and transportation system analysis. The use of computer simulation started when D.L. Gerlough published his dissertation: "Simulation of freeway traffic on a general-purpose discrete variable computer" at the University of California, Los Angeles, in 1955 [2]. From those times, computer simulation has become a widely used tool in transportation engineering with a variety of applications from scientific research to planning, training and demonstration.

The 24th February Road is an East-West principal arterial of about 5.4 km length from KNUST junction to the UTC traffic light. The road is a 2-lane dual carriageway, and paved over its entire length. The road provides the main route that leads into the Kumasi metropolis from the southern parts of Ghana. The Anloga, Stadium and Amakom signalized intersections are three major intersections on one of the major arterials of Kumasi. These selected signalized intersection approaches are traversed by the same main arterial road entering Kumasi from Accra. They share similar traffic and driver characteristics. The road corridor is relatively heavily trafficked [3]. The intersections are characterized by long vehicular queues at the approaches of the intersections, especially during morning and evening peak periods of the day. However, for some time now observation of the current traffic situation at the selected intersections shows that very long queues of vehicles form on the intersection legs. Traffic congestion levels continue to rise daily at the selected intersections and there are significant travel time delays and lower levels of service [3].

Previous studies by [3] on the performance of the intersections attributed congestion to critical capacity, intersection controls and abuse to motorists and/or pedestrians. It was concluded that most of the sections on the 24th February road have their capacities approaching critical (v/c ratio>0.6) and that this contributes in part to the congestion which results in delay and subsequent poor performance of the road. It was therefore recommended that this section of the 24th February road will be highly critical in the next 3 to 5 years (period from 2004 – 2009). As part of the recommendations [3], the report proposed to improve upon the signalization and capacity at the intersection through; revision of signal timing, phasing plan, lane assignment/designation and inclusion of exclusive NMT phase phase at Anloga Junction, revision of signal timing and phasing plan and inclusion of concurrent NMT phase at Stadium intersection, revision of signal timing, phasing plan and inclusion of exclusive NMT phase and removal of illegal taxi rank at the filling station on the Asawasi leg at Amakom intersection.

In the study although the micro simulation Synchro/SimTraffic was reportedly used for the analysis however, the models were not calibrated. Even though some of the recommendations have been implemented at the selected intersections, long queues and frequent delays still persist during peak hour conditions [3]. Since in the application of micro simulation tools, one major step is calibration or adaptation before the prediction can be said to mimic site conditions, this needs to be checked. Also micro simulation tools application is relatively new in Ghana and the procedures for calibration, and application in modeling is not very well understood by practising engineers. Outputs of micro simulation models are useful to traffic engineers to identify existing problems and come out with interventions. A micro simulation model in Synchro is a software that makes use of limited data by trying to mimic the present traffic situation. This is then used in forecasting the future traffic conditions based on the present. This study seeks to contribute to the knowledge base in the area by calibrating the Synchro/SimTraffic models and using the concept to predict the performance of the selected intersections. This study therefore evaluated performance of the selected signalized intersections using micro simulation models in Synchro/SimTraffic.

II. METHODOLOGY

2.1 Calibration Procedure

The calibration procedure employed [4] approach by first selecting the intersection, collecting required input data for the Synchro model and collecting calibration data. These were then followed by comparing calibration data with simulated results from the field and finally calibrating the model.

2.2 Site Selection and Description

The selected signalized intersections were selected based on their accident and safety records in the past and also the levels of congestion associated with these intersections. The intersection ns were also selected for easy coordination. Fig. 1 shows the map of Kumasi showing the selected intersections.

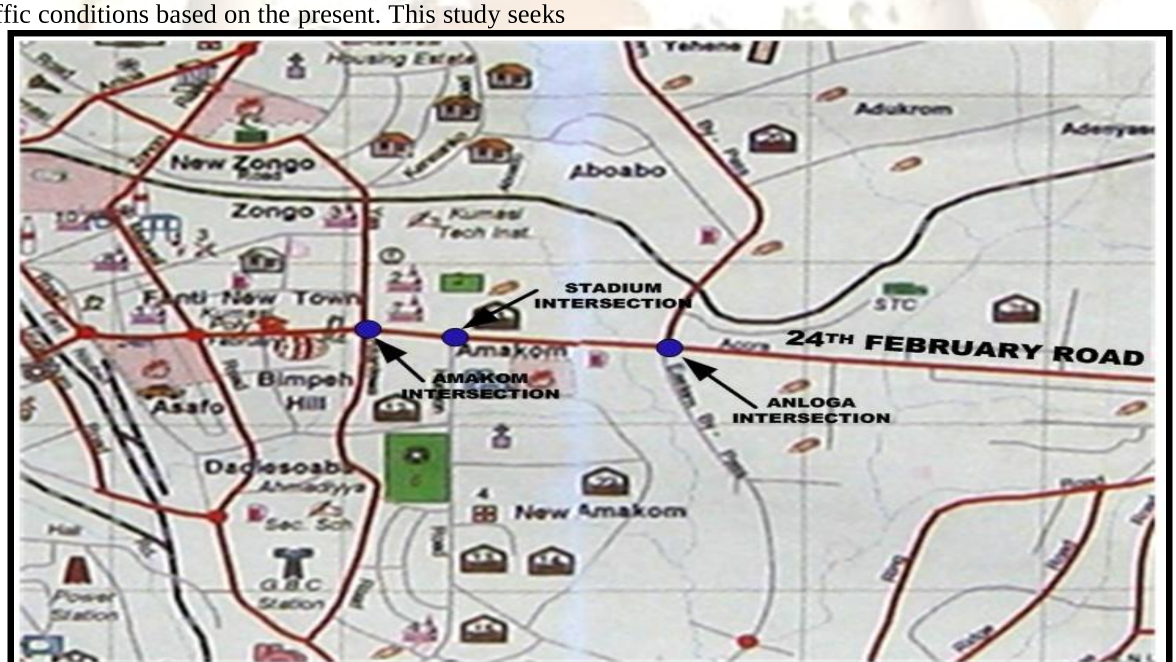

Figure 1: Map of Kumasi showing the selected signalized intersections on the 24th February Road

Source: from Department of Urban Roads, Kumasi-Ghana

2.2.1 Anloga Intersection

The Anloga junction is a signalised intersection comprising four principal arterials. It is about 2.6 km West of the KNUST junction. The intersection is 600m away from Oforikrom traffic light and 950m away from the Stadium traffic light. The intersection has four legs. Fig. 2 shows the geometry of Anloga intersection.

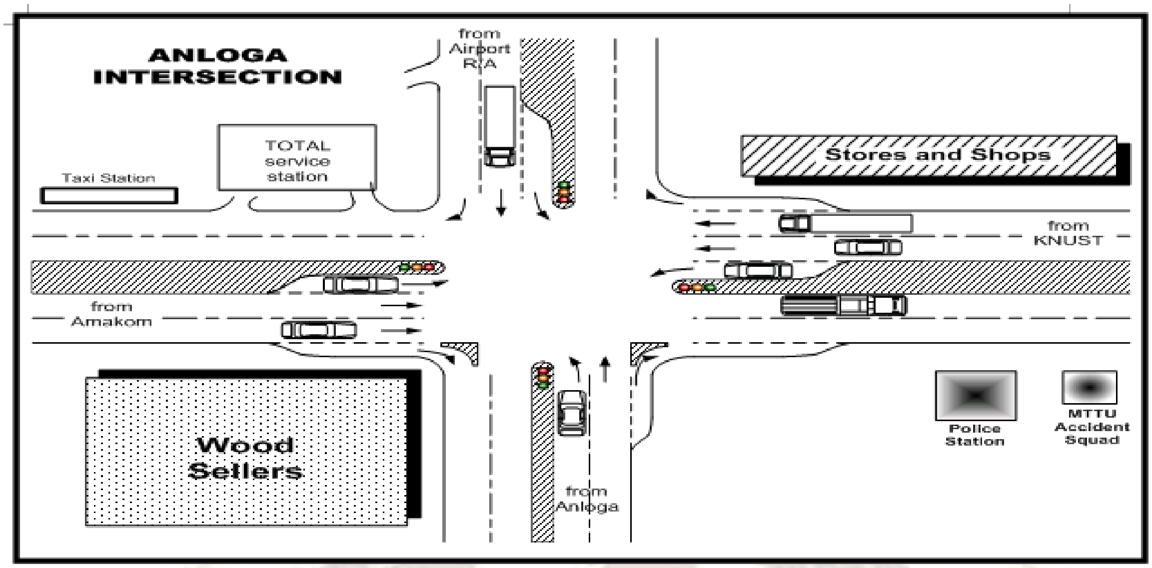

Figure 2: Geometry of Anloga Intersection

Source: from study

2.2.2 Stadium Intersection

The intersection with Hudson road is signalized and about 3.6km west of the KNUST junction. It is 950m away from Anloga junction and 350m away from the Amakom traffic light. The intersection has three legs with one approach/entry and exit lanes on the minor road, (Hudson road), and two approach/entry and exit lanes on the 24th February road. The two approach/entry and exit lanes road is a 2-lane dual carriageway, which is paved over its entire length. It is the intersection of a Principal arterial and a Minor arterial. Figure 3 is sectional map of Kumasi showing the Stadium intersection. Fig. 3 shows the geometry of stadium intersection.

