PERFORMANCE EVALUATION OF SELECTED SIGNALIZED INTERSECTIONS ON THE 24TH FEB. ROAD, KUMASI USING MICRO SIMULATION MODEL

By

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College of Engineering

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CERTIFICATION/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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Head of Department

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DEDICATION

This piece of work is dedicated to

Miss Adelaide Neequaye (*my mum*)

and

Paa Yaw Asante Nyantakyi and Emile Owuraku Nyantakyi (*my sons*)

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ABSTRACT

Signalized intersections are a critical element of an urban road transportation system. 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. The need for simulation programs has become more important in cities where rapid growth takes place such as Kumasi, which has more than 40 pre-timed signalized intersections with various peak hour times during the day. Therefore, the need for calibrating simulation models to local studies is urgent in order to minimize error between modeled results and the real life.

The main objectives of this research are to assess performance measures at selected intersections using micro simulation models in Synchro/SimTraffic; to investigate the optimal cycle lengths, splits and offsets for the selected signalized intersections on the corridor, and to recommend measures to improve upon the performance of the selected signalized intersections under isolated or coordinated manner

To achieve these objectives, field studies were undertaken to collect traffic data, network and geometric data and signal control data.

Chi square test analysis was done to determine the level of significance between the computed and adjusted saturation flow rates, speeds and headways for the selected intersections.

Regression analysis was carried out to establish which of either speed or headway was a better predictor or had a strong correlation with saturation flow rates.

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The calibration parameters were accepted and used for further analysis and modeling because the differences between the computed and adjusted calibration parameters are not significant at 5% level of significance.

From the modeling undertaken using the calibrated Synchro/SimTraffic, the following conclusions were made: changes in phasing plan without geometric improvement at the selected intersections only improved upon the delay; changes in phasing plan with geometric improvement improved upon the intersection's level of service.

The following recommendations were proposed: Stadium and Amakom signalized intersections should be coordinated to allow as many vehicles as possible to traverse those intersections without any delay as well as an interchange should be constructed at the Anloga intersection to allow free movement of vehicles thereby minimizing congestion and accident occurrences.

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CHAPTER 1

1.0 INTRODUCTION

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 Junction and Amakom signalized intersections are three major intersections on 24th February Road, 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.

Simulation of traffic as a tool for investigating traffic systems has increased in popularity over the last decades. A large portion of this rise in popularity can be traced back to the rapid development in the personal computer area. Fast personal computers have made it possible to develop advanced traffic micro-simulation software packages. Traffic simulation is a powerful and cost-efficient tool for traffic planning and designing, testing different alternatives and evaluating traffic management schemes. The simulation model enables the engineer to predict the outcomes of a proposed change to the traffic system before it is implemented, and to evaluate the merits of competing designs. For the simulation model to correctly predict the system response however, it must first be shown to reproduce the existing traffic condition. The procedure by which the parameters of the model are adjusted so that the simulated response agrees with the measured field conditions is what is known as model calibration.

1.1 Background

Transportation systems are an integral part of a modern day society designed to provide efficient and economical movement between the component parts of the system and offer maximum possible mobility to all elements of our society. A competitive, growing economy requires a transportation system that can move people, goods, and services quickly and effectively. Road transportation is a critical link between all the other modes of transportation and proper functioning of road transportation, both by itself and as a part of a larger interconnected system, ensures a better performance of the transportation system as a whole.

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Signalized intersections as critical elements of an urban road transportation systems regulate the flow of vehicles through urban areas. Traffic flows through signalized intersections are filtered by the signal system causing vehicular delays. Vehicular delay at signalized intersections increases the total travel time through an urban road network, resulting in a reduction in the speed to say 10km/hr, reliability, and cost-effectiveness of the transportation system. Increase in delay results in the degradation of the environment through increases in air and sound pollution. Thus, delay can be perceived as an obstacle that has a detrimental effect on the economy and it has been the traffic engineers endeavor to optimize the signal system to perform at a minimum delay.

In recent years, population in cities, vehicle ownership and traffic volume in links increase dramatically due to the continuous high speed growth of economy, which cause traffic congestion of different levels in most cities. Under this circumstance, velocity of vehicles drops largely and in some cities, the velocity during peak hour is even lower than 10 km/h. All of these already influence the normal performance of urban function, hamper the continuous and steady growth of economy and affect residents' daily lives. Unfortunately, even though great efforts are made, the situation of traffic congestion becomes worse and worse. Thus, it is significant to evaluate the performance of the selected signalized intersections on the 24th February Road, in order to improve upon their capacities.

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1.2 Problem Statement

There are four signalized intersections on the 24th February road. Three of them are selected for study (Anloga, Stadium and Amakom). These intersections are characterized by long queues on the approaches especially during morning and evening peak periods of the day. However, for some time now observation of the 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.

Previous studies by BCEOM and ACON (2004) which evaluated the performance of these intersections on the 24th February road found that the major causes of congestion were attributed to critical capacity, intersection controls and abuse by motorists and/or pedestrians. They also concluded that traffic on most of the sections on the 24th February road is approaching (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. They therefore recommended that these sections of the 24th February road will be highly critical in the next 3 to 5 years (period from 2004 – 2009)

The study proposed measures to improve upon the signalization and capacity at the selected intersections through; Revision of signal timing, phasing plan, lane assignment/designation and inclusion of exclusive NMT 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, it is not clear how the models in it were calibrated or whether it was done at all. Even though some of the recommendations have been implemented at the intersections, long queues and frequent delays still persist during peak hour conditions. 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 practicing engineers. This study seeks to contribute to the knowledge in this area by calibrating the Synchro/SimTraffic models and using them to undertake intersection design and modeling the performance of the selected intersections.

1.3 Research Objectives

The main objectives of this study are as follows:

1) To assess performance measures at selected intersections using micro simulation models in Synchro/SimTraffic.
2) To investigate the optimal cycle lengths, splits and offsets for the selected signalized intersections on the corridor.
3) To recommend measures to improve upon the performance of the selected signalized intersections under isolated or co-ordinated manner.

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1.4 Justification

The justification and relevance of the study is based on the following:

- Outputs or results from micro simulation models can help the traffic engineer know the problems existing at the intersections and also come out with improvement plans at the intersections. This will go a long way in reducing frequent delays and queue spillbacks and improve upon the levels of service at the selected intersections on the 24th February Road.
- The need to use simulation tools in evaluating the performance of intersections in the road sector in Ghana. 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.

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CHAPTER 2

2.0 LITERATURE REVIEW

2.1 History of Signalized Intersections

Signalized intersections are vital nodal points in a transportation network and their efficiency of operation greatly influences the entire network's performance. Traffic signals are installed at these nodal points in order to allocate the right-of-way to the different competing streams of vehicles passing through the intersection.

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Pretimed signals, which are the most common type in use, typically rotate through preset signal timing patterns, which are determined in such a way as to optimize the intersection's level of service. The level of service at a signalized intersection can be assessed based on various criteria. However, delay incurred by vehicles is traditionally the most important criterion because its meaning is generally well understood by the driving population. Various models have been developed to estimate the delay incurred by vehicles at traffic signalized intersections. However, most of the previous research efforts have focused on fairly homogeneous traffic arrivals at signalized intersections. (HCM, 2000)

Unfortunately, non-homogeneous traffic is a more typical occurrence than homogeneous traffic at urban intersections. At many intersections, vehicles with wide-ranging static and dynamic characteristics can indeed be observed. Moreover, the lateral and longitudinal placements of vehicles on the carriageway are complex, with no discernible lane discipline. To further complicate matters, these non-homogeneous traffic behave differently with some of the traffic stream stopping at bus stops as is the case with buses. (HCM, 2000)

Simulation is defined as dynamic representation of some part of the real world achieved by building a computer model and moving it through time (Drew, 1968). 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 (Kallberg, 1971). 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 five driving forces behind this development are the advances in traffic theory, in computer hardware technology and in programming tools, the development of the general information infrastructure, and the society's demand for more detailed analysis of the consequences of traffic measures and plans. The basic application areas of simulation have mainly remained the same, but the applications have grown in size and complexity. In the 1990's demand analysis through simulation has emerged as a new application area. It was tried to give an overall view of the development, present use and future directions of simulation in road traffic planning and research.

2.2 Traffic Operation Elements

Signalized intersection's operations are a function of three elements described in the following sections along with a discussion on their effect on operations (*HCM 2000*).

- Traffic volume characteristics.

- Roadway geometry.
- Signal timing.

2.2.1 Traffic Volume Characteristics

The traffic characteristics used in an analysis can play a critical role in determining intersection treatments. Over conservative judgment may result in economic inefficiencies due to the construction of unnecessary treatments, while the failure to account for certain conditions (such as a peak recreational season) may result in facilities that are inadequate and experience failing conditions during certain periods of the year.

According to *HCM 2000*, an important element of developing an appropriate traffic profile is distinguishing between traffic demand and traffic volume. For an intersection, traffic demand represents the arrival pattern of vehicles, while traffic volume is generally measured based on vehicles' departure rate. For the case of overcapacity or constrained situations, the traffic volume may not reflect the true demand on an intersection. In these cases, the user should develop a demand profile. This can be achieved by measuring vehicle arrivals upstream of the overcapacity or constrained approach. The difference between arrivals and departures represents the vehicle demand that does not get served by the traffic signal. This volume should be accounted for in the traffic operations analysis.

Traffic volume at an intersection may also be less than the traffic demand due to an overcapacity condition at an upstream or downstream signal. When this occurs, the upstream or downstream facilities "starve" demand at the subject intersection. This effect is often best accounted for using a micro simulation analysis tool.

2.2.2 Intersection Geometry

The geometric features of an intersection influence the service volume or amount of traffic an intersection can process. A key measure used to establish the supply of an intersection is saturation flow, which is similar to capacity in that it represents the number of vehicles that traverse a point per hour; however, saturation flow is reported assuming the traffic signal is green the entire hour. By knowing the saturation flow and signal timing for an intersection, one can calculate the capacity (capacity = saturation flow times the ratio of green time to cycle length). Saturation headway is determined by measuring the average time headway between vehicles that discharge from a standing queue at the start of green, beginning with the fourth vehicle. Saturation headway is expressed in time (seconds) per vehicle. (HCM, 2000)

The saturation flow rate, s, is an important parameter for estimating the performance of a particular movement. Saturation flow rate is simply determined by dividing the average saturation headway into the number of seconds in an hour, 3,600, to yield units of vehicles per hour. For example, vehicles departing from a queue with an average headway of 2.2 seconds have a saturation flow rate of 3600 / 2.2 = 1636 vehicles per hour per lane. Saturation flow rate for a lane group is a direct function of vehicle speed and separation distance. These are in turn functions of a variety of parameters, including the number and width of lanes, lane use (e.g., exclusive versus shared lane use, aggregated in the HCM as lane groups), grades, and factors that constrain vehicle movement such as presence or absence of conflicting vehicle and/or pedestrian traffic, on-street parking, and bus movements. As a result, saturation flow rates vary by movement, time, and location and commonly range from 1,500 to 2,000 passenger cars per hour per lane. (HCM, 2000)

The HCM provides a series of detailed techniques for estimating and measuring saturation flow rate. It should be noted that this is significantly different than the ideal saturation flow rate, which is typically assumed to be 1,900 passenger cars per hour per lane.