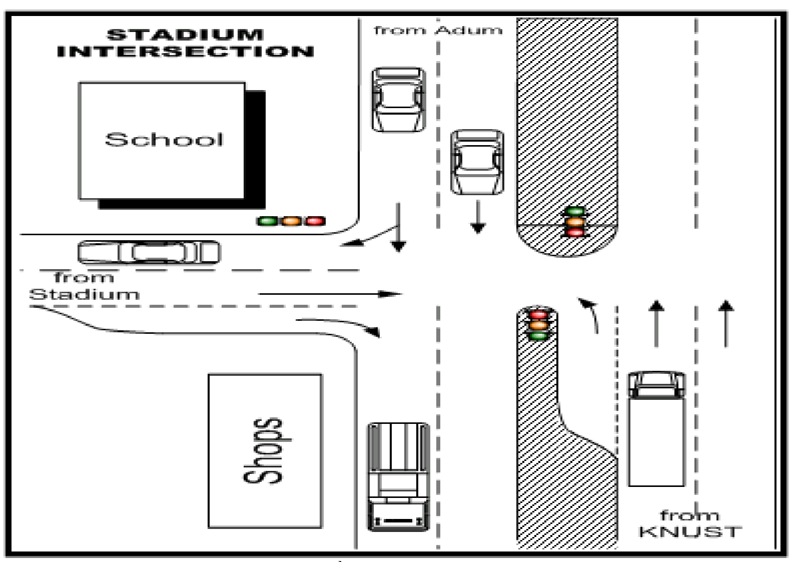

Figure 3: Geometry of 24th February/Hudson Road Intersection

Source: from study

2.2.3 Amakom Intersection

The Amakom traffic light, formerly called the Amakom roundabout, is a signalised intersection. It is about 4km West of the KNUST junction. The intersection is 350m away from Stadium traffic light and 700m away from labour roundabout. The intersection has four legs with one approach/entry and exit through lanes on each leg of the minor road, (Yaa Asantewaa road), and two approach/entry and exit through lanes on the 24th February road. It is the intersection of a Principal arterial and a Collector road.

Fig. 4 is the map of Kumasi showing the intersection under study.

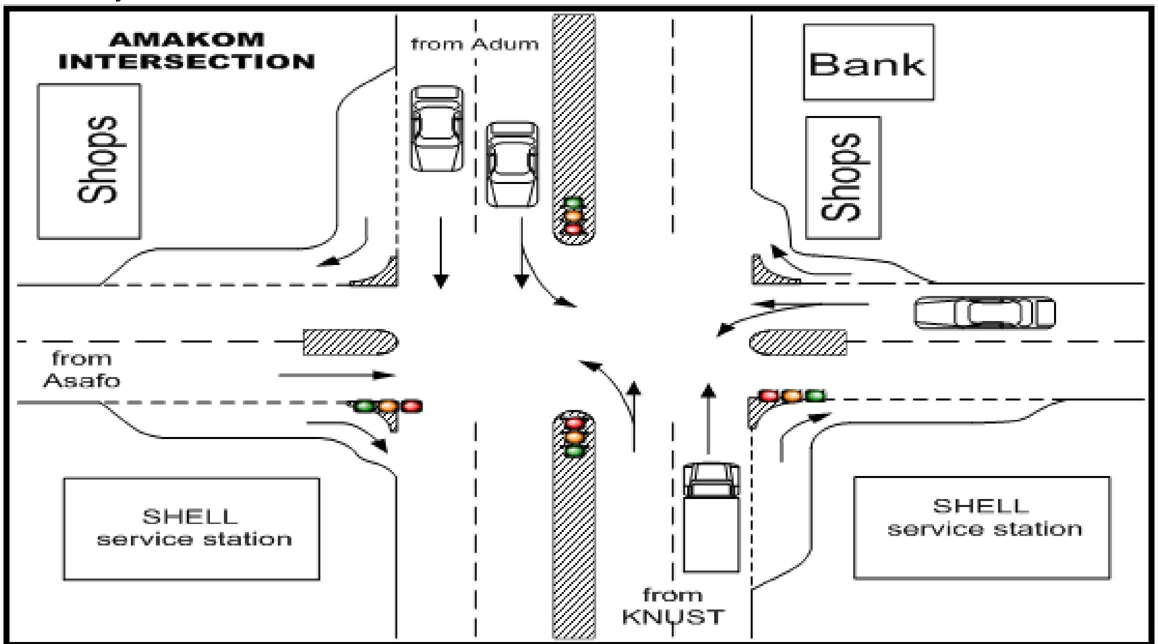

Figure 4: Geometry of 24th February/ Yaa Asantewaa Road Intersection

Source: from study

2.3Basic Theoretical Background

The concept of "Car-following" describes the detailed movement of vehicles proceeding close together in a single lane. This theory is based on the assumption that each driver reacts in some specific fashion to a stimulus from the vehicle ahead of him.

One of the oldest and most well known cases of the use of simulation in theoretical research is the "car-following" analysis based on the Generalized General Motors (GM) models. In these models a differential equation governs the movement of each vehicle in the platoon under analysis [5]. Car-following, like the intersection analysis, is one of the basic equations of traffic flow theory and simulation, and the analysis has been active after almost 40 years from the first trials [6]. The car-following theory is of significance in microscopic traffic flow theory and has been widely applied in traffic safety analysis and traffic simulation ([7]; [8]). There have been many car-following models in the past 60 years, and the models can be divided into two categories. One is developed from the viewpoint of traffic engineering and the other is based on statistical physics. From the perspective of traffic engineers [9], car-following models can be classified as stimulus-response models ([10]; [11]), safety distance models [12], psycho-physical models [13], and artificial intelligence models ([14]; [15]).

The car-following theory is based on a key assumption that vehicles will travel in the center line of a lane, which is unrealistic, especially in developing countries. In these countries, poor road conditions, irregular driving discipline, unclear road markings, and different lane widths typically lead to non-lane-based car-following driving [16]. Heterogeneous traffic, characterized by diverse vehicles, changing composition, lack of lane discipline, etc., results in a very complex behavior and a non-lane-based driving in most Asian countries [17]. Therefore, it is difficult for every vehicle to be moving in the middle of the lane. Vehicles are positioned laterally within their lanes, and the off central-line effect results in lateral separations. However, to the limit of our knowledge, the effect of lateral separation in the car-following process has been ignored by the vast majority of models. A few researchers have contributed efforts on this matter. [16] first developed a car-following model with lateral discomfort. He improved a stopping distance based approach that was proposed by [12], and presented a new car-following model, taking into account lateral friction between vehicles.

[18] proposed a non-lane-based car following model using a modified full-velocity difference model. All the above models have assumed that drivers are able to perceive distances, speeds, and accelerations. However, car-following behavior is a human process. It is difficult for a driver of the following vehicle to perceive minor lateral separation distances, and drivers may not

have precise perception of speeds and distances, not to mention accelerations.

2.3.1 Car-following Models

The logic used to determine when and how much a car accelerates or decelerates is crucial to the accuracy of a microscopic simulation model. Most simulation models use variations on the GM model. Although it was developed in the 1950s and 1960s, it has remained the industry standard for describing car-following behavior and continues to be verified by empirical data. A variation on the GM model is the PITT car-following model, which is utilized in FRESIM. The GM family of models is perceived to be the most commonly used in microscopic traffic simulation models and are, therefore, the focus of this article.

2.3.1.1Generalized General Motors Models

The first GM model modeled car-following is a stimulus-response process in which the following vehicle attempts to maintain space headway. When the speed of a leading vehicle decreases relative to the following vehicle, the following vehicle reacts by decelerating. Conversely, the following vehicle accelerates when the relative speed of the leading vehicle increases. This process can be represented by the first GM model, given below:

$$\ddot{\chi}_F = \alpha_F \times \left(\dot{\chi}_L(t) - \dot{\chi}_F(t) \right) \qquad \text{Eq. (1)}$$

Where: $\ddot{\chi}_F$ = acceleration of the following vehicle,

$\dot{\chi}_F$ = speed of the following vehicle,

$\dot{\chi}_L$ = speed of the leading vehicle,

α_F = sensitivity of the following vehicle, and

t = time.

2.3.1.2 PITT Car-following Model

FRESIM uses the PITT car-following model, which is expressed in terms of desired space headway, shown in the equation below.

$$h_s(t) = L + m + kV_2 + bk[V_1(t) - V_2(T)]^2 \qquad \text{Eq. (2)}$$

Where: $hs(t)$ = desired space headway at time t,

L = length of leading vehicle,

m = minimum car-following distance (PITT constant),

k = car-following sensitivity factor for following vehicle,

b = relative sensitivity constant,

$v_1(t)$ = speed of leading vehicle at time t, and

$v_2(t)$ = speed of following vehicle at time t.

Equation above can be solved for the following vehicle's acceleration, given by the equation below.

$$a = \frac{2 \times \left[x - y - L - m - V_2(K+T) - bk(V_1(t) - V_2(t))^2 \right]}{T^2 + 2KT} \qquad \text{Eq. (3)}$$

Where: a = the acceleration of the following vehicle,

T = the duration of the scanning interval,

x = position of the leading vehicle, and

y = position of the following vehicle.

2.4Algorithm on Synchro/SimTraffic software

Simulation is basically a dynamic representation of some part of the real world achieved by building a computer model and moving it through time. The results obtained from any simulation model will be as good as the model replicates the specific real world characteristics of interest to the analyst.

Once a vehicle is assigned performance and driver characteristics, its movement through the network is determined by three primary algorithms:

2.4.1 Car following

This algorithm determines behavior and distribution of vehicles in traffic stream. Synchro varies headway with driver type, speed and link geometry whereas SimTraffic generates lower saturation flow rates.

2.4.2 Lane changing

This is always one of the most temperamental features of simulation models. There are three types of lane-changing which includes

- Mandatory lane changes (e.g., a lane is obstructed or ends)
- Discretionary lane changes (e.g., passing)
- Positioning lane changes (e.g., putting themselves in the correct lane in order to make a turn): There is heavy queuing and this is a common problem for modeling positioning lane changes. Vehicles often passed back of queue before attempting lane

change and their accuracy relates to degree of saturation and number of access points such as congested conditions which requires farther look ahead and densely-spaced access (i.e. short segments) which presents a problem.