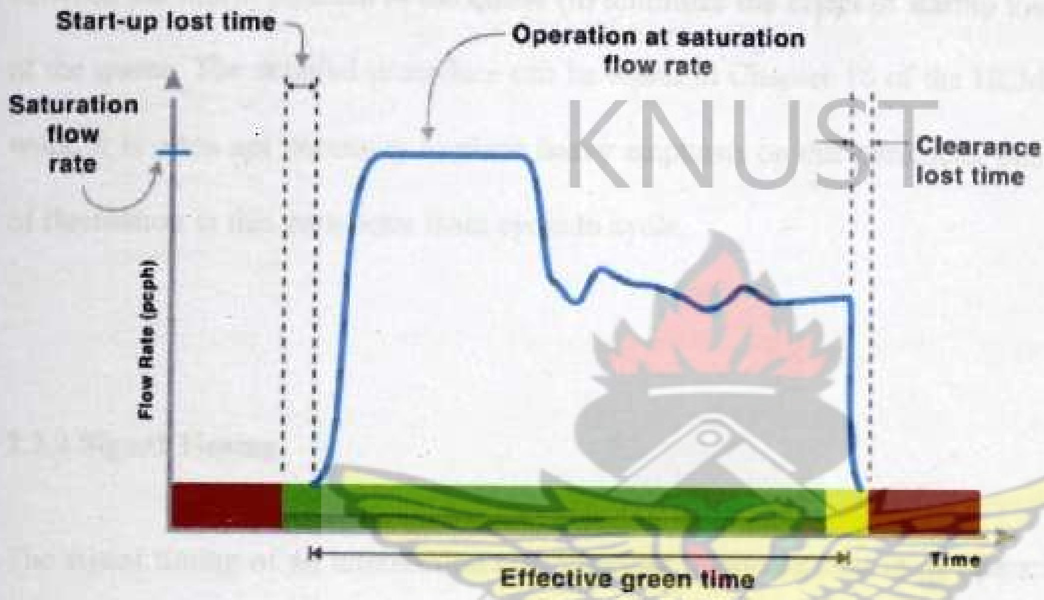

Figure 2.1: Typical flow rates at a signalized movement

The ideal saturation flow rate may not be achieved (observed) or sustained during each signal cycle. There are numerous situations where actual flow rates will not reach the average saturation flow rate on an approach including situations where demand is not able to reach the stop bar; queues are less than five vehicles in a lane, or during cycles with a high proportion of heavy vehicles. To achieve optimal efficiency and maximize vehicular throughput at the signalized intersection, traffic flow must be sustained at or near saturation flow rate on each approach. In most HCM analyses, the value of saturation flow rate is a constant based on the parameters input by the user, but in reality, this is a value that varies depending on the cycle by cycle variation of situations and users.

The HCM provides a standardized technique for measuring saturation flow rate. It is based on measuring the headway between vehicles departing from the stop bar, limited to those vehicles between the fourth position in the queue (to minimize the effect of startup lost time) and the end of the queue. The detailed procedure can be found in Chapter 16 of the HCM. For signal timing work, it is often not necessary to place heavy emphasis on this parameter due to the high degree of fluctuation in this parameter from cycle to cycle.

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2.2.3 Signal Timing

The signal timing of an intersection also plays an important role in its operational performance. Key factors include:

- *Effective green time*. Effective green time represents the amount of usable time available to serve vehicular movements during a phase of a cycle. It is equal to the displayed green time minus start up lost time plus end gain. The effective green time for each phase is generally determined based on the proportion of volume in the critical lane for that phase relative to the total critical volume of the intersection. If not enough green time is provided, vehicle queues will not be able to clear the intersection, and cycle failures will occur. If too much green time is provided, portions of the cycle will be unused resulting in inefficient operations and frustration for drivers on the adjacent approaches.

- *Clearance interval.* The clearance interval represents the amount of time needed for vehicles to safely clear the intersection and includes the yellow change and red clearance intervals. The capacity effect of the clearance interval is dependent upon the lost time.
- *Lost time.* Loss time represents the unused portion of a vehicle phase. Lost time occurs twice during a phase: at the beginning when vehicles are accelerating from a stopped position and at the end when vehicles decelerate in anticipation of the red indication. Longer lost times reduce the amount of effective green time available and thus reduce the capacity of the intersection. Wide intersections and intersections with skewed approaches or unusual geometrics typically experience greater loss times than conventional intersections.
- *Cycle length.* Cycle length determines how frequently during the hour each movement is served. It is either a direct input, in the case of pre-timed or coordinated signal systems running a common cycle length, or an output of vehicle actuations, minimum and maximum green settings, and clearance intervals. Cycle lengths that are too short do not provide adequate green time for all phases and result in cycle failures. Longer cycle lengths result in increased delay and queues for all users.
- *Progression.* Progression is the movement of vehicle platoons from one signalized intersection to the next. A well-progressed or well-coordinated system moves platoons of vehicles so that they arrive during the green phase of the downstream intersection. When this occurs, fewer vehicles arrive on red, and vehicle delay and queues are minimized. A poorly coordinated system moves platoons such that vehicles arrive on red, which increases the delay and queues for those movements beyond what would be experienced if random arrivals occurred.

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2.3 Traffic signal controls

The traffic light is a very important electronic device which is used in nearly every city in the country. The purpose of a traffic light system is to safely control the flow of traffic at street intersections. While the basic operation of each traffic light system is obviously similar, there are many different methods of implementation. The three main types of traffic light operation are pre-timed, semi-actuated, and full-actuated.

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2.3.1 *Pre-timed:* A signal whose timing is fixed over specified time periods and does not respond to changes in traffic flow at the various intersection approaches. No vehicle detection is necessary with this mode of operation.

2.3.2 *Semi-Actuated:* A signal, whose timing is affected when vehicles are detected by video or pavement-embedded inductance loop detectors on some, but not all, approaches. This mode of operation is usually found when a low-volume road intersects a high-volume road, often referred to as the minor and major streets, respectively. In such cases, green time is allocated to the major street until vehicles are detected on the minor street, then the green indication is briefly allocated to the minor street and then returned to the major street.

2.3.3 *Fully-Actuated:* A signal whose timing is completely influenced by the traffic volumes, when detected, on all of the approaches. Fully actuated signals are most commonly used at intersections of two major streets and where substantial variations exist in all approach traffic volumes over the course of a day.

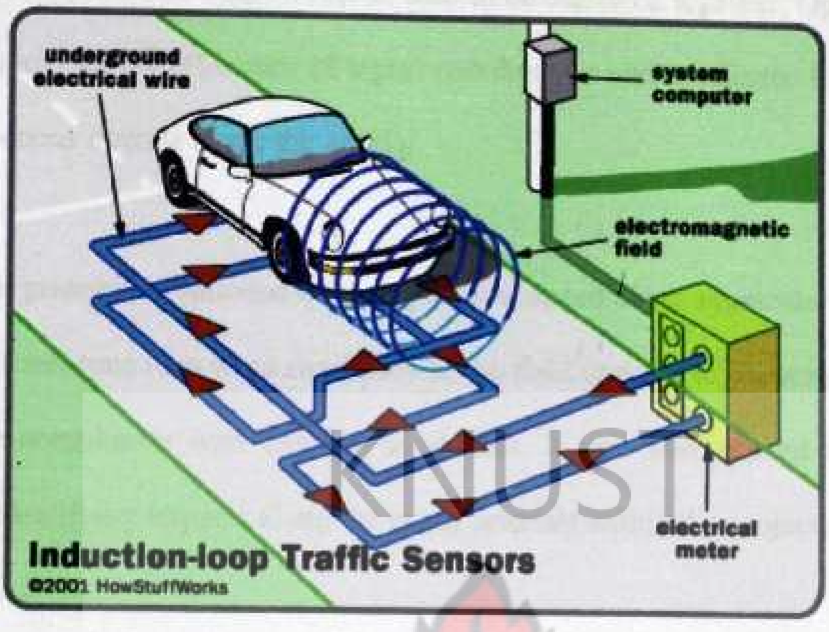

Figure 2.2.Traffic actuated control system

2.4 Measures of Effectiveness (MOE)

Measures of effectiveness are quantitative measures that summarize the operating performance of a traffic environment. In terms of the traffic network, these often include:

- Level of Service (LOS) and delay,
- Travel speed and time, and
- Queuing.

LOS is a qualitative description of the performance of an intersection based on the average delay per vehicle. Intersection levels of service range from LOS A, which indicates free flow or excellent conditions with short delays, to LOS F, which indicates congested or overloaded conditions with extremely long delays. Urban and suburban arterials are characterized by platoon flows or traffic flow operations where vehicles tend to be clustered together. Operational quality is controlled primarily by the efficiency of signal coordination and is affected by how individual signalized intersections operate along the arterial.

Level of service is primarily a function of average travel speed along segments, and is calibrated from field data. Travel time runs were conducted in the field in order to calibrate the Sim Traffic models to ensure compliance with existing conditions. It was also applied as a measure of effectiveness for identifying impacts along the major arterials within the project area.

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2.4.1 Level of Service (LOS)

Levels of Service for **signalized intersections** were calculated in Synchro using the *Highway Capacity Manual 2000* (HCM 2000) methodology. The LOS is based on the average delay (in seconds per vehicle) for the various movements within the intersection. A combined weighted average delay and LOS are presented for each of the signalized intersections. The average delay for signalized intersections was calculated using the Synchro analysis software and is correlated to the level of service designation as shown below in Table 2.1.

Table 2-1: Level of Service Criteria – Signalized Intersections

Level of Service	Description of Operations	Average Delay* (sec)
A	Operations with very low delay occurring with favorable progression and/or short cycle length.	≤ 10.0
B	Operations with low delay occurring with good progression and/or short cycle lengths.	10.1 – 20.0
C	Operations with average delays resulting from fair progression and/or longer cycle lengths. Individual cycle failures begin to appear.	20.1 – 35.0
D	Operations with longer delays due to a combination of unfavorable progression, long cycle lengths, or high V/C ratios. Many vehicles stop and individual cycle failures are noticeable.	35.1 – 55.0
E	Operations with high delay values indicating poor progression, long cycle lengths, and high V/C ratios. Individual cycle failures are frequent occurrences. This is considered to be the limit of acceptable delay.	55.1 – 80.0
F	Operation with delays unacceptable to most drivers occurring due to over saturation, poor progression, or very long cycle lengths.	≥ 80.1

* Delay presented in seconds per vehicle.
Source: Highway Capacity Manual, Transportation Research Board, 2000

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2.5 Traffic as a simulation object

Road transportation, that is, efficient movement of people and goods through physical road and street networks is a fascinating problem. Traffic systems are characterized by a number of features that make them hard to analyze, control and optimize. The systems often cover wide physical areas, the number of active participants is high, the goals and objectives of the participants are not necessarily parallel with each other or with those of the system operator (system optimum vs. user optimum), and there are many system inputs that are outside the control of the operator and the participants (the weather conditions, the number of users, etc.).

In addition, road and street transportation systems are inherently dynamic in nature, that is, the number of units in the system varies according to the time, and with a considerable amount of randomness. The great number of active participants at present at the same time in the system means a great number of simultaneous interactions. Transportation systems are typical man-machine systems, that is, the activities in the system include both human interaction (interaction between driver-vehicle-elements) and man-machine-interactions (driver interaction with the vehicle, with the traffic information and control system and with the physical road and street environment).