2.4.3 Gap Acceptance

Gap acceptance affects driver behavior at unsignalized intersections, driveways (e.g., right-in-right-out) and right-turn-on-red (RTOR) movements. If default parameters are too aggressive, vehicle delay will be underestimated and there is serious implication for frontage roads. Conversely, parameters which are too conservative may indicate need for a signal when one isn't necessary. Gap acceptance parameters are network-wide in SimTraffic.

2.5 Data Collection for Synchro

Microscopic simulation model Synchro has many model parameters. In order to build a Synchro simulation model for the selected intersections 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 traffic control systems. The second type is the observation data employed for the calibration of simulation model parameters such as average link speeds, headways and saturation flow using standard procedures.

2.5.1 Videotaping Traffic at Anloga Intersection

Anloga intersection was filmed because of the possibility of obtaining good elevation observer positions as well as capturing the complex traffic situations that exist at the intersection. The Anloga intersection was filmed for three (3) hours, from 0700hours and 1000 hours on Wednesday, the 22nd of April 2012. A total of 50 cycles during the morning peak conditions were captured. Two digital cameras were mounted on a tripod stand on top of a building at Anloga intersection to record traffic volumes and queues, headways and signal heads. The digital video recorder has the capability to record and playback simultaneously the signal from the cameras. This allows for relating in real-time discharging traffic with the state of the traffic signals. The video images on DVD were played back and analyzed manually on a laptop computer by number of trained observers. Traffic volumes and headways were collected from the films. Actual site observations and traffic signal timings were determined manually using stop watches at the intersection and checked with the designer's data. The films were played back and the site conditions also observed to explain some of the values obtained.

2.5.2 Extracting Data from Tapes

In the third phase of data collection, videotapes were played back and traffic volumes and headway data were extracted. First, the vehicles were carefully counted and time-measured while playing back the tapes. The data extraction was conducted on a lane-by-lane basis. Then, these measurements were used to calculate intermediate results and estimate saturation flows. Table 1 shows the traffic volume data and geometric data for Anloga intersection.

2.5.3 Manual Collection of Data at Stadium and Amakom Intersections

Manual collection of data was done at Stadium and Amakom intersections because it was difficult in getting good elevation observer positions as well as the fact that traffic situations are not as complex as Anloga intersection.

Traffic volume data and geometric data as indicated in Tables 2 and 3 for turning movements were collected manually at Stadium and Amakom intersections on Wednesday, the 13th and 27th of May 2012 respectively between 0700hours and 1000 hours during the morning peak period of the day. Traffic signal timings were also determined manually using stop watches at the Stadium and Amakom intersection. At Stadium and Amakom intersections, two enumerators each were positioned on each approach. The number of vehicles turning left, right and through traffic were counted and the times when an approach has green and red indications were recorded. Two other enumerators each also recorded headways and speeds of vehicles as they traverse the intersections.

From Tables 1, 2 and 3, it can be observed that a peak hour volumes of 3,660, 2,434 and 2,980 vehicles respectively were registered at Anloga, Stadium and Amakom intersections respectively. These values represented the worst traffic situation for an average day. Therefore, the existing control needs to be replaced with a more effective, efficient and reliable control scheme to ensure smooth and safe operations of all types.

Table 1: Summary Peak Hour Traffic Volume and Geometric Data at Anloga Intersection

From	To	Movement Code	Veh/hr	% Heavy Veh	No of Lanes	Lane Width	Storage Lengths	% of App Vol	App Vol (veh/hr)	% of total Int volume	Total Int Vol (veh/hr)
	Anloga	WBL	7	50	1	3.0	75	0.5			
KNUST	Amakom	WBT	1013	3	2	3.4		76.1	1332	36.4	
	Airport R/A	WBR	317	7	1	3.0	75	23.4			
	Airport R/A	EBL	291	12	1	3.4	86	18.8			
Amakom	KNUST	EBT	1211	6	2	3.3		78.4	1545	42.2	
	Anloga	EBR	43	15	1	4.7	54	2.8			3660
	Amakom	NEBL	141	9	1	4.8		63.5			
Anloga	Airport R/A	NEBT	30	40	1	4.8		13.5	222	6.1	
	KNUST	NEBR	51	21	1	4.7	34	23.0			
	KNUST	SWBL	144	16	1	3.0	65	25.7			
Airport R/A	Anloga	SWBT	20	60	1	3.8		3.6	561	15.3	
	Amakom	SWBR	3974	10	1	3.8	58	70.7			

Source: from study

Table 2: Summary Peak Hour Traffic Volume and Geometric Data at Stadium Junction Intersection

From	To	Movement Code	Veh/hr	% Heavy Veh	No of Lanes	Lane Width	Storage Lengths	% of App Vol	App Vol (veh/hr)	% of total Int volume	Total Int Vol (veh/hr)
	Stadium Jn	WBL	432	8	1	2.9	73	30.9			
Anloga Jn	Amakom	WBT	967	1	2	3.5		69.1	1399	57.5	
	N/A	WBR									
	N/A	EBL									
Amakom	Anloga Jn	EBT	805	5	2	3.7		95.2	846	34.8	2434
	Stadium Jn	EBR	41	13			shared	4.8			
	Amakom	NBL	106	4	1	3.6		56.1			
Stadium	N/A	NBT							189	7.7	
	Anloga Jn	NBR	83	20	1	2.7	34	43.9			

Source: from study

Table 3: Summary Peak Hour Traffic Volume and Geometric Data at Amakom Intersection

From	To	Movement Code	Veh/hr	% Heavy Veh	No of Lanes	Lane Width	Storage Lengths	% of App Vol	App Vol (veh/hr)	% of total Int volume	Total Int Vol (veh/hr)
	Amakom	WBL	117	22	1	3.8	56	8.5			
Anloga Jn	Kejetia	WBT	1153	3	2	2.9		83.4	1383	46.4	
	Asawasi	WBR	113	24	1	4.5	43	8.1			
	Asawasi	EBL	76	7	1	3.6	46	8.6			
Kejetia	Anloga Jn	EBT	742	2	2	3.6		83.6	886	29.7	
	Amakom	EBR	69	0			54	7.8			2980
	Kejetia	NBL	53	9			shared	24.7			
Amakom	Asawasi	NBT	153	17	1	3.4		71.2	215	7.2	
	Anloga Jn	NBR	9	100			16	4.1			
	Anloga Jn	SBL	120	13	1	4	23	24.2			
Asawasi	Amakom	SBT	222	13	1	4		44.8	496	16.7	
	Kejetia	SBR	154	7	1	3	22	31			

Source: from study

2.5.4 Signal Control Data for Selected Intersections

Cycle length is the time required for one complete sequence of signal indications (phases). Usually it is measured in seconds. Cycle lengths and phases for the intersection were recorded using stopwatch. Cycle lengths for the selected signalized intersection were obtained as 210, 68 and 172 seconds respectively. Table 4 shows the signal timing data for the selected intersections.

Table 4: Signal Timing Data for Selected Intersections

Intersection	Location	Cycle Length (C)	Actual Green Time (G)	Actual Yellow Time (Y)	Actual Red Time (R)	Total Lost Time (l_1+l_2)	Effective Green Time (g)
Anloga	From KNUST	210	93	4	2	4	95
	To KNUST	210	93	4	2	4	95
Stadium	From KNUST	68	35	4	2	4	37
	To KNUST	68	30	4	2	4	32
Amakom	From KNUST	172	56	4	2	4	58
	To KNUST	172	47	4	2	4	49

Source: from study

The resulting effective green time (g) was therefore calculated using the equation below

$$g = G + Y + R - (l_1 + l_2) \qquad \text{Eq. (4)}$$

Where l_1 = start up lost time
l_2 = clearance lost time

2.6 Calibration Data for Selected Intersections

In order to realistically model traffic at the intersection, it is important to have realistic calibration data. To calibrate the model for local condition speeds and Headways data were collected.

2.6.1 Spot Speed Data

Speed is the most important parameter describing the state of a given traffic stream. In a moving traffic stream, each vehicle travels at a different speed. Thus, the traffic stream does not have a single characteristic speed but rather a distribution of individual vehicle speeds. The spot speed data to and from KNUST approaches at the selected intersections were collected using the Doppler principle (radar). The speed data were collected as the tail of the vehicles crosses the stop bar. The first four vehicles in queue were not counted because they were accelerating from rest. A radar gun was operated by a single person. Operators randomly targeted the vehicles and recorded the digital readings displayed on the unit. Through traffic speeds to and from KNUST approaches at Anloga intersection were recorded for a maximum of 60 vehicles inclusive of private cars, commercial vehicles and heavy goods vehicles. Similarly, through traffic speeds from and to Tech approaches at Stadium and Amakom intersections were recorded for a maximum of 30 vehicles and 60 vehicles inclusive of private cars, commercial vehicles and heavy goods vehicles. The unit of speed is in km/hr.

2.6.2 Headway and Saturation Flow Data

The headway is the time starting from when the tail of the lead vehicle crosses the stop bar until the front of the following car crosses the stop bar. Headway data for three cycles each were collected at the intersection using stop watches. Tables 5, 6 and 7 show the summary of computed saturation flow, headway and speed for the selected intersections. The unit of saturation flow is passenger car units per hour per lane (pcu/hr/lane) and that of headway is in seconds (sec).

2.7 Calibration of the Synchro Models

The successful utilization of the model depends on selecting the proper values of the parameters that describe the traffic performance and driving behavior characteristics. Synchro/SimTraffic model is carefully calibrated and validated to provide meaningful results.

2.7.1 Chi Square Test Analysis (P-value)

This was used to determine the level of significance between the computed and simulated saturation flows, speeds and headways at the selected intersections. When $P < 0.05$, it was considered significant and $P > 0.05$ was considered not significant.

2.7.2 Paired sample T-Test Analysis

This analysis was carried out to either confirm or reject the chi squared test analysis. It was also used to evaluate the variation in the computed and simulated saturation flows, speeds and headways at the selected intersections.