In addition, the laws of interaction are approximate in nature; the observations and reactions of drivers are governed by human perception and not by technology based sensor and monitoring systems. In all, traffic systems are an excellent application environment for simulation based research and planning techniques, an application area where the use of analytical tools, though very important, is limited to subsystem and sub-problem level. The reasons to use simulation in

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the field of traffic are the same as in all simulation; the problems in analytical solving of the question at hand, the need to test, evaluate and demonstrate a proposed course of action before implementation, to make research, to learn, and to train people.

2.6 Areas and approaches in Traffic Simulation

The applications of traffic simulation programs can be classified in several ways. Some basic classifications are microscopic, mesoscopic and macroscopic. Special areas are traffic safety and the effects of advanced traffic information and control systems. A newly emerged area is that of demand estimation through microscopic simulation. Recent advances in computer hardware and software technology have led to the increased use of traffic simulation models. Depending on the required objective of the simulation, models range from microscopic models, which detail the movement of individual vehicles, to macroscopic models that use gross traffic descriptors such as flow. Because of the fine level of detail required in a microscopic model, applications tend towards traffic operations over a relatively small geographical area. Macroscopic models are generally applied over a much larger, system-wide, geographical area and are more useful for transportation planning rather than traffic engineering (Roger and Sutti, 1986).

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 GM models. In these models a differential equation governs the movement of each vehicle in the platoon under analysis (Gerlough and Huber, 1975). Car-following, like the intersection analysis, is one of the basic questions of traffic flow theory and simulation, and still under active analysis after almost 40 years from the first trials

(McDonald *et al.* 1998). The traditional simulation problem with practical orientation in road and street traffic analysis is related to questions of traffic flow, that is, to capacity and operational characteristics of facilities. Delays and queue lengths at intersections are a never-ending object of analysis and simulation studies with a newly grown international interest in roundabouts.

In the area of traffic signal control, the classic Webster's formula (Webster and Cobbe, 1966) is an example of early use of simulation with practical results. In this formula a simulation-based correction is added to an analytical delay formula derived by the use of queuing theory. In traditional fixed time signal control only the traffic was reacting to signals, now the signals are also reacting to traffic, and the analysis of controller reactions is quite as important as the analysis of traffic itself. New solutions, like the connection of a real controller to the simulation system (Kosonen and Pursula, 1991) are used in the analysis.

Most traffic system simulation applications today are based on the simulation of vehicle-vehicle interactions and are microscopic in nature. Traffic flow analysis is one of the few areas, where macroscopic (or continuous flow) simulation has also been in use.

2.7 Trends in traffic simulation

The development in traffic simulation from the early days in the 1950's and 1960's has been tremendous. This, of course, is partly related to the development of computer technology and programming tools. On the other hand, the research in traffic and transportation engineering has also advanced during this 40-year period. Simulation is now an everyday tool for practitioners and researchers in all fields of the profession. In the following, some of the development trends in sight are shortly discussed. Most of these trends are related to microscopic simulation.

It is, however, noteworthy that there are some quite interesting new developments in the theoretical macroscopic models for fundamental traffic flow analysis, which give new insight to the fundamental speed flow-density relationships (Helbing *et al*. 1997). The applications are growing in size, that is, we are moving from the quite well covered local or one facility type applications to network wide systems where several types of facilities are integrated in one system. Another trend that increases the need of computing power is the more and more precise description of the physical road and street environment, especially in local applications, like in simulation of intersections.

2.8 Adaptation of the SYNCHRO Model

The intersections will be selected after which the required input data needed by Synchro (geometric data, traffic volume data, signal control data) will be collected. In order to calibrate or adapt the Synchro to the local conditions in Ghana, the following three parameters (saturation flow, average speeds and headway) at the selected intersections will then be collected. Results from the simulation model are then compared with the field measured parameters. After that the individual components of the simulated model will then be refined and adjusted so that the simulated model parameters accurately represent the field measured parameters at the selected intersections.

2.9 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.

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2.9.1 Generalized General Motors Models

The first GM model modeled car-following as 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:

$$\ddot{\chi}_F = \alpha_F \times \left(\dot{\chi}_L(t) - \dot{\chi}_F(t) \right)$$

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

$\dot{\chi}_F(t)$ = Speed of the following vehicle,

$\dot{\chi}_L(t)$ = Speed of the leading vehicle,

α_F = sensitivity of the following vehicle, and

t = time.

2.9.2 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.

$$h_s(t) = L + m + kV_2 + bk[V_1(t) - V_2(T)]^2$$

where: $h_s(t)$ = 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.

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

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.

k = car-following sensitivity factor

2.10 Delay equations developed by Webster

Signalized intersections were developed in England in the early 20th century. With the introduction of these controls to maneuver conflicting streams of vehicular and passenger traffic, researchers have concentrated on estimating delays due to these controls and in developing the optimum signal timings to minimize delay especially for pre-timed signals. Webster's equation is one of the foremost delay equations developed in 1958 assuming practical distributions like Poisson (random) arrivals with uniform discharge headways. Webster introduced three terms to the delay equation as shown below.

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$$d = \frac{c(1-\lambda)^2}{2(1-\lambda x)} + \frac{x^2}{2q(1-x)} - 0.65\left(\frac{c}{q}\right)^{\frac{1}{2}} x^{(2+5\lambda)}$$

where,

d is the average delay per vehicle,

c is the cycle time,

λ is the ratio of the effective green to the cycle length,

q is the flow rate,

x is the degree of saturation.

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The first term in equation above represents the delay when traffic is considered to arrive at a uniform rate. The second term is a correction to consider the random nature of the arrivals. The third term is the empirical correction term introduced to give a closer fit to the simulated delay values. Furthermore, Webster used differential calculus techniques on the developed delay estimate to compute the cycle length for the minimum average delay.

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2.11 Platoon Dispersion Model

The on-off nature of traffic signal tends to create bunches or platoons of vehicles. The platoons of vehicles disperse as they travel away from the lights due to the different speeds of the individual vehicles. Thus the arrival pattern at an intersection downstream from another signal is different from an isolated intersection. Robertson developed the platoon dispersion model for the Road Research Laboratory in United Kingdom in 1969. The dispersion model was developed based on the observations made at four sites in West London at approximately 300, 600 and 1000 ft downstream of the stop bar.

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The predicted flow rate at any time step is expressed as a linear combination of the original platoon flow rate in the corresponding time step (with a lag of t) and the flow rate of the predicted platoon in the step immediately preceding it. A smoothing factor F is used in the model to best fit the actual and calculated platoon shapes and is inversely proportional to the travel time on the link. The arrivals at the downstream intersection are estimated depending on the discharge patterns from upstream intersection. The smoothing factor is found to be site specific and depends on the road width, gradient, parking, opposing flow level, etc.

Rouphail developed a closed form solution for the recursive model developed by Robertson, and studied the effect of platoon dispersion on signal coordination and delay estimation. Flow rates in the predicted platoon measured at the k_{th} interval of the j_{th} simulated cycle are expressed in terms of the demand and capacity rates at the source intersection in addition to signal-control and travel-time parameters.

2.12 Methods of Measurements

Several methods have been proposed and used by authors to estimate capacity parameters, mainly the saturation flow rate and lost times. The methods are described in chronological order.

Headway Method (Greenshields et al. 1947; TRB 1997): estimates the average time headway between the vehicles discharging from a queue as they pass the stop-line. The first several vehicles are skipped to avoid the effect of vehicles' inertia in the initial seconds of the green time. The saturation flow rate is calculated as a reciprocal of the mean headway.

TRL Method (TRRL 1963): vehicles are counted during three saturated intervals of green. The saturation flow rate is calculated dividing the count of the middle interval over the length of the interval itself.

Regression Technique (Branston and Gipps 1981; Kimber et al. 1985; Stoke et al.1987): used to develop an equation involving the saturated green time, number of vehicles in various categories, and lost time. A regression analysis yields the saturation flow, the lost times, and the passenger car equivalents for vehicles other than passenger cars. Several studies have suggested the use of a full-motion video recording to collect data, which would provide an accurate record of the data. In addition, the impact of special conditions can be considered.

For example, in a certain signal cycle, when the discharge of the vehicles is impacted by a vehicle not moving for a long period of time, that data can be isolated and analyzed separately. It also allows for real-time analysis of the data. Also, any unusual event that may affect the saturation flow rate, such as buses, stalled vehicles, and unloading trucks can be identified.

2.13 Optimization Procedure Utilized by Simulation

One of the primary objectives of this research is to investigate optimal cycle lengths, splits and offsets for the selected signalized intersection. Various simulation programs and optimization techniques have evolved that aid the traffic engineer in the optimization process. Delay and its derivative are used as the objective function in most optimization softwares. For example, SYNCHRO optimizes based on the percentile delay.

SYNCHRO, developed by Trafficware Inc., is a software package that can model and optimize traffic signal timings. SYNCHRO minimizes a parameter called percentile delay in its optimization. The Percentile Delay is the weighted average of a delay corresponding to the 10th, 30th, 50th, 70th and 90th percentile volumes. SYNCHRO accommodates for progression by calculating the progression factor (PF) used in the delay equation using the ratio of uniform delay calculated by SYNCHRO with coordination and uniform delay calculated by SYNCHRO assuming random arrivals.

2.14 Calibration Procedure

Calibration is the process by which the individual components of a simulation model are refined and adjusted so that the simulation model accurately represents field measured and observed traffic conditions.

Sacks et al. (2002) identified four key issues on model validation: (1) identifying explicit meaning of validation in particular context, (2) acquiring relevant data, (3) quantifying uncertainties, and (4) predicting performance measures under new conditions. They demonstrated an informal validation process using CORSIM simulation model and emphasized the importance of data quality and visualization. However, the authors have not established any formal procedure for simulation model calibration and validation.

Hellinga (2003) proposed general requirements for the calibration of traffic simulation models. The proposed calibration process consists of three main phases: study definition, initial calibration, and evaluation of model outputs. The first step involves the tasks and activities prior to the modeling, such as identification of study goal, required field data, desired simulation performance measures, and so forth. The second step is to make sure that network coding is as accurate as possible. The third step is to run the simulation and compare the field data with simulation output. This process provides basic guidelines but does not give a direct procedure for conducting calibration and validation.

A ~~set of~~ guidelines for the calibration and validation of traffic simulation models was recommended by Milam et al. (2003). The guidelines include conduction of field-data collection,

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calibration of models to match field conditions, validation of models under certain criterion, such as 95 to 105 percent of observed value, and estimation of the minimum number of simulations with desired confidence interval. In addition, simulation outputs from CORSIM such as traffic counts, travel time, and queue were illustrated in the validation procedure. These guidelines could be viewed as a critical point for the application of traffic simulation models.

Park and Schneeberger (2003) proposed a calibration procedure consisting of nine steps and demonstrated the proposed procedure through a case study. The main components include (1) preparation before formal calibration such as data collection and selection of parameters and MOEs; (2) calibration effort such as experimental design, simulation runs, surface function development and candidate parameter sets generations; and (3) evaluation and validation. The case study results showed that the proposed procedure was effective in the calibration and validation for VISSIM mainly for signalized intersections. The study only made use of a single day of data collection and generated the parameter sets based on a linear regression model, which did not account for the correlations among the parameters.

A description was given for the application of calibration and validation techniques to traffic simulation models by Cohen (2004). The author recommended adjusting the major parameters close to the field observations during the calibration process and comparing the model performance measures with field data during the validation process. The MOEs used for validation should be independent of the measurement of the calibration parameters. In particular, Cohen asserted that calibration must be performed either by measuring the calibration parameters directly or by measuring a surrogate. Finally, the paper provided guidance to the model users on how one should proceed if the validation of the models fails. The procedure only applies to a few major parameters, which requires the knowledge and judgment of the model users. In addition, some parameters, especially those related to driver behavior, are not easily observed in the field.

Dowling et al. (2004) proposed a practical, top-down approach that consisted of three steps. First, capacity at the key bottlenecks in the system was calibrated. Second, traffic flow at non-bottleneck locations was calibrated. Finally, the overall model performance was calibrated against field performance measures. The authors divided the model parameters into categories and started with the most important parameters, usually global parameters. Then further fine tuning with link-specific parameters was conducted if necessary. However, the procedure also focuses on a few selected key parameters, which are not easy to identify. As this approach calibrates model parameters one by one, the result may be trapped into a local optimal.

A systematic validation approach of a microscopic simulation model was described by Zhang et al. (2004). The procedure includes animation comparison and quantitative/statistical analysis at both macroscopic and microscopic levels. Data at the macroscopic level include the averages and other statistics of traffic variables and fundamental relationships of traffic flow parameters. Data at the microscopic level include the speed change pattern, vehicle trajectory plot, and headway distributions. Animation comparison was supplemented to examine the model validity. The procedure emphasized the importance of real-world datasets to model validation. However, the datasets used in this study were truly microscopic and were very expensive to obtain.

CHAPTER 3

3.0 METHODOLOGY

3.1 Calibration of Synchro for Site Conditions

According to Carole Turley (2007), calibration procedure employed for the site is displayed in the flowchart below in Figure 3.1.

Figure 3.1: Flowchart of Calibration procedure

3.2 Site Selection and Description

The intersections were selected based on their accident and safety records in the past and also the levels of congestion associated with these intersections. The intersections were also selected for easy coordination.

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Figure 3.2: Map of Kumasi showing the selected intersections on the 24[th] February Road

Anloga Intersection

The Anloga junction is a signalised intersection comprising three (3) 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 (4) legs with the following configuration:

- East/West approaches - 24th February road, having two (2) approaches through and exit lanes
- North-East approach - Okomfo Anokye road, having one (1) approach through lane and two (2) exit lanes
- South-East approach – Undeveloped Eastern By-Pass, having one (1) approach/exit lanes

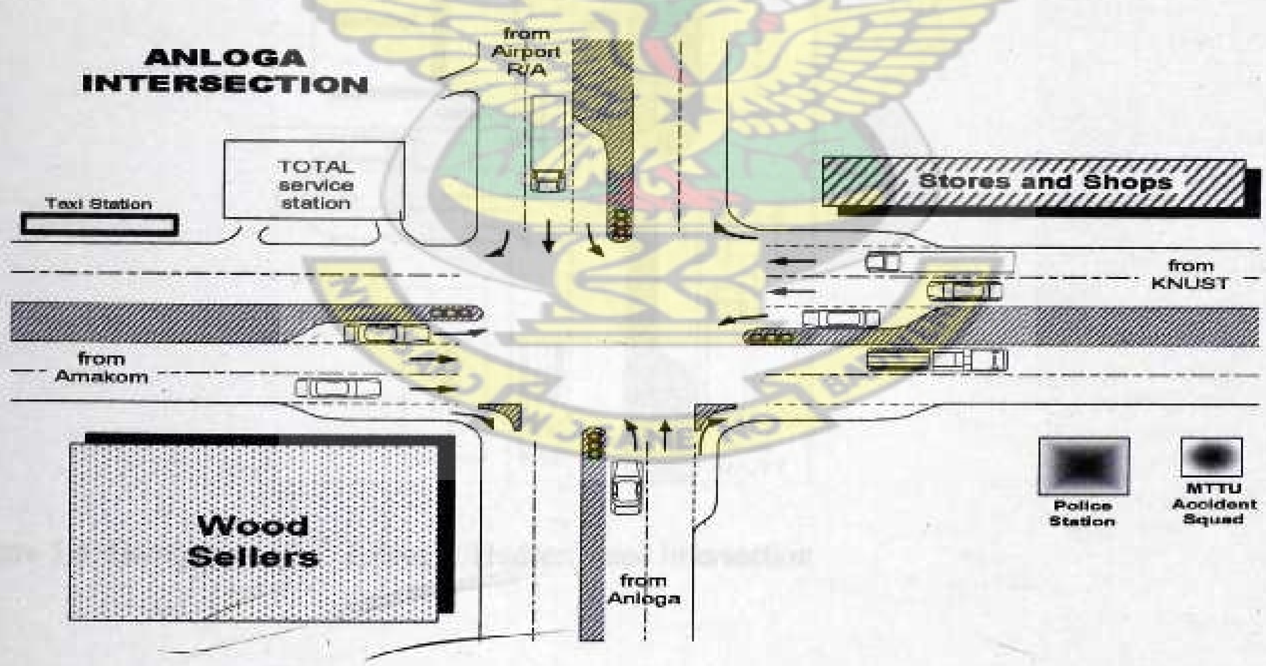

Figure 3.3: Geometry of Anloga Intersection

Stadium Junction Intersection

The intersection with Hudson road is signalised and it is 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 (3) legs with one (1) approach/entry lane and (1) exit lane on the minor road, (Hudson road), and two (2) approach/entry and exit lanes on the 24th February road. There is (1) exclusive left turn lane from Anloga intersection towards stadium. It is the intersection of a Principal arterial and a Minor arterial:

- 24th February Road – Principal Arterial, and
- Hudson road – Minor Arterial

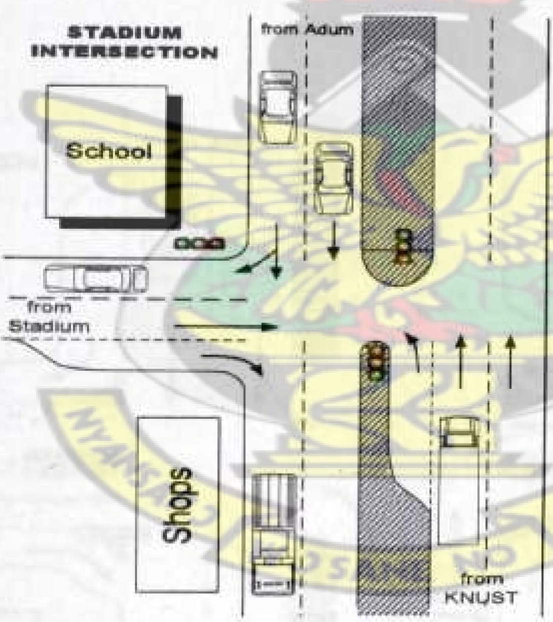

Figure 3.4: Geometry of 24th February/Hudson Road Intersection

Amakom Intersection

The Amakom traffic light, formerly called the Amakom roundabout, is a signalised intersection. It is about 4 km West of the KNUST junction. The intersection is 350m away from Stadium Junction traffic light and 700m away from Asafo roundabout. The intersection has four (4) legs with one (1) approach/entry and exit through lanes on each leg of the minor roads, (Yaa Asantewaa road), and two (2) approach/entry and exit through lanes on the 24th February road. It is the intersection of a Principal arterial and a Collector road:

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- 24th February Road – Principal Arterial, and
- Yaa Asantewaa road – Collector Road

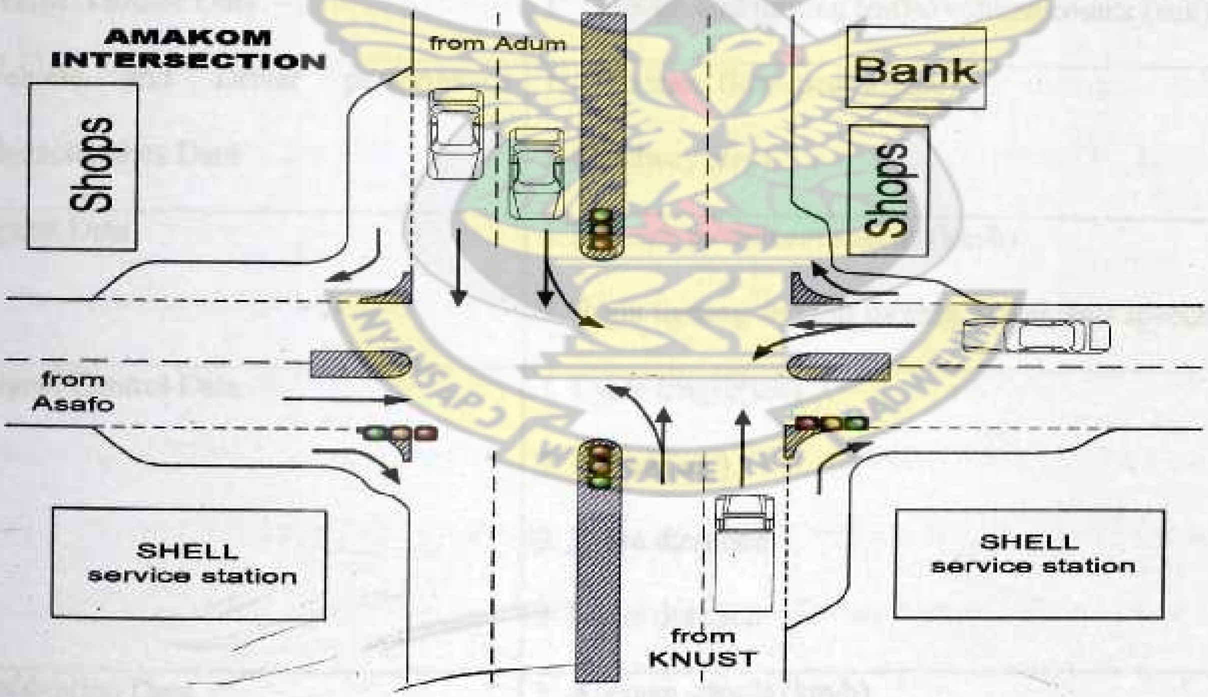

Figure ~~3.5.~~ Geometry of 24th February/ Yaa Asantewaa Road Intersection

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3.3 Data Collection for Synchro

Microscopic simulation model Synchro has complicated data input requirements and have many model parameters. In order to build a Synchro simulation model for the selected intersections and to calibrate it for the local traffic conditions, three important parameters are required. These are saturation flow rates, headway and speed.

Table 3.1: Summary of Data Collected at Selected Signalized Intersections

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Major Category	Data Type
Geometric Data	1. Number of lanes 2. Lane widths (m) 3. Lane storage lengths for turning movements (m)
Traffic Volume Data	1. Through and turning traffic volume counts (veh)
Vehicle and Driver performance characteristics Data	1. Saturation flow (pcu/hr/lane) 2. Headway (sec)
Speed Data	1.Through movement speeds (km/h) 2. Right turning and left turning movements speeds
Signal Control Data	1. Cycle length(sec) 2. Offsets(sec) 3. Phase direction 4. Phase duration
Calibration Data	1. Average speeds (km/h) 2. Headways (sec) 3. Saturation flow rates (pcu/hr/lane)

3.3.1 Videotaping Traffic at Anloga

Anloga intersection was filmed because of the possibility of obtaining good elevated 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 2009. 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, queues and signal heads. The headway values were taken manually with the help of stop watches.