2.7.3 Regression Analysis

Calibration of the model was based on regression analysis using traffic volume data to-and-from KNUST approaches at the selected intersections. It was carried out to establish whether speed or headway had a strong correlation with saturation flow. The predictors were speed and headway whiles the dependent variable was saturation flow.

2.8 Performance Assessment for the Selected Intersections

Levels of service and delay were used to evaluate the performance at the selected intersections. A Level of service is a letter designation that describes a range of operating conditions on a particular type of facility. The 1994 Highway Capacity defines levels of service as “qualitative measures that characterize operational conditions within a traffic stream and their perception by motorists and passengers.” Six levels of service are defined for capacity analysis. They are given letter designations A through F, with LOS A representing the best range of operating conditions and LOS F the worst. Delay is defined in terms of the average stopped time per vehicle traversing the intersections.

2.8.1Change of Phase without Geometric Improvement

For effective investigation of the optimized cycle lengths and offsets at the selected signalized intersections, three different alternatives with four-phase operational plan in were proposed at Anloga. Similarly, three (3) different alternatives with three-phase operational plan were proposed at Stadium intersection. Furthermore, two (2) different alternatives with four-phase operational plan were proposed at Amakom intersection.

2.8.2 Change of Phase with Geometric Improvement

The best alternative chosen was further investigated upon by the addition of lane(s) to the through put traffic at the selected intersections. This was done to determine whether there was improvement in the level of service. This is because improvement in the Level of Service (LOS) at the selected intersections would result in overall and enhanced performance on the 24th February road corridor.

2.8.3 Grade Separation Option

Continuous addition of lanes to the through put traffic will adversely affect the reservations at the intersection and when that condition persists, grade separation option will be considered to see how best to allow free flow of traffic to traverse the intersection concerned.

2.9 Sensitivity Analysis

This was carried out to verify in detail the sensitivity of the obtained results to the variation of the input parameters at the selected intersections.

III. RESULTS AND DISCUSSION

3.1 Saturation flow, Headway and Speed Data

Table 5 shows the summary of computed saturation flows and headways for the selected intersections.

Table 5: Computed saturation flow, headway and speed

Intersection	Performance measures	Direction	No. of vehicles	Mean	Maximum	Minimum	Standard Deviation
Anloga	Saturation flow	From KNUST	60	1662	4186	467	733.89
		To KNUST	60	1742	3913	403	712.85
	Headway	From KNUST	60	2.59	7.71	0.86	1.29
		To KNUST	60	2.56	8.93	0.92	1.51
	Speed	From KNUST	60	26.63	36	20	3.89
		To KNUST	60	28.43	46	20	6.11
Stadium	Saturation flow	From KNUST	30	1226	2707	360	570.71
		To KNUST	30	1643	2727	678	511.65
	Headway	From KNUST	30	3.7	10.0	1.33	1.99
		To KNUST	30	2.44	5.31	1.32	0.89
	Speed	From KNUST	30	26.9	33	22	3.02
		To KNUST	30	24.8	28	22	1.67
Amakom	Saturation flow	From KNUST	60	1489	3214	497	726.58
		To KNUST	60	1286	3186	256	582.51
	Headway	From KNUST	60	3.01	7.24	1.12	1.42
		To KNUST	60	3.54	14.08	1.13	2.18
	Speed	From KNUST	60	26.3	32	23	2.58
		To KNUST	60	26.5	30	22	1.96

Source: from study

3.1.1 Explanation of Saturation Flow Values

Saturation flow, which is the maximum rate of flow of traffic across the stop line at an intersection, is a very important measure in junction design and signal control applications. Low values of Saturation flow means less vehicles can cross the stop line when the signal turns green. Data collection techniques for the determination of saturation flow are well elaborated by [19].

The saturation flow from KNUST junction approach was 1662pcu per hour per lane and that to KNUST junction approach was 1742pcu per hour per lane at Anloga intersection.

Also, for the stadium intersection, the saturation flow rates from Tech approach was 1226pcu per hour per lane and that to Tech approach was 1643pcu per hour per lane. Similarly, the saturation flow rates from Tech approach was 1489pcu per hour per lane and that to Tech approach was 1254pcu per hour per lane for the Amakom intersection.

Video playbacks for Anloga intersection allowed for the observation of extraneous factors contributing to the reduction in saturation flows and flow interruptions at approaches for specific times and cycles some of which were rejected for analysis due to flow interruptions. This was attributed to vehicle mix, geometry of intersection, driver behaviour, public transport proportion in traffic stream, stops near intersection along routes (within 20m) and pedestrian indiscriminate crossing due to location of attractions and roadside activity. These were identified as principal factors affecting flow, which collaborates [20]; [21]; and [22], who have severally reported similar results. The presence of fuel service stations, high pedestrian volumes and erratic pedestrian crossing and roadside activities significantly affected the saturation flow rates at the Anloga intersection. High approach down grades and the presence of heavy vehicles at Anloga intersection were also found to be major contributory factors to the deviation of the saturation flow from the ideal.

3.2Results of Calibration of Synchro Models for Selected Intersections

3.2.1 Chi-squared Analysis (p-values)

Table 6 shows the results of the comparison between computed and simulated saturation flows, speeds and headways for the selected intersections using the Chi Square Test (p-value).

Table 6: Comparison of Computed and Simulated Performance measures

Intersection	Performance measures	Direction	Computed values Sc	Simulated values, Sa	Ratio (Sc/Sa)	Chi square Test (P-values
	Saturation flow	From KNUST	1662	1700	0.978	0.3269
		To KNUST	1742	1756	0.992	
Anloga	Headway	From KNUST	2.59	2.97	0.872	0.6525
		To KNUST	2.56	3.27	0.783	
	Speed	From KNUST	26.63	23	1.103	0.2383
		To KNUST	28.43	24	1.185	
	Saturation flow	From KNUST	1226	1267	0.968	0.2494
		To KNUST	1643	1643	1.000	
Stadium	Headway	From KNUST	3.7	3.06	1.209	0.58011
		To KNUST	2.44	3.18	0.767	
	Speed	From KNUST	26.9	25	1.076	0.4792
		To KNUST	24.8	22	1.127	
	Saturation flow	From KNUST	1489	1535	0.970	0.2398
		To KNUST	1286	1288	0.998	
Amakom	Headway	From KNUST	3.01	3.36	0.896	0.6902
		To KNUST	3.54	2.94	1.204	
	Speed	From KNUST	26.3	21	1.252	0.1714
		To KNUST	26.5	23	1.152	

Source: from study

P-values > 0.05 meant that there were no significant differences between the computed and simulated values in terms of saturation flow, headway and speed data for the selected signalized intersections. Table 6 indicated that the field saturated flow, headway and speed values were similar to the simulated saturated flow, headway and speed values obtained from Synchro for the selected intersections.

3.2.2 T-test analysis

For saturation flow at each approach of the selected intersections, the test results showed that there was no significant ($p>0.05$) difference between the computed and simulated saturation flow values. For headways at each approach of the selected intersections, the test results showed that there was no significant ($p>0.05$) difference between the computed and simulated headway values.

For speeds however at each approach of the selected intersections, the test results showed that there was significant ($p<0.05$) difference between the computed and simulated speed values. It was found that a certain percentage of the variation in the simulated speed values was explained by the field speed values and Eta squared values explained in Tables 7, 8 and 9. Detailed Results of level of significance test carried out at the selected Intersections (T-Test Analysis)

Table 7: Results of paired Sample Test at Anloga Intersection (Sat flow SIM)

Approaches	Paired Difference					t	Eta squared (%)	Sig. (2-tailed) p-value
	Mean	Standard deviation	Standard error mean	95% confidence interval of the difference				
				Lower	Upper			
Saturation from KNUST	-38.033	729.842	94.222	-226.6	150.505	-0.404	0.28	0.688
Saturation to KNUST	-14.450	711.929	91.910	-198.361	169.461	-0.157	4.2	0.876
Speed from KNUST	-3.650	1.696	0.219	-4.088	-3.212	-16.673	82.5	0.000
Speed to KNUST	4.433	5.444	0.703	3.027	5.840	6.308	4.2	0.000
Headway from KNUST	-0.673	1.299	0.168	-1.009	-0.338	-4.014	21.5	0.000
Headway to KNUST	-0.409	1.519	0.196	-0.802	-0.017	-2.089	0.0029	0.041

Source: from study

Paired t-Test results in Table 7 showed there was no significant ($p> 0.05$) saturation flow variation of vehicular movements between the field and the simulated situation for approaches from both direction. Eta squared statistic 0.0028 indicated a small size effect; meaning only 0.28% of the variation in the simulated saturation flow was explained by the field saturation flow from KNUST. Again Eta squared statistic 0.00042 indicated a small size effect which meant that only 4.2% of the variation in the simulated saturation flows was explained by the field saturation to KNUST.

Paired t-Test results in Table 7 showed there was significant ($p<0.05$) speed variation of vehicular movements between the field and the simulated situation for approaches from both direction. Eta squared statistic 0.825 indicated a large size effect; meaning 82.5% of the variation in the simulated speed was explained by the field speed from KNUST. Again Eta squared statistic 0.403 indicated a large size effect which meant that 40.3% of the variation in the simulated speeds was explained by the field speeds to KNUST.

Paired t-Test results in Table 7 showed there was significant ($p<0.05$) headway variation of vehicular movements between the field and the simulated situation for approaches from both direction. Eta squared statistic 0.215 indicated a small size effect; meaning 21.5% of the variation in the simulated headway was explained by the field headway from KNUST. Again Eta squared statistic 0.000029 indicated a small size effect which meant that 0.0029% of the variation in the simulated headways was explained by the field headways to KNUST.