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 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. .

Videotapes were played back and needed information such traffic volumes and headways were extracted. It was extremely important to follow the procedure carefully to minimize measurement errors. First, the vehicles were 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 capacity parameters like saturation flow rates.

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3.3.2 Manual Collection of Data at Stadium and Amakom Intersections

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

Traffic volume data such as through and turning traffic volume counts and geometric data such as number of lanes, lane widths, lane storage lengths for turning movements were collected manually at Stadium and Amakom intersections on Wednesday, the 13th of May 2009 between 0700hours and 1000 hours during the morning peak period of the day. Traffic signal timings were also determined manually using stop watches at these intersections. At Stadium junction and Amakom intersections, two enumerators each were positioned on each approach. They counted the number of vehicles turning left and right as well as vehicles going straight and recorded the times when an approach has green and red indications. Four other enumerators also recorded headways and speeds of vehicles as they traverse the two intersections.

3.3.3 Signal Control Data for the Selected Intersections

Signal control data consists of cycle lengths, phases, splits, and offsets. Cycle lengths, phases, offsets and splits for the selected intersections were recorded using stopwatch. Similarly, the actual green time (G), actual yellow time (Y) and all red time (R) were recorded with stop watches. Furthermore, the lost times (startup and clearance times) and effective green times were also recorded. Usually all the parameters are measured in seconds. Table 4.4 under results and discussion shows the signal timing information for the selected intersections.

3.4 Calibration Data for Selected Intersections

In order to realistically model traffic at the selected intersections, it was important to have realistic saturated flow rates, headways and speeds. In some cases it may be necessary to change the default parameters to match local driver parameters. To calibrate the model for local conditions, the following data were collected.

- Speeds within intersections
- Headways between intersections

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3.4.1 Spot Speed Data

Speed is the most important parameter describing the state of a given traffic stream. Speed is defined as the rate of motion, in distance per unit of time. 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 were collected from the two major approaches at Anloga, Stadium and Amakom intersections using the Doppler principle (i.e., radar).

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The speed data were collected as the tail of the vehicles crosses the stop bar. The first few vehicles in queue were not counted because they were accelerating. 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 from the two major 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 the two major approaches at Stadium junction and Amakom intersections were recorded for a maximum of 30 vehicles and 60 vehicles inclusive of private cars, commercial vehicles and heavy goods vehicles.

3.4.2 Headway and Saturation Flow Rates Data

Headway data for the selected intersections were collected with a stopwatch. The headway is the time starting when the tail of the lead vehicle crosses the stop bar until the front of the following car crosses the stop bar. Headway data were collected for three cycles each at the selected signalized intersections. Tables B1 – B3 in Appendix B show the headway data with their corresponding saturation flow rates collected for the KNUST and Amakom approaches at Anloga intersection, Anloga and Amakom approaches at Stadium intersection and Kejetia and Anloga approaches at Amakom intersection.

3.5 Analysis

3.5.1 Calibration of the Synchro Models

Chi Square Test Analysis

This was done to determine the level of significance between the computed and adjusted saturation flow rates, speeds and headways for the selected intersections. When ρ-values are less than 0.05 or very small, it was concluded that there existed significant differences between the computed and adjusted parameters. However, when the ρ-values are greater than 0.05, it was concluded that the differences between the computed and adjusted parameters are not significant.

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Paired Sample t-test Analysis

This analysis was carried out to evaluate the variations in the computed and adjusted saturation flow rates, speeds and headways for the selected intersections

Regression Analysis

In order to calibrate the model, a regression analysis was carried out from the two major approaches for the selected intersections. It was carried out to establish which of either speed or headway was a better predictor or had a strong correlation with saturation flow rates. It was again carried out to come out with an equation connecting saturation flow rates (q), headway (h) and speed (u).

3.5.2 Performance Assessment at Selected Intersections

In order to effectively evaluate the performance of the selected intersections, the following performance assessments were carried out:

Change of Phase without Geometric Improvement

In order to effectively investigate the optimized cycle lengths, splits and offsets for the selected signalized intersections on the corridor, three (3) different alternatives with four-phase operational plan were proposed at the Anloga intersection. 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.

Change of Phase with Geometric Improvement

The best alternatives chosen from the selected intersections were further investigated by the addition of lane(s) for through traffic at each of the selected intersections. This was done to see whether there were improvements in the level of services at the selected intersections. This is because improvement in the Level of Service (LOS) at any intersection results in an overall enhanced operating performance on the 24th February road corridor.

Grade Separation Option

Continuous addition of lanes to the through put traffic will adversely affect the reservation or right of way (ROW) at the selected intersections and when that condition persists, grade separation option will be considered to see how best to allow free flow of traffic to traverse the selected intersections concerned.

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CHAPTER 4

4.0 RESULTS AND DISCUSSION

4.1 Existing Traffic Volume and Geometric Data

Table 4.1: Summary Peak Hour Traffic Volume and Geometric Data at Anloga Intersection

Intersection: **Anloga Jn**

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)
KNUST	Anloga	WBL	7	50	1	3	75	0.5	1332	36.4	3,660
	Amakom	WBT	1013	3	2	3.4		76.1			
	Airport R/A	WBR	317	7	1	3	75	23.4			
Amakom	Airport R/A	EBL	291	12	1	3.4	86	18.8	1545	42.2	
	KNUST	EBT	1211	6	2	3.3		78.4			
	Anloga	EBR	43	15	1	4.7	54	2.8			
Anloga	Amakom	NEBL	141	9	1	4.8		63.5	222	6.1	
	Airport R/A	NEBT	30	40	1	4.8		13.5			
	KNUST	NEBR	51	21	1	4.7	34	23.0			
Airport R/A	KNUST	SWBL	144	16	1	3	65	25.7	561	15.3	
	Anloga	SWBT	20	60	1	3.8		3.6			
	Amakom	SWBR	397	10	1	3.8	58	70.7			

The intersection registered a peak hour volume of 3,660 vehicles. This value represents the worst traffic situation for an average day. The results of the motorized traffic volumes explain the breakdown of traffic during peak periods of the day. 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 of traffic.

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Table 4.2: Summary Peak Hour Traffic Volume and Geometric Data at Stadium Junction Intersection

Intersection: Stadium

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	Stadium	WBL	432	8	1	2.9	73	30.9	1399	57.5	2,434
	Amakom	WBT	967	1	2	3.5		69.1			
	N/A	WBR									
Amakom	N/A	EBL							846	34.8	
	Anloga	EBT	805	5	2	3.7		95.2			
	Stadium	EBR	41	13			Shared	4.8			
Stadium	Amakom	NBL	106	4	1	3.6		56.1	189	7.7	
	N/A	NBT									
	Anloga Junction	NBR	83	20	1	2.7	34	43.9			

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The intersection registered a peak hour volume of 2,434 vehicles. This value represents the worst traffic situation for an average day. The results of the motorized traffic volumes explain the breakdown of traffic during peak periods of the day. 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 of traffic.

Table 4.3: Summary Peak Hour Traffic Volume and Geometric Data at Amakom Intersection

Intersection: **Amakom**

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 Jn	Amakom	WBL	117	22	1	3.8	56	8.5	1383	46.4	2,980
	Kejetia	WBT	1153	3	2	2.9		83.4			
	Asawasi	WBR	113	24	1	4.5	43	8.1			
Kejetia	Asawasi	EBL	76	7	1	3.6	46	8.6	886	29.7	
	Anloga Junction	EBT	742	2	2	3.6		83.6			
	Amakom	EBR	69	0			54	7.8			
Amakom	Kejetia	NBL	53	9			shared	24.7	215	7.2	
	Asawasi	NBT	153	17	1	3.4		71.2			
	Anloga Junction	NBR	9	100			16	4.1			
Asawasi	Anloga Junction	SBL	120	13	1	4	23	24.2	496	16.7	
	Amakom	SBT	222	13	1	4		44.8			
	Kejetia	SBR	154	7	1	3	22	31.0			

The intersection registered a peak hour volume of 2,980 vehicles. This value represents the worst traffic situation for an average day. The results of the motorized traffic volumes explain the breakdown of traffic during peak periods of the day. 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 of traffic.

Table 4.4: Summary of Signal Timing Data for the Selected Intersections

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

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

$$g = G + Y + R - (l_1 + l_2)$$

Where l_1 = start up lost time

l_2 = clearance lost time

G = Actual green time

Y = Actual yellow time

R = Actual red time

4.2 Headways and Saturation Flow Rates Data

Tables 4.5 and 4.6 below show the summary of computed saturation flow rates and headways for Anloga, Stadium junction and Amakom intersections.

Table 4.5: Summary of Approach Saturation Flow Rates at the Selected Intersections

Intersections	Direction	Sample Size (n)	Mean (pcu/h/l)	Maximum (pcu/h/l)	Minimum (pcu/h/l)	Standard Deviation
Anloga	From Tech Jn	60	1662	4186	467	733.89
	To Tech Jn	60	1742	3913	403	712.85
Stadium	From Tech Jn	30	1226	2707	360	570.71
	To Tech Jn	30	1643	2727	678	511.65
Amakom	From Tech Jn	60	1489	3214	497	726.58
	To Tech Jn	60	1286	3186	256	582.51

Table 4.6: Summary of Approach Headways at the Selected Intersections

Intersections	Direction	Sample Size (n)	Mean (sec)	Maximum (sec)	Minimum (sec)	Standard Deviation
Anloga	From Tech Jn	60	2.59	7.71	0.86	1.29
	To Tech Jn	60	2.56	8.93	0.92	1.51
Stadium	From Tech Jn	30	3.7	10.0	1.33	1.99
	To Tech Jn	30	2.44	5.31	1.32	0.89
Amakom	From Tech Jn	60	3.01	7.24	1.12	1.42
	To Tech Jn	60	3.54	14.08	1.13	2.18

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4.3 Speed Data

Table 4.7 below shows the summary of computed speeds for Anloga, Stadium and Amakom intersections

Table 4.7: Summary of Approach Speeds for the Selected Intersections

Intersections	Direction	Sample Size (n)	Mean (km/hr)	Maximum (km/hr)	Minimum (km/hr)	Standard Deviation
Anloga	From Tech Jn	60	26.63	36	20	3.89
	To Tech Jn	60	28.43	46	20	6.11
Stadium	From Tech Jn	30	26.9	33	22	3.02
	To Tech Jn	30	24.8	28	22	1.67
Amakom	From Tech Jn	40	26.3	32	23	2.58
	To Tech Jn	40	26.5	30	22	1.96

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4.4 Summary of saturation Flows

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 the Transport Research Laboratory United Kingdom (TRL, 1993).

At Anloga intersection, the saturation flow rates from Tech junction approach was 1662pcu per hour per lane and that to Tech approach was 1742pcu per hour per lane. Also, for the stadium junction 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 flow rates and flow interruptions at approaches for specific times and cycles some of which were rejected for analysis due to flow interruptions. Vehicle mix, geometry of intersection, driver behaviour, public transport proportion in traffic stream stops near intersections along routes (within 20m) and pedestrian indiscriminate crossing due to location of attractions and roadside activity were identified as principal factors affecting flow. This collaborates Turner and Harahap (1993); Lu and Pernia, (1999); Kockelman and Shabih, (1999); Minh and Sano ~~(2003)~~ and Jacobs et al.(1981), 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, Amakom and Stadium in that order. High approach down grades and the presence of heavy vehicles especially at the Anloga intersection were also found to be major contributory factors to the deviation of the saturation flow rate from the ideal.

4.5 Results of Calibration of Synchro Models

Tables 4.8 and 4.10 below show the results of the comparison between computed and adjusted saturation flow rates speeds and headways for the selected intersections using the Chi Square Test (ρ-value).

Table 4.8: Comparison of Computed and Adjusted Saturation Flow Rates

Location	Direction	Computed Sat. Flow Rates (Sc) pcu/hr/lane	Adjusted Sat. Flow Rates (Sa)pcu/hr/lane	Ratio Sc/Sa	Chi square Test (ρ-value)
Anloga	To Tech	1742	1756	0.992	0.3269
	From Tech	1662	1700	0.978	
Stadium	To Tech	1643	1643	1.000	0.2494
	From Tech	1226	1267	0.968	
Amakom	To Tech	1286	1288	0.998	0.2398
	From Tech	1489	1535	0.970	

From Table 4.8 above, since ρ – values are all greater than 0.05 at 5% level of significance; it means that there is no significant difference between the computed saturation flow rates and adjusted saturation flow rates for the selected intersections. This therefore means that the field saturation flow rates are equal to the simulated saturation flow rates obtained from Synchro for Anloga, Stadium and Amakom intersections.

Table 4.9: Comparison of Computed and Adjusted Average Speeds

Location	Direction	Computed Average Speed (Spc) km/hr	Adjusted Average Speed (Spa)km/hr	Ratio Spc/Spa	Chi Test (ρ-value)
Anloga	To Tech	28.43	24	1.185	0.2383
	From Tech	26.63	23	1.103	
Stadium	To Tech	24.8	22	1.127	0. 4792
	From Tech	26.9	25	1.076	
Amakom	To Tech	26.5	23	1.152	0.1714
	From Tech	26.3	21	1.252	

From Table 4.9 above, ρ– values are all greater than 0.05 meaning that there is no significant difference between the computed speeds and adjusted speeds for the selected intersections. This means that the field speed values are equal to the speed values obtained from Synchro for Anloga, Stadium and Amakom intersections.

Table 4.10: Comparison of Computed and Adjusted Headways

Location	Direction	Computed Average Headway(hyc) secs	Adjusted Average Headway (hya) secs	Ratio hyc/hya	Chi Test (ρ-value)
Anloga	To Tech	2.56	3.27	0.783	0.6525
	From Tech	2.59	2.97	0.872	
Stadium	To Tech	2.44	3.18	0.767	0.58011
	From Tech	3.7	3.06	1.209	
Amakom	To Tech	3.54	2.94	1.204	0.69016
	From Tech	3.01	3.36	0.896	

From Table 4.10 above, ρ – values are also greater than 0.05 which means that there is no significant difference between the computed headways and adjusted headways for the selected intersections. This means that the field headway values are equal to the headway values obtained from Synchro for Anloga, Stadium and Amakom intersections.

A paired sampled t – test analysis was carried out to evaluate the variation in the computed and adjusted saturation flow rates, speeds and headways for the selected intersections.

For saturation flow rates at each approach of the selected intersections, the test results showed that there is no significant difference between the computed and adjusted saturation flow rates since the significance (2 – tailed) is more than 0.05.

For headways at each approach of the selected intersections, the test results showed that there is no significant difference between the computed and adjusted headways since the significance (2 – tailed) is more than 0.05.

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For speeds however at each approach of the selected intersections, the test results showed that there is significant difference between the computed and adjusted speeds since the significance (2 – tailed) is less than 0.05. It was found that a certain percentage of the variation in the simulated speed values is explained by the field speed values for the selected intersections.

Table 4.11: Summary Results of level of significance test carried out at Selected Intersections

Calibration Parameters	ρ - value	t - test
Saturation Flow Rates	Not Significant	Not Significant
Headways	Not Significant	Not Significant
Speed	Not Significant	Significant

4.6 Results of Regression Analysis

In order to calibrate the model, a regression analysis was carried out from the two major approaches at the selected intersections to establish which of either speed or headway is a better predictor or has a strong correlation with saturation flow rates. From the analysis carried out, it was established that headway had a strong correlation with saturation flow rates at each approach of the selected intersections.

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Table 4.12: Summary Results of Field Saturation, Speed and Headway from KNUST (Anloga)

Correlations

		Saturation flow rates for KNUST Approach at Anloga Intersection	SpeedFIELD	Headway for KNUST Approach at Anloga Intersection
Pearson Correlation	Saturation flow rates for KNUST Approach at Anloga Intersection	1.000	-.126	-.794
	SpeedFIELD	-.126	1.000	.070
	Headway for KNUST Approach at Anloga Intersection	-.794	.070	1.000
Sig. (1-tailed)	Saturation flow rates for KNUST Approach at Anloga Intersection	.	.170	.000
	SpeedFIELD	.170	.	.296
	Headway for KNUST Approach at Anloga Intersection	.000	.296	.
N	Saturation flow rates for KNUST Approach at Anloga Intersection	60	60	60
	SpeedFIELD	60	60	60
	Headway for KNUST Approach at Anloga Intersection	60	60	60

It can be seen that headway (-0.794) is a better predictor (strength) for saturation flow since it correlates very well with saturation flow though speed (-0.126) correlates well.

Table 13: Model Summary (b)

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.797(a)	.635	.622	451.078

a. Predictors: (Constant), Headway for KNUST Approach at Anloga Intersection, Speed FIELD

b. Dependent Variable: Saturation flow rates for KNUST Approach at Anloga Intersection

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The adjusted R square value is 0.622 which when converted to percentage is 62.2 percent. Thus the model (headway and speed) explains 62.2 percent of the variance in the saturation flows.

Table 4.14: ANOVA (b)

Model 1		Sum of Squares	df	Mean Square	F	Sig.
	Regression	20179755.840	2	10089877.920	49.589	.000(a)
	Residual	11597872.093	57	203471.440		
	Total	31777627.933	59			

a. Predictors: (Constant), Headway for KNUST Approach at Anloga Intersection, Speed FIELD

b. Dependent Variable: Saturation flow rates for KNUST Approach at Anloga Intersection

Table 15: Summary Results of co-efficient using Regression Analysis at Anloga

Coefficients

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.	Collinearity Statistics	
		B	Std. Error	Beta			Tolerance	VIF
1	(Constant)	362.416	576.527		5.832	.000		
	SpeedFIELD	-18.644	21.366	-.070	-.873	.387	.995	1.005
	Headway for KN Approach at Anl Intersection	463.630	47.144	-.789	-9.834	.000	.995	1.005

a. Dependent Variable: Saturation flow rates for KNUST Approach at Anloga Intersection

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The model shows that headway (-0.789) makes a unique contribution to explaining the saturation flow when the variance in the model is controlled for. On the other hand, Speed (-0.070) makes a less contribution to the model.

A look at the significant values confirms this assertion. Headway has a significance of 0.000 which is less than 0.05; hence headway makes a significant unique contribution to the prediction of saturation flow whiles Speed (0.387) makes a non significant contribution.

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

$$q = -463.630u - 18.644h + 3362.416$$

4.7 Comparing Computed and Adjusted performance indicators

In fulfillment of objective 1 of the thesis, the performance indicators were computed manually and compared with the adjusted performance indicators generated by the calibrated Synchro model to determine the extent of variation in the output.

The table below shows the compared computed and adjusted performance indicators at the selected intersections.

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Table 4.16: Comparison of computed and adjusted performance indicators at selected intersections

Performance Indicators	Anloga		Stadium		Amakom	
	Computed	Adjusted	Computed	Adjusted	Computed	Adjusted
Cycle Length (sec)	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 (sec)		135.0		41		133

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The computed intersection delays were arrived at using Webster's delay equation from HCM 2000 shown below.

D = [0.5C(1-g/C)2]/[1-(Xc*g/C)]

Where C = cycle length (s)

g = effective green time (s)

Xc = degree of saturation or v/c ratio

g/C = ratio of effective green to cycle length

Figure 4.1: Pictorial view of Anloga Intersection using SimTraffic Simulation

Figure 4.2: Pictorial view of Stadium Intersection using SimTraffic Simulation

Figure 4.3: Pictorial view of Amakom Intersection using SimTraffic Simulation

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4.8 Analysis of Alternative Phasing Plans

4.8.1 Change in phasing plan without Geometric Improvement

4.8.1.1 Anloga Intersection

In fulfillment of objective 2 of the thesis, an assessment of the phasing plan was made for various scenarios. The assessment was made for all three selected intersections.

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Figure 4.4 below shows the existing traffic situations together with their possible alternative phasing plans at the Anloga intersection.

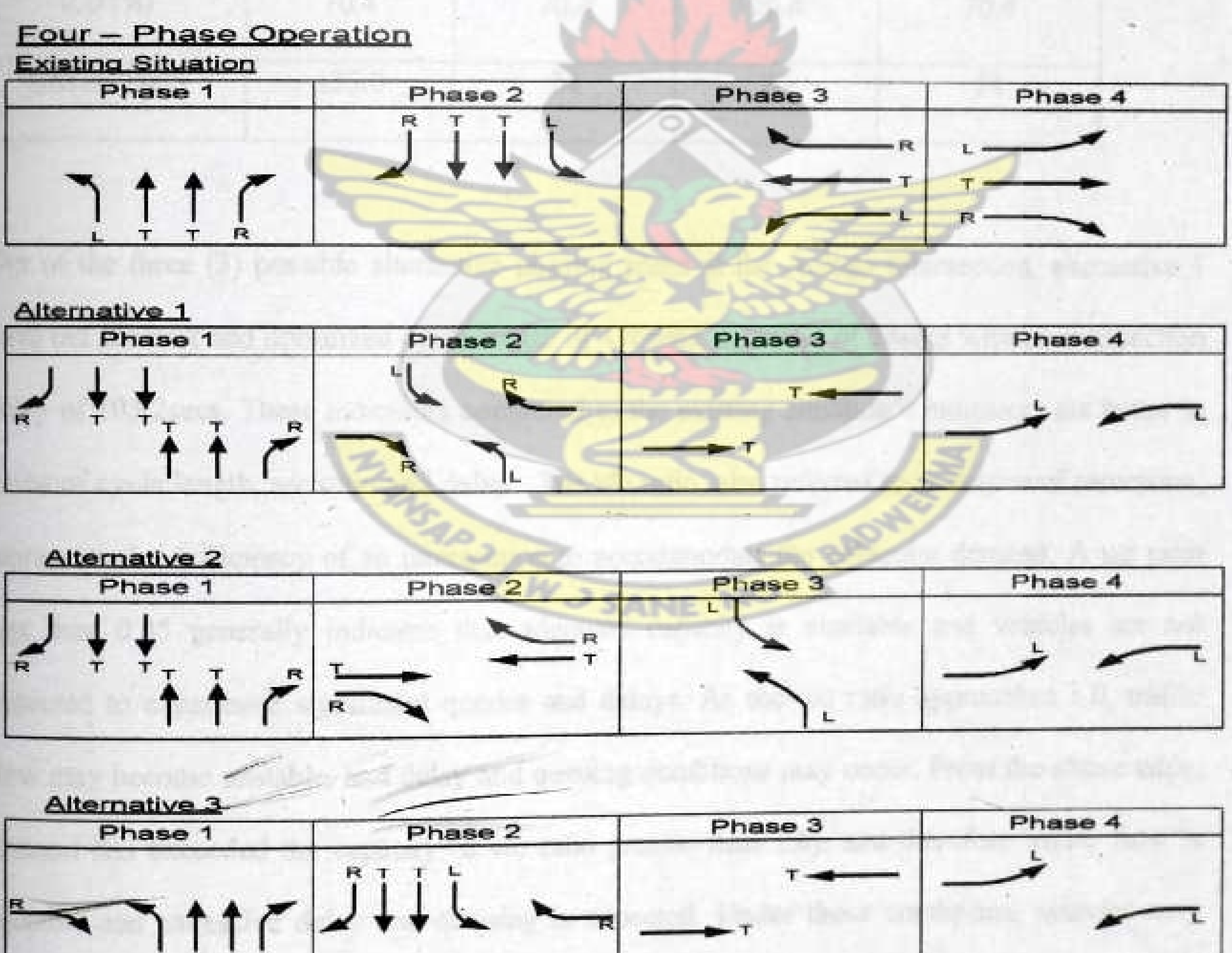

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Table 4.17: Summary Results of performance indicators for 3 Different Alternatives at Anloga with existing situation