Table 8: Results of paired Sample Test at Stadium Junction Intersection (Sat flow SIM)

Approaches	Paired Difference					t	Eta squared (%)	Sig. (2-tailed) p-value
	Mean	Standard deviation	Standard error mean	95% confidence interval of the difference				
				Lower	Upper			
Saturation from KNUST	-41.067	571.345	104.313	-254.410	172.277	-0.394	0.53	0.697
Saturation to KNUST	0.633	509.752	93.068	-189.71	190.978	0.007	0.00017	0.995
Speed from KNUST	1.900	2.280	0.416	1.049	2.751	4.565	41.8	0.000
Speed to KNUST	2.800	0.997	0.182	2.428	3.172	15.389	89.1	0.000
Headway from KNUST	0.5213	1.997	0.365	-0.225	1.267	1.430	6.59	0.164
Headway to KNUST	0.623	0.913	0.167	-0.964	-0.283	-3.741	32.23	0.001

Source: from study

Paired t-Test results in Table 8 showed there was no significant ($p > 0.05$) saturation flow variation of vehicular movements between the field and the simulated situation for approaches from both direction. Eta squared statistic 0.0053 indicated a small size effect; meaning only 0.53% of the variation in the simulated saturation flow was explained by the field saturation flow from KNUST. Again Eta squared statistic 1.69×10^{-6} indicated a small size effect which meant that only 0.00017% of the variation in the simulated saturation flows was explained by the field saturation to KNUST.

Paired t-Test results in Table 8 showed there was significant ($p<0.05$) speed variation of vehicular movements between the field and the simulated situation for approaches from both direction. Eta squared statistic 0.418 indicated a large size effect; meaning 41.8% of the variation in the simulated speed was explained by the field speed from KNUST. Again Eta squared statistic 0.891 indicated a large size effect which meant that 89.1% of the variation in the simulated speeds was explained by the field speeds to KNUST.

Paired t-Test results in Table 8 showed there was no significant ($p > 0.05$) headway variation of vehicular movements between the field and the simulated situation from KNUST and significant ($p<0.05$) headway variation of vehicular movements between the field and the simulated situation to KNUST. Eta squared statistic 0.0659 indicated a moderate size effect; meaning 6.59% of the variation in the simulated headway was explained by the field headway from KNUST. Again Eta squared statistic 0.3223 indicated a large size effect which meant that 32.23% of the variation in the simulated headways was explained by the field headways to KNUST.

Table 9: Results of paired Sample Test at Amakom Intersection (Sat flow SIM)

Approaches	Paired Difference					t	Eta squared (%)	Sig. (2-tailed) p-value
	Mean	Standard deviation	Standard error mean	95% confidence interval of the difference				
				Lower	Upper			
Saturation from KNUST	-45.800	728.645	94.068	-234.029	142.429	-0.487	0.4	0.628
Saturation to KNUST	-2.683	580.118	74.893	-152.54	147.177	-0.036	0.0022	0.972
Speed from KNUST	5.300	1.843	0.291	4.711	5.889	18.193	84.9	0.000
Speed to KNUST	3.475	1.198	0.189	3.092	3.858	18.345	85.1	0.000

Headway from KNUST	-0.353	1.414	0.183	-0.719	0.119	-1.936	6.0	0.058
Headway to KNUST	0.600	2.211	0.285	0.029	1.171	2.103	7.0	0.040

Source: from study

Paired t-Test results in Table 9 showed there was no significant ($p > 0.05$) saturation flow variation of vehicular movements between the field and the simulated situation for approaches from both direction. Eta squared statistic 0.004 indicated a small size effect; meaning only 0.4% of the variation in the simulated saturation flow was explained by the field saturation flow from KNUST. Again Eta squared statistic 0.000022 indicated a small size effect which meant that only 0.0022% of the variation in the simulated saturation flows was explained by the field saturation to KNUST. There was significant ($p<0.05$) speed variation of vehicular movements between the field and the simulated situation for approaches from both direction. Eta squared statistic 0.849 indicated a large size effect; meaning 84.9% of the variation in the simulated speed was explained by the field speed from KNUST. Again Eta squared statistic 0.851 indicated a large size effect which meant that 85.1% of the variation in the simulated speeds was explained by the field speeds to KNUST. There was no significant ($p>0.05$) headway variation of vehicular movements between the field and the simulated situation from KNUST and significant ($p<0.05$) headway variation of vehicular movements between the field and the simulated situation to KNUST. Eta squared statistic 0.060 indicated a small size effect; meaning 6.0% of the variation in the simulated headway was explained by the field headway from KNUST. Again Eta squared statistic 0.070 indicated a moderate size effect which meant that 7.0% of the variation in the simulated headways was explained by the field headways to KNUST.

3.2.3 Regression Analysis

In order to calibrate the model, a regression analysis was carried out from and to KNUST approaches at the selected intersections to establish which of either speed or headway was a better predictor or has a strong correlation with saturation flow. Table 10 shows the comparison of field and simulated saturation flow, headway and speed for the selected intersections.

Table 10: Comparison of Field and Simulated Saturation flow, Speed and Headway for Selected Intersections

Intersection	Approach	Conditions	R	R square (R^2)	Adjusted R Square (R^2)	Standard Error of the Estimate
Anloga	From KNUST	Field	0.797	0.635	0.622	451.078
		Simulated	1.000	1.000	1.000	0.000
	To KNUST	Field	0.826	0.683	0.672	408.457
		Simulated	1.000	1.000	1.000	0.000
Stadium	From KNUST	Field	0.856	0.733	0.713	305.676
		Simulated	1.000	1.000	1.000	0.000
	To KNUST	Field	0.922	0.850	0.839	205.364
		Simulated	1.000	1.000	1.000	0.000
Amakom	From KNUST	Field	0.865	0.748	0.735	367.072
		Simulated	1.000	1.000	1.000	0.000
	To KNUST	Field	0.827	0.685	0.668	337.738
		Simulated	1.000	1.000	1.000	0.232

Source: from study

For Anloga, it was established that headway had a strong correlation with saturation flow from KNUST approach with $R^2_{field} = 0.635$ and $R^2_{simuated} = 1.000$. Similarly headway had a strong correlation with saturation flow to KNUST approach with $R^2_{field} = 0.683$ and $R^2_{simuated} = 1.000$

The adjusted R^2 value was expressed in percentage as 62.2%. Thus the model (headway and speed) explained 62.2% of the variance in the saturation flows for field condition as shown in Table 10..

The adjusted R^2 value was expressed in percentage as 67.2%. Thus the model (headway and speed) explained

67.2% of the variance in the saturation flows for field condition.

For Stadium, it was established that headway had a strong correlation with saturation flow from KNUST approach with R^2_{field} = 0.733 and $R^2_{simuated}$ = 1.000. Similarly headway had a strong correlation with saturation flow to KNUST approach with R^2_{field} = 0.85 and $R^2_{simuated}$ = 1.000 as shown in Table 10.

The adjusted R^2 value was expressed in percentage as 71.3%. Thus the model (headway and speed) explained 62.2% of the variance in the saturation flows for field condition as shown in Table 10

The adjusted R^2 value was expressed in percentage as 83.9%. Thus the model (headway and speed) explained 83.9% of the variance in the saturation flows for the field condition.

For Amakom intersection, it was established that headway had a strong correlation with saturation flow from KNUST approach with R^2_{field} = 0.748 and $R^2_{simuated}$ = 1.000. Similarly headway had a strong correlation with saturation flow to KNUST approach with R^2_{field} = 0.683 and $R^2_{simuated}$ = 1.000 as can be seen in Table 10.

The adjusted R^2 value was expressed in percentage as 73.5%. Thus the model (headway and speed) explained 73.5% of the variance in the saturation flows for field condition as shown in Table 10. The adjusted R^2 value was expressed in percentage as 66.8%. Thus the model (headway and speed) explained 66.8% of the variance in the saturation flows for field condition.

The adjusted R^2 value was expressed in percentage as 100% for the selected intersections. This therefore meant that the model (headway and speed) explained 100% of the variance in the saturation flows for simulated conditions for the selected intersections.

Table 11: Results of Field and Simulated Conditions co-efficients (From KNUST) Anloga Intersection

Predictors	Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
	B	Std. Error	Beta			Tolerance	VIF
Constant	3362.416	576.527		5.832	0.000		
SpeedFIELD	-18.644	21.366	-0.070	-0.873	0.387	0.995	1.005
Headway for KNUST Approach	-463.630	47.144	-0.789	-9.834	0.000	0.995	1.005
Constant	1373.000	0.000		6×10^8	0.000		
Speed for KNUST Approach	1.98×10^{15}	0.000	0.000	0.000	1.000	0.738	1.355
HeadwaySIM	100.000	0.000	1.000	3×10^8	0.000	0.738	1.355

Source: from study

For the field condition, the model showed that headway (-0.789) made a unique contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (-0.070) which made a small contribution to the model as can be seen in Table 11. Headway had a significance level of 0.000 which was less than 0.05; thus contributed greatly to the prediction of saturation flow whiles Speed (0.387) made an insignificant contribution.

The field equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = -18.644u - 463.630h + 3362.416 \qquad \text{Eq. (5)}$$

For the simulated condition, the model showed that headway (1.000) made the strongest contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (0.000) which made no contribution to the model as can be seen in Table 11. Headway had a significance level of 0.000 which was less than 0.05; thus contributed to the prediction of saturation flow whiles Speed (1.000) made an insignificant contribution.

The simulated equation connecting saturation flow (q), headway (h) and speed (u) is:

$$q = \left(1.98 \times 10^{-15}\right)u + 100h + 1373 \qquad \text{Eq. (6)}$$

Table 12: Results of Field and Simulated Conditions co-efficients (To KNUST) Anloga Intersection

Predictors	Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
	B	Std. Error	Beta			Tolerance	VIF
Constant	3252.782	279.859		11.623	0.000		
Speed to KNUST Approach	-17.815	8.758	-0.153	-2.034	0.047	0.987	1.014
Headway to KNUST Approach	-392.412	35.508	-0.830	-11.051	0.000	0.987	1.014
Constant	1459.000	0.000		$9x10^8$	0.000		
SpeedSIM	$3.94x10^{-14}$	0.000	0.000	0.000	1.000	0.767	1.304
HeadwaySIM	100.000	0.000	1.000	$4x10^8$	0.000	0.767	1.304

Source: from study

For the field condition, the model showed that headway (-0.830) made a unique contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (-0.153) which made less contribution to the model as shown in Table 12. Headway had a significance level of 0.000 which was less than 0.05, thus greatly contributed to the prediction of saturation flow though speed (0.047) made insignificant contribution.