Performance Indicators	Existing Situation	Alternative 1	Alternative 2	Alternative 3
Cycle Length (sec)	200	176	176	176
v/c ratio	2.79	1.25	2.65	2.65
Int. Delay (sec)	183.5	103.2	170.7	170.7
Int. LOS	F	F	F	F
ICU (%)	70.4	70.4	70.4	70.4
Offsets (sec)	135.0	24	24	24

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Out of the three (3) possible alternative phasing plans at the Anloga intersection, alternative 1 gave out the best and optimized cycle length of 176secs, an offset of 24secs with an intersection delay of 103.2secs. These indicators compared to the existing situation's indicators are better in terms of cycle length, v/c ratio and delay. The v/c ratio, also referred to as degree of saturation, represents the sufficiency of an intersection to accommodate the vehicular demand. A v/c ratio less than 0.85 generally indicates that adequate capacity is available and vehicles are not expected to experience significant queues and delays. As the v/c ratio approaches 1.0, traffic flow may become unstable, and delay and queuing conditions may occur. From the above table, demand has exceeded the capacity (a v/c ratio greater than 1.0), and therefore traffic flow is unstable and excessive delay and queuing is expected. Under these conditions, vehicles may require more than one signal cycle to pass through the intersection (known as a cycle failure).

4.8.1.2 Stadium Junction Intersection

Figure 4.5 below shows the existing traffic situations together with their possible alternative phasing plans at the Stadium intersection.

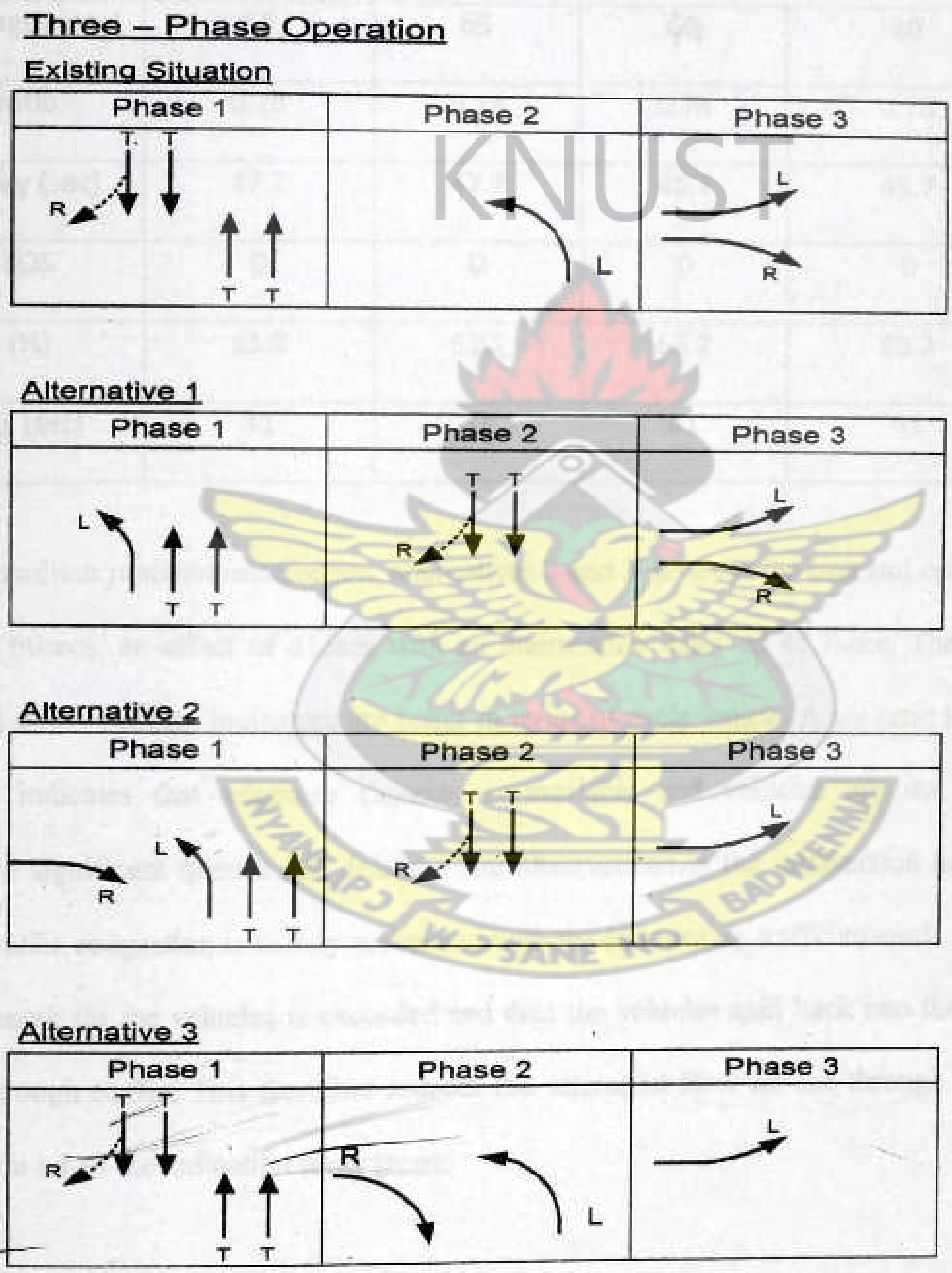

Table 4.18: Summary Results of performance indicators for 3 Different Alternatives at Stadium Junction with Base Scenario

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

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With the stadium junction intersection, alternatives 2 and 3 gave out the best and optimized cycle length of 60secs, an offset of 41secs with an intersection delay of 45.7secs. These indicators compared to the existing indicators are better in terms of cycle length. A v/c ratio less than 0.85 generally indicates that adequate capacity is available and vehicles are not expected to experience significant queues and delays. Site observations at the intersection however show that the traffic congestion is mostly associated with the left turning traffic towards stadium. The storage length for the vehicles is exceeded and then the vehicles spill back into the double lane for the through traffic. This therefore reduces the saturation flow for the through traffic at the intersection when the indication turns green.

4.8.1.3 Amakom Intersection

Figure 4.6 below shows the existing traffic situations together with their possible alternative phasing plans at the Amakom intersection.

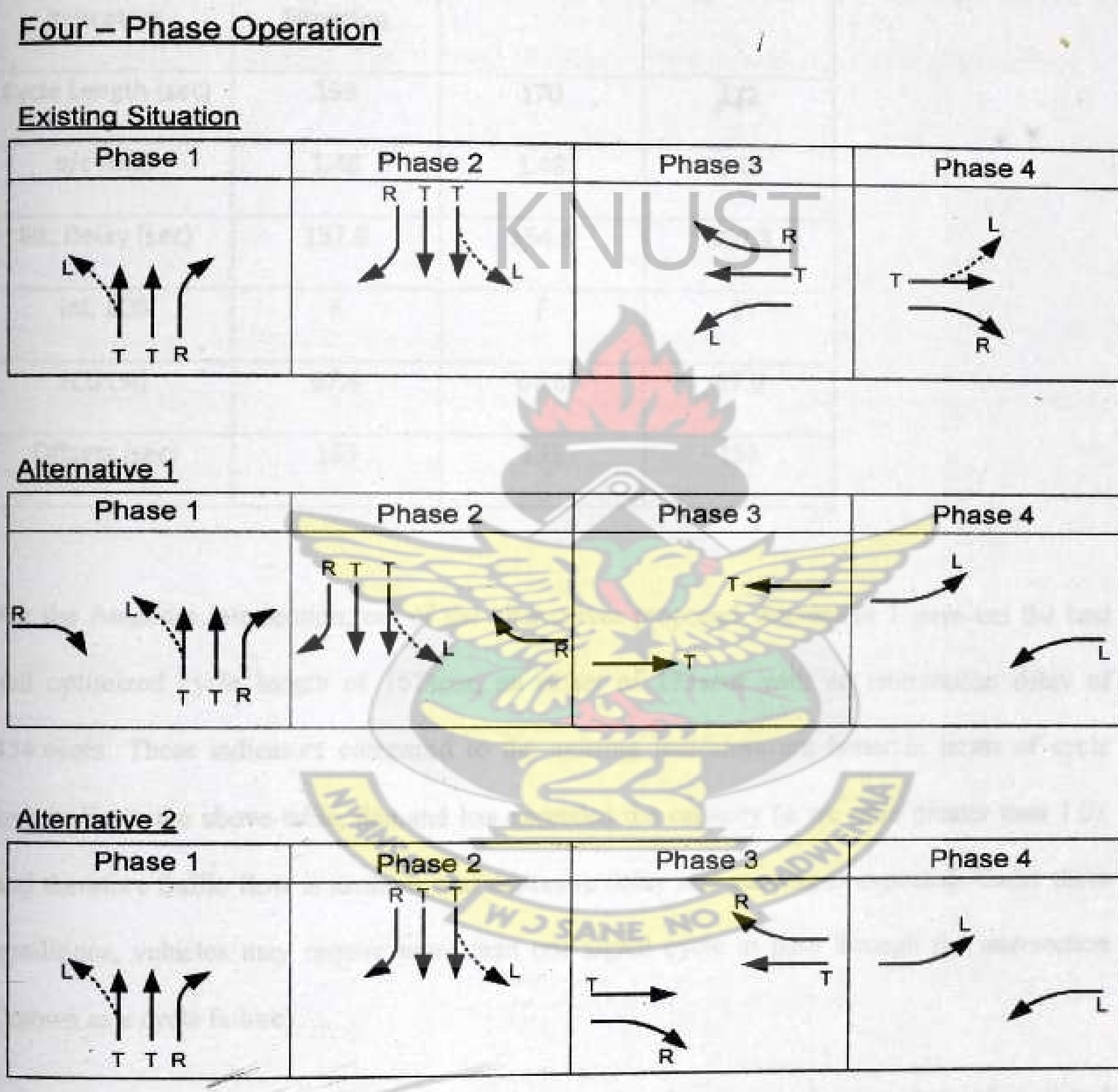

Table 4.19: Summary Results of performance indicators for 2 Different Alternatives at Amakom with Base Scenario

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

For the Amakom intersection, out of the alternatives proposed, alternative 1 gave out the best and optimized cycle length of 167secs, an offset of 133secs with an intersection delay of 154.6secs. These indicators compared to the existing indicators are better in terms of cycle length. From the above table, demand has exceeded the capacity (a v/c ratio greater than 1.0), and therefore traffic flow is unstable and excessive delay and queuing is expected. Under these conditions, vehicles may require more than one signal cycle to pass through the intersection (known as a cycle failure).