The field equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = -17.815u - 392.412h + 3252.782 \qquad \text{Eq. (7)}$$

For the simulated condition, the model showed that headway (1.000) made a unique contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (0.000) which made no contribution to the model as shown in Table 12. Headway had a significance level of 0.000 which was less than 0.05, thus greatly contributed to the prediction of saturation flow whiles speed (1.000) made insignificant contribution.

The simulated equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = \left(3.94 \times 10^{-14}\right)u + 100h + 1459 \qquad \text{Eq. (8)}$$

Table 13: Results of Field and Simulated Conditions co-efficients (From KNUST) Stadium Intersection

Performance Measures	Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
	B	Std. Error	Beta			Tolerance	VIF
Constant	2404.311	508.995		4.724	0.000		
Speed for KNUST Approach	-10.446	19.046	-0.055	-0.548	0.588	0.973	1.028
Headway for KNUST Approach	-242.446	28.918	-0.845	-8.384	0.000	0.973	1.028
Constant	949.000	0.000					
SpeedSIM	$4.3x10^{-14}$	0.000	0.000			0.756	1.323
HeadwaySIM	100.000	0.000	1.000			0.756	1.323

Source: from study

For the field condition, the model showed that headway (-0.845) made a strong contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (-0.055) which made less contribution to the model as can be seen in Table 13. Headway had a significance level of 0.000 which was less than 0.05; thus greatly contributed to the prediction of saturation flow whiles speed (0.548) made insignificant contribution.

The field equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = -10.446u - 242.446h + 2404.311 \qquad \text{Eq. (9)}$$

For the simulated condition, the model showed that headway (1.000) made a unique contribution in explaining the saturation flow when the variance in the

model was controlled for as compared to that of speed (0.000) which made no contribution to the model as can be seen in Table 13.
The simulated equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = \left(-4.3\times10^{-14}\right)u + 100h + 949 \qquad \text{Eq. (10)}$$

Table 14: Results of Field and Simulated Conditions co-efficients (To KNUST) Stadium Intersection

Performance Measures	Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
	B	Std. Error	Beta			Tolerance	VIF
Constant	3057.240	571.109		5.353	0.000		
SpeedField	-5.109	23.856	-0.017	-0.214	0.832	0.917	1.090
Headway	-528.143	44.824	-0.917	-11.783	0.000	0.917	1.090
Constant	1337.000	0.000					
SpeedSIM	$1.47\text{x}10^{-14}$	0.000	0.000			0.756	1.323
HeadwaySIM	100.000	0.000	1.000			0.756	1.323

Source: from study

For the field condition, the model established that headway (0.917) made a unique contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (0.017) which made less contribution to the model as shown in Table 14. Headway had a significance level of 0.000 which was less than 0.05, thus greatly contributed to the prediction of saturation flow though speed (0.832) made insignificant contribution.
The field equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = -5.109u - 528.143h + 3057.240 \qquad \text{Eq. (11)}$$

For the simulated condition, the model established that headway (1.000) made a unique contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (0.000) which made no contribution to the model as shown in Table 14.
The simulated equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = \left(1.47\times10^{-14}\right)u + 100h + 1337 \qquad \text{Eq. (12)}$$

Table 15: Results of Field and Simulated Conditions co-efficients (From KNUST) Amakom Intersection

Predictors	Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
	B	Std. Error	Beta			Tolerance	VIF
Constant	2714.454	622.587		4.360	0.000		
Speed from KNUST Approach	6.586	22.765	0.024	0.289	0.774	0.998	1.002
Headway from KNUST Approach	-476.444	45.546	-0.864	-10.461	0.000	0.998	1.002
Constant	1199.000	0.000		$5\text{x}10^{8}$	0.000		
SpeedSIM	$1.27\text{x}10^{-14}$	0.000	0.000	0.000	1.000	0.877	1.141
HeadwaySIM	100.000	0.000	1.000	$3\text{x}10^{8}$	0.000	0.877	1.141

Source: from study

For the field condition, the model established that headway (-0.864) made a unique contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (0.024) which made a less contribution to the model as can be seen in Table 15. Headway has a significance level of 0.000 which was less than 0.05; thus greatly contributed to the prediction of saturation flow whiles speed (0.774) made insignificant contribution.
The field equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = 6.586u - 476.44h + 2741.454 \qquad \text{Eq. (13)}$$

For the simulated condition, the model established that headway (1.000) made the strongest contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (0.000) which made no contribution to the model as can be seen in Table 15. Headway has a significance level of 0.000 which made significant contribution to the prediction of saturation flow whiles Speed (1.000) made an insignificant contribution.

The simulated equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = \left(1.27 \times 10^{-14}\right)u + 100h + 1199 \qquad \text{Eq. (14)}$$

Table 16: Results of Field and Simulated Conditions co-efficients (To KNUST) Amakom Intersection

Predictors	Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.	Collinearity Statistics Tolerance	Collinearity Statistics VIF
Constant	3073.799	1014.829		3.029	0.004		
Speed to KNUST Approach	-24.841	34.732	-0.083	-0.715	0.479	0.630	1.586
Headway to KNUST Approach	-333.976	44.364	-0.875	-7.528	0.000	0.630	1.586
Constant	983.996	1.376		715.028	0.000		
Speed SIM	0.270	0.045	0.012	5.969	0.000	0.877	1.141
Headway SIM	101.318	0.203	1.004	499.941	0.000	0.877	1.141

Source: from study

For the field condition, the model established that headway (-0.875) made the strongest contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed (0.083) which made a less contribution to the model as shown in Table 16. Headway had a significance level of 0.000 which was less than 0.05; thus greatly contributed to the prediction of saturation flow whiles Speed (0.479) made an insignificant contribution.

The field equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = -24.841u - 333.976h + 3073.799 \qquad \text{Eq. (15)}$$

For the simulated condition, the model established that headway (1.004) made the strongest contribution in explaining the saturation flow when the variance in the model was controlled for as compared to speed (0.012) which made a less contribution to the model as shown in Table 16. Headway had a significance level of 0.000 which was less than 0.05; thus greatly contributed to the prediction of saturation flow as well as Speed (0.000).

The simulated equation connecting saturation flow (q), speed (u) and headway (h) is:

$$q = 0.27u + 101.318h + 983.996 \qquad \text{Eq. (16)}$$

3.3 Comparison of Computed and Simulated Performance Indicators

Computed manual performance measures at the selected intersections were compared with the simulated performance indicators generated by the calibrated Synchro model as can be seen in Table 17.

Table 17: Comparison of Computed and Simulated Performance Indicators

Performance Indicators	Anloga		Stadium		Amakom	
	Computed	Adjusted	Computed	Adjusted	Computed	Adjusted
Cycle Length	210	200	68	65	172	193
v/c ratio	1.46	2.79	1.53	0.78	1.78	1.48
Int. Delay (s)	94	183.5	37	47.7	126	157.1
Int. LOS	F	F	D	D	F	F
ICU (%)		70.4		63.2		67.6
Offsets		135.0		41		133

Source: from study

From Table 17, Anloga and Amakom intersections had level of service F. The level of service F described a forced-flow operation at low speeds, where volumes were below capacity. These conditions usually resulted from queues of vehicles backing up a restriction downstream. Speeds were reduced substantially and stoppages occurred for short or long periods of time because of the downstream congestion.

It represented worst conditions. Stadium intersection had level of service D which approached unstable flow, with tolerable operating speeds being maintained, though considerably affected by changes in operating conditions. Drivers had little freedom to maneuver, and comfort and convenience were low. These conditions could be tolerated, however, for short periods of time. It represented intermediate conditions.

3.4 Analysis of Alternative Phasing Plans

3.4.1 Change in phasing plan without Geometric Improvement

Figs. 5 to 7 show the existing traffic situations together with their possible alternative phasing plans at the selected intersections.

Anloga Intersection

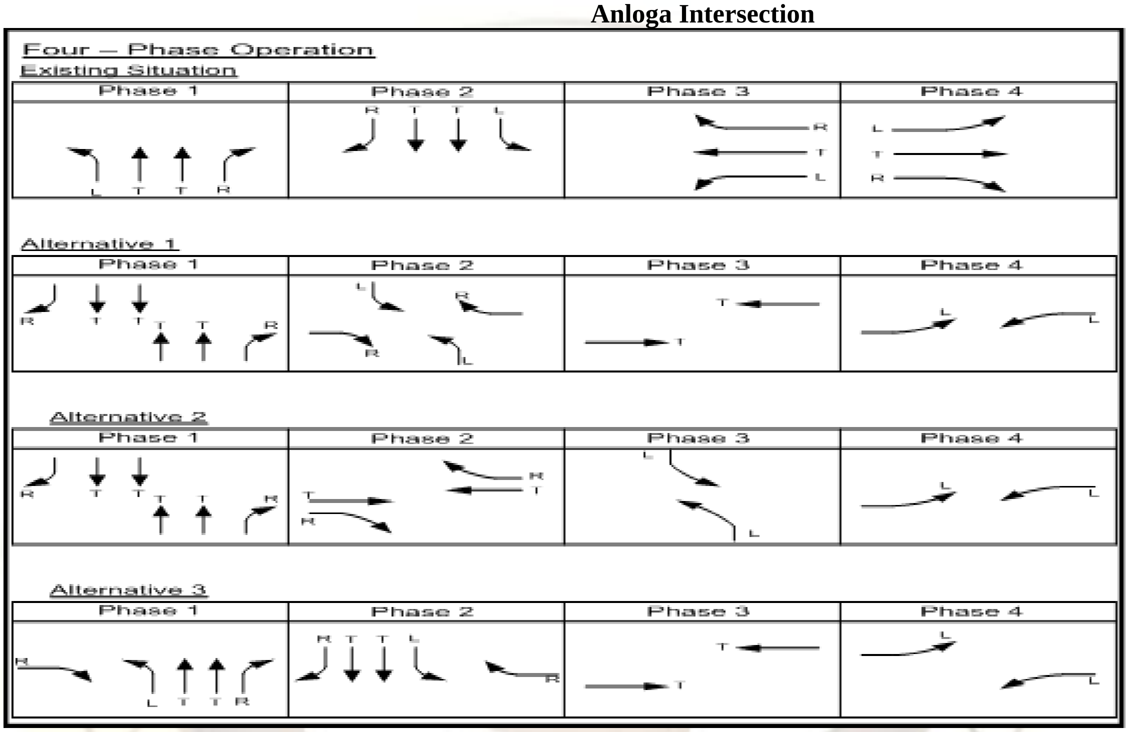

Figure 5: Existing traffic with alternative phasing plans at Anloga intersection

Source: from study

Table 18: Results of Performance Indicators for three Different Alternatives

Performance Indicators	Existing Situation	Alternative 1	Alternative 2	Alternative 3
Cycle Length(s)	184	176	176	176
v/c ratio	2.77	1.25	2.65	2.65
Int. Delay (s)	183.5	103.2	170.7	170.7
Int. LOS	F	F	F	F
ICU (%)	70.4	70.4	70.4	70.4
Offsets	135.0	24	24	24

Source: from study

Out of the three possible alternative phasing plans at the Anloga intersection, alternative 1 gave out the best optimized cycle length of 176secs with an offset of 24secs and an intersection delay of 103.2secs as shown in Table 18. These indicators compared to the existing indicators were better in terms of cycle length, v/c ratio and delay. Level of service F described a forced-flow operation at low speeds, where volumes were below capacity. These conditions usually resulted from queues of vehicles backing up a restriction downstream. Speeds were reduced substantially and stoppages occurred for short or long periods of time because of the downstream congestion. It represented worst conditions.

Stadium Intersection

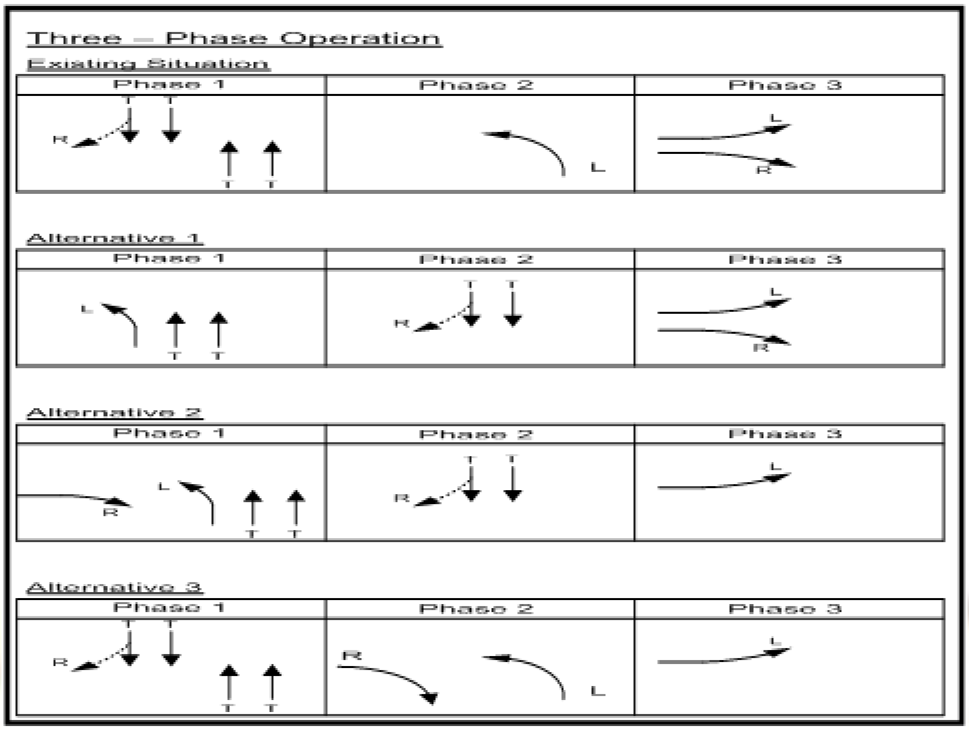

Figure 6: Existing traffic with alternative phasing plans at Stadium Junction intersection

Source: from study

Table 19: Results of performance indicators for three Different Alternatives

Performance Indicators	Existing Situation	Alternative 1	Alternative 2	Alternative 3
Cycle Length	65	65	60	60
v/c ratio	0.78	0.78	0.78	0.78
Int. Delay (s)	47.7	47.7	45.7	45.7
Int. LOS	D	D	D	D
ICU (%)	63.2	63.2	63.2	63.2
Offsets	41	41	41	41

Source: from study

Out of the three possible alternative phasing plans at the Stadium Junction intersection, alternative 2 gave out the best optimized cycle length of 60secs with an offset of 41secs and an intersection delay of 45.7secs as in Table 19. These indicators compared to the existing indicators were better in terms of cycle length, v/c ratio and delay and yet the LOS was still D. Level of service D approached unstable flow, with tolerable operating speeds being maintained, though considerably affected by changes in operating conditions. Drivers had little freedom to maneuver, and comfort and convenience were low. These conditions could be tolerated, however, for short periods of time. It represented intermediate conditions. Site observations at the intersection however showed that the traffic congestion was mostly associated with the left turning traffic towards stadium. The storage length for the vehicles was exceeded and therefore the vehicles spilled back into the double lane for the through traffic. This therefore reduced the saturation flow for the through traffic at the intersection when the indication turned green.

Amakom Intersection

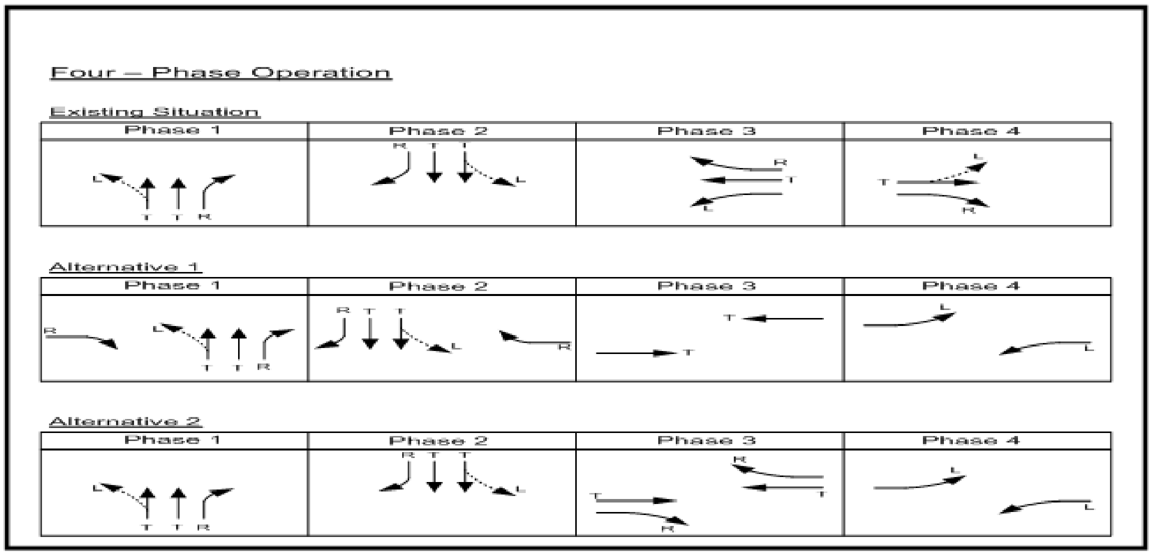

Figure 7: Existing traffic with alternative phasing plans at Amakom intersection

Source: from study

Table 20: Results of performance indicators for two Different Alternatives

Performance Indicators	Existing Situation	Alternative 1	Alternative 2
Cycle Length	193	170	172
v/c ratio	1.48	1.48	1.48
Int. Delay (s)	157.9	154.6	160.3
Int. LOS	F	F	F
ICU (%)	67.6	67.6	67.6
Offsets	133	133	133

Source: from study

Out of the two possible alternative phasing plans at the Amakom intersection, alternative 1 gave out the best optimized cycle length of 170secs with an offset of 133secs and an intersection delay of 154.6secs as shown in Table 20. These indicators compared to the existing indicators were better in terms of cycle length, v/c ratio and delay. Level of service F described a forced-flow operation at low speeds, where volumes were below capacity. These conditions usually resulted from queues of vehicles backing up a restriction downstream. Speeds were reduced substantially and stoppages occurred for short or long periods of time because of the downstream congestion. It represented worst conditions.

3.4.2 Change in phasing plan with Geometric Improvement

The best alternative from the phasing plan in Figures 5, 6 and 7 were further investigated upon by adding lane(s) to the through put traffic at the selected intersections.

Anloga Intersection

The best alternative from the phasing plan in Fig. 5 was further investigated upon by adding lane(s) to the through put traffic at Anloga intersection. Table 21 shows the summary of performance indicators with geometric improvement.

Table 21: Summary of performance indicators with geometric improvement at Anloga Intersection

Performance Indicators	Existing Situation	Alternative 1	Addition of 1 lane	Addition of 2 lanes
Cycle Length	184	176	176	176
v/c ratio	2.77	1.25	0.86	0.78
Int. Delay (s)	183.5	103.2	62.7	58.6
Int. LOS	F	F	E	E
ICU (%)	70.4	70.4	62.0	57.1
Offsets	135.0	24	24	24

Source: from study

There was the need to introduce geometric improvement as correctional measure. With the addition of one lane on the major approach through put traffic, the v/c ratio reduced from 1.25 to 0.86 with a corresponding reduction in the delay in Table 21. The level of service also changed from F to E meaning that there had been an improvement at the intersection. When 2 lanes each were added to each major through put approach, the v/c ratio again reduced from 0.86 to 0.78 with a corresponding reduction in the delay. The level of service E could not be described by speed alone, but represented operations at lower operating speeds, typically, but not always, in the neighborhood of 30miles per hour, with volumes at or near the capacity of the highway. Flow was unstable, and there may be stoppages of momentary duration. This level of service was associated with operation of a facility at capacity flows. It represented intermediate conditions and performance had improved slightly at the intersection. Similarly, the existing reservation was further checked against the geometric improvement.

Stadium Intersection

The best alternative from the phasing plan in Fig. 6 was further investigated upon by adding lane(s) to the through put traffic at the intersection. Table 22 shows the summary of performance indicators with geometric improvement.

Table 22: Summary of performance indicators with geometric improvement at Stadium Intersection

Performance Indicators	Existing Situation	Alternative 2	Addition of 1 lane	Addition of 2 lanes
Cycle Length	65	60	60	60
v/c ratio	0.78	0.78	0.78	0.78
Int. Delay (s)	47.7	45.7	40.7	39.0
Int. LOS	D	D	D	D
ICU (%)	63.2	63.2	56.2	52.1
Offsets	41	41	41	41

Source: from study

There was the need to introduce geometric improvement as correctional measure. When one lane was added to the major through put traffic from each approach, the v/c ratio was 0.78 with a corresponding reduction in delay as in Table 22. When 2 lanes each were added to each major through put approach, the v/c ratio was again 0.78 with a corresponding reduction in delay. Level of service D approached unstable flow, with tolerable operating speeds being maintained, though considerably affected by changes in operating conditions. Drivers had little freedom to maneuver, and comfort and convenience were low. These conditions could be tolerated, however, for short periods of time. It represented intermediate conditions. Similarly, the existing reservation was further checked against the geometric improvement.

Amakom Intersection

The best alternative in Fig. 7 was further investigated upon by adding lane(s) to the through put traffic at Amakom intersection. Table 23 shows the summary of performance indicators with geometric improvement.

Table 23: Performance indicators with geometric improvement at Amakom Intersection

Performance Indicators	Existing Situation	Alternative 1	Addition of 1 lane	Addition of 2 lanes
Cycle Length	193	170	170	170
v/c ratio	1.48	1.48	1.03	0.82
Int. Delay (s)	157.9	154.6	73.9	60.4
Int. LOS	F	F	E	E
ICU (%)	67.6	67.6	49.8	43.6
Offsets	133	133	133	133

Source: from study

Geometric improvements were done as a correctional measure. With the addition of one lane on the major approach through put traffic, the v/c ratio reduced from 1.48 to 1.03 with a corresponding reduction in delay as seen in Table 23. The level of service also changed from F to E meaning that there had been an improvement at the intersection. When 2 lanes each were added to each major through put approach, the v/c ratio again reduced from 1.03 to 0.82 with a corresponding reduction in the delay. The level of service E could not be described by speed alone, but represented operations at lower operating speeds, typically, but not always, in the neighborhood of 30miles per hour, with volumes at or near the capacity of the highway. Flow was unstable, and there could be stoppages of momentary duration. This level of service was associated with operation of a facility at capacity flows. It represented intermediate conditions and performance had improved slightly at the intersection. Similarly, the existing reservation was further checked against the geometric improvement.

3.4.3 Grade Separation Option

With the continuous addition of lanes to the through put traffic on the major approaches at Anloga intersection, a situation may arise where there will not be enough space to contain the addition of lanes. Since the intersections will still be operating at a level of service E, there is therefore the need to provide a facility that can accommodate the traffic congestion at the intersection on a long term basis. This then calls for a grade separation (interchange) at the intersection to allow the continuous flow of traffic.

3.4.4 Sensitivity Analysis

This was carried out to verify in detail the sensitivity of the obtained results to the variation of the input parameters at the selected intersections.

When the mean input parameters values from Table 5 such as speed and headway were substituted in the modeled equations or obtained results as shown in Table 24, the following saturation flow values were obtained for the field and simulated conditions from and to KNUST approaches at the selected intersections.

Table 24: Sensitivity Analysis of Saturation Flows for the Selected Intersections

Intersection	Conditions	Approach	Obtained Results	Speed	Headway	Saturation Flow
Anloga	Field	From KNUST	$q = -18.644u - 463.630h + 3362.416$	26.63	2.59	1665
		To KNUST	$q = -17.815u - 392.412h + 3252.782$	28.43	2.56	1742
	Simulated	From KNUST	$q = \left(1.98 \times 10^{-15}\right)u + 100h + 1373$	26.63	2.59	1632
		To KNUST	$q = \left(3.94 \times 10^{-14}\right)u + 100h + 1459$	28.43	2.56	1715
Stadium	Field	From KNUST	$q = -10.446u - 242.446h + 2404.311$	26.63	2.59	1498
		To KNUST	$q = -5.109u - 528.143h + 3057.240$	28.43	2.56	1560
	Simulated	From KNUST	$q = \left(-4.3 \times 10^{-14}\right)u + 100h + 949$	26.63	2.59	1208
		To KNUST	$q = \left(1.47 \times 10^{-14}\right)u + 100h + 1337$	28.43	2.56	1971
Amakom	Field	From KNUST	$q = 6.586u - 476.44h + 2741.454$	26.63	2.59	1683
		To KNUST	$q = -24.841u - 333.976h + 3073.799$	28.43	2.56	1513
	Simulated	From KNUST	$q = \left(1.27 \times 10^{-14}\right)u + 100h + 1199$	26.63	2.59	1458
		To KNUST	$q = 0.27u + 101.318h + 983.996$	28.43	2.56	1251

Source: from study

From the sensitivity analysis, it was deduced from Table 24 that there was generally no variation between the saturation flow values of the field and simulated conditions from and to KNUST approaches at the selected intersections. For Anloga intersection, 98.7% indicated that there was no variation between the saturation flow values of the field and simulated conditions from the KNUST approach with 1.3% indicating that there was variation between the saturation flow values of the field and simulated conditions from KNUST approach. Similarly, 98.4% indicated that there was variation between the saturation flow values of the field and simulated conditions to KNUST approach with 1.6% indicating that there was variation between the saturation flow values of the field and simulated conditions to KNUST approach.

For Stadium intersection, 80.4% indicated that there was no variation between the saturation flow values of the field and simulated conditions from the KNUST approach with 19.4% indicating that there was slight variation between the saturation flow values of the field and simulated conditions from KNUST approach. Similarly, 79.1% indicated that there was no variation between the saturation flow values of the field and simulated conditions to KNUST approach with 20.9% indicating that there was slight variation between the saturation flow values of the field and simulated conditions to KNUST approach.

For Amakom intersection, 86.3% indicated that there was no variation between the saturation flow values of the field and simulated conditions from the KNUST approach with 13.7% indicating that there was slight variation between the saturation flow values of the field and simulated conditions from KNUST approach. Similarly, 82.7% indicated that there was no variation between the saturation flow values of the field and simulated conditions to KNUST approach with 17.3% indicating that there was slight variation between the saturation flow values of the field and simulated conditions to KNUST approach.

The variations in the saturation flows for field and simulated conditions was attributed to the fact that headway made the strongest contribution in explaining the saturation flow when the variance in the model was controlled for as compared to that of speed which made no contribution to the model. The sensitivity analysis further confirmed that the obtained results from the simulation model were as good as the model replicated the specific real world characteristics of interest. Generally, the obtained results showed no variations to the input parameters from the sensitivity analysis from and to KNUST approaches and that the obtained results could be validated at intersections with similar Traffic volume characteristics, roadway geometry and signal timing.

IV. CONCLUSION

The Chi square test and t-test analyses revealed that headway had a strong correlation with saturation flow for both field and simulated conditions at 5% significance level. Again changes in phasing plan without geometric improvement at the selected intersections did not improve upon the overall intersection’s level of service. Furthermore changes in phasing plan with geometric improvement enhanced the intersection’s level of service. The existing controls needed to be replaced with a more effective, efficient and reliable control scheme to ensure smooth and safe operations. Stadium and Amakom signalized intersections should be coordinated to allow as many vehicles as possible to traverse those intersections without any delay. This will reduce travel time of motorists and also reduce congestion to the barest minimum. An interchange should be constructed at the Anloga intersection to allow free movement of vehicles thereby minimizing congestion and accident occurrences.The obtained results showed no variations to the input parameters from the sensitivity analysis from and to KNUST approaches and that, the obtained results could be validated at intersections with similar Traffic volume characteristics, roadway geometry and signal timing. Obtained results from micro simulation models can help the traffic engineer to comprehend the problems existing and design improvement plans at the intersection to reduce frequent delays and queue spillbacks which will consequently improve upon the levels of service at the selected intersections. Obtained results could be used in forecasting the future traffic conditions based on the present. Further research needs to be investigated on the effect of left turning traffic at Stadium intersection.

V. ACKNOWLEDGEMENT

The authors would like to acknowledge the management of Kumasi Polytechnic, Kumasi headed by the Rector Prof. N.N.N. Nsowah-Nuamah, for providing financial assistance and also Department of Urban Roads (DUR), Kumasi for giving information on signalized intersections in the Kumasi Metropolis. Several supports from staff of the Civil Engineering Department, KNUST and Kumasi Polytechnic, Kumasi are well appreciated.

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