Since the LOS is still F at the intersection, it means that the situation has not changed at all and therefore something ought to be done to improve upon the intersection's performance.

The above alternatives when implemented will not result in performance improvement at the selected intersections. Rather, the alternatives that gave out the best cycle lengths and intersection delays will be further investigated by changing their geometry (addition of lane(s)) at the selected intersections. Performance at the selected intersections will then be evaluated in terms of delay, LOS and v/c ratio to know whether there has been significant improvement or not.

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4.8.2 Change in phasing plan with Geometric Improvement

The selected alternatives in figures 4.4 – 4.6 thus alternatives 1, 2 and 1 respectively from the phasing plan were further investigated upon by adding lane(s) to the through put traffic at the selected intersections.

Table 4.20: Summary of performance indicators with geometric improvement at Anloga

Performance Indicators	Existing Situation	Alternative 1	Addition of 1 lane	Addition of 2 lanes
Cycle Length (sec)	200	176	176	176
v/c ratio	2.79	1.25	0.86	0.78
Int. Delay (sec)	183.5	103.2	62.7	58.6
Int. LOS	F	F	E	E
ICU (%)	70.4	70.4	62.0	57.1
Offsets (sec)	135.0	24	24	24

The problem therefore is the saturation flow from the through put traffic, since when one lane was added to the major through put traffic from each approach at Anloga intersection, it was realized that the v/c ratio reduced from 1.25 to 0.86 with a corresponding reduction in the delay. The level of service also changed from F to E meaning that there has been an improvement at the intersection. When 2 lanes each was added to each major through put approach, there was a further reduction in the v/c ratio with a further corresponding decrease in delay. The level of service was still E meaning compared to the existing situation, the performance at the intersection had improved thereby ensuring free flow of traffic at the intersection when implemented. Similarly, the existing reservation was checked against the geometric improvement.

Table 4.21: Summary of performance indicators with geometric improvement at Stadium Junction

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

Therefore the problem is the saturation flow from the through put traffic, since when one lane was added to the major through put traffic from each approach at Stadium intersection, it was realized that the v/c ratio was still 0.78 with a corresponding reduction in the delay. The level of service did not change at all. However, when 2 lanes each was added to each major through put approach, the v/c ratio was the same with a further corresponding decrease in delay. The level of service was still D meaning compared to the existing situation, the performance at the intersection will be improved thereby ensuring free flow of traffic at the intersection when implemented. Again reservation was checked against geometric improvement.

Table 4.22: Summary of performance indicators with geometric improvement at Amakom

Performance Indicators	Existing Situation	Alternative 1	Addition of 1 lane	Addition of 2 lanes
Cycle Length (sec)	193	170	170	170
v/c ratio	1.48	1.48	1.03	0.82
Int. Delay (sec)	157.1	154.6	73.9	60.4
Int. LOS	F	F	E	E
ICU (%)	67.6	67.6	49.8	43.6
Offsets (sec)	133	133	133	133

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 delay of 73.9secs. The level of service changed from F to E. However, when 2 lanes are added on the major approaches through put traffic, the level of service was still E with a v/c ratio of 0.82 and a corresponding decrease in delay. This means that the performance at the intersection has improved thereby ensuring free flow of traffic when implemented. Existing reservation was checked against geometric improvement.

4.8.3 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 intersection 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. Constructing an interchange at Anloga will enable motorists to move very fast through it and therefore if nothing is done at Stadium and Amakom intersections to contain the fast moving vehicles from Anloga, then the interchange will not serve the purpose for which it will be constructed. Therefore, as an interchange is constructed at Anloga junction, Stadium and Amakom intersections must be coordinated to allow the free flow of traffic from Anloga and also help prevent spillbacks from Amakom and Stadium intersections.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The main conclusions of this study are summarized in the following points:

- The results of the calibration showed that the difference between the computed and adjusted performance indicators
- Changes in phasing plan without geometric improvement at the selected intersections only improved upon delay time at the intersections studied.
- Changes in phasing plan with geometric improvement at the selected intersections improved upon the intersection's level of service. There were considerable improvements in performance indicators such as delay and v/c ratio.
- Optimal cycle lengths Anloga, Stadium and Amakom intersections are 176secs, 60secs and 170secs respectively.
- Offsets for Anloga, Stadium and Amakom intersections are 135secs, 41secs and 133secs respectively.
- Providing an interchange at Anloga alone will not solve the congestion problems on the corridor unless Stadium junction and Amakom intersections are also coordinated.
- Flow along poorly maintained sidewalks allows pedestrians to cross at several locations thus introducing delays and compromising on overall performance of the intersections.
- The presence of hawkers also affects headways and flows through the intersections.

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5.2 RECOMMENDATIONS

Based on the conclusions arrived at, the following recommendations were proposed:

- Stadium junction 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 provided at the Anloga intersection to allow free movement of vehicles thereby minimizing congestion and accident occurrences.
- Hawking and roadside activity close to the intersections should be prohibited and strictly enforced.
- Intersection performance at signal intersections on arterial roads can be improved through proper and appropriate location of traffic generators, fuel stations, pedestrian and other road activity.
- Traffic rule enforcement should be intensified especially at intersections through road signs and arrest of drivers who violate them should be enforced to remove their impact on poor performance of signals.
- Further research needs to be investigated on the effect of left turning traffic at Stadium intersection.
- Further research needs to be undertaken on the 24th February Road by looking at the selected intersections as one unit.
- Further research needs to be carried out by investigating whether a combination of traffic actuated signals and an increase in the number of lanes on the major arterials as a medium term measure would improve upon the performance of the intersections.

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APPENDIX

APPENDIX A

ANLOGA INTERSECTION

Table A1: Turning movement counts at 15-minute interval for KNUST Approach

	Left Turns				Through Traffic				Right Turns			
Time	Veh	HV	Tot	%H	Veh	HV	Tot	%H	Veh	HV	Tot	%H
7:00 – 7:15	2	0	2	0	249	7	256	2.7	64	1	65	1.5
7:15 – 7:30	0	0	0	0	269	4	273	1.5	65	1	66	1.5
7:30 – 7:45	2	0	2	0	224	10	234	4.3	58	7	65	10.8
7:45 – 8:00	0	1	1	100	236	4	240	1.7	84	6	90	6.7
8:00 – 8:15	2	0	2	0	257	9	266	3.4	71	0	71	0
8:15 – 8:30	1	0	1	0	255	5	260	1.9	86	0	86	0
8:30 – 8:45	1	1	2	50	226	10	236	4.2	65	5	70	7.1
8:45 – 9:00	2	0	2	0	207	13	220	5.9	58	8	66	12.1
9:00 – 9:15	2	0	2	0	264	6	270	2.2	75	2	77	2.6
9:15 – 9:30	1	0	1	0	235	5	240	2.1	66	3	69	4.3
9:30 – 9:45	2	0	2	0	222	5	227	2.2	64	0	64	0
9:45-10:00	2	0	2	0	170	3	173	1.7	54	2	56	3.6

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Table A2: Turning movement counts at 15-minute interval for AIRPORT Approach

	Left Turns				Through Traffic				Right Turns			
Time	Veh	HV	Tot	%H	Veh	HV	Tot	%H	Veh	HV	Tot	%H
7:00 – 7:15	24	4	28	14.3	2	3	5	60	51	4	55	7.3
7:15 – 7:30	20	0	20	0	9	1	10	10	50	7	57	12.3
7:30 – 7:45	22	3	25	12	1	1	2	50	96	4	100	4
7:45 – 8:00	28	4	32	12.5	2	1	3	33.3	104	2	106	18.9
8:00 – 8:15	30	4	34	11.8	2	0	2	0	103	5	108	4.6
8:15 – 8:30	25	3	28	10.7	3	0	3	0	75	8	83	9.6
8:30 – 8:45	31	2	33	6.1	2	1	3	33.3	65	5	70	7.1
8:45 – 9:00	32	6	38	15.8	1	0	1	0	87	3	90	3.3
9:00 – 9:15	29	4	33	12.1	2	1	3	33.3	83	5	88	5.7
9:15 – 9:30	32	4	36	11.1	3	0	3	0	85	2	87	2.3
9:30 – 9:45	35	2	37	5.4	2	0	2	0	73	0	73	0
9:45-10:00	17	1	8	5.6	1	1	2	50	85	1	86	1.2

Table A3: Turning movement counts at 15-minute interval for AMAKOM Approach

	Left Turns				Through Traffic				Right Turns			
Time	Veh	HV	Tot	%H	Veh	HV	Tot	%H	Veh	HV	Tot	%H
7:00 – 7:15	55	6	61	9.8	231	12	243	4.9	8	0	8	0
7:15 – 7:30	45	2	47	4.3	273	21	294	7.1	6	0	6	0
7:30 – 7:45	50	3	53	5.7	303	13	316	4.1	9	0	9	0

7:45 – 8:00	58	5	63	7.9	289	7	296	2.4	10	0	10	0
8:00 – 8:15	42	2	44	4.5	233	16	249	6.4	11	0	11	0
8:15 – 8:30	51	5	56	8.9	266	8	274	2.9	11	2	13	15.4
8:30 – 8:45	57	3	60	5	265	9	274	3.3	3	0	3	0
8:45 – 9:00	68	6	74	8.1	271	14	285	4.9	8	0	8	0
9:00 – 9:15	75	6	81	7.4	273	13	286	4.5	8	0	8	0
9:15 – 9:30	63	6	69	8.7	262	17	279	6.1	11	0	11	0
9:30 – 9:45	59	8	67	11.9	322	18	340	5.3	6	2	8	25
9:45-10:00	55	6	61	9.8	293	13	306	4.2	11	1	12	8.3

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Table A4: Turning movement counts at 15-minute interval for ANLOGA Approach

	Left Turns				Through Traffic				Right Turns			
Time	Veh	HV	Tot	%H	Veh	HV	Tot	%H	Veh	HV	Tot	%H
7:00 – 7:15	36	2	38	5.3	4	0	4	0	4	0	4	0
7:15 – 7:30	32	1	33	3	3	0	3	0	11	0	11	0
7:30 – 7:45	26	2	28	7.1	5	2	7	28.6	14	0	14	0
7:45 – 8:00	29	3	32	9.4	2	3	5	60	9	0	9	0
8:00 – 8:15	34	1	35	2.9	3	2	5	40	10	0	10	0
8:15 – 8:30	37	1	38	2.6	8	4	12	33.3	11	0	11	0
8:30 – 8:45	35	1	36	2.8	5	1	6	16.7	5	0	5	0
8:45 – 9:00	14	0	14	0	3	2	5	40	8	0	8	0
9:00 – 9:15	19	5	24	20.8	5	2	7	28.6	12	1	13	7.7

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9:15 – 9:30	26	1	27	3.7	1	4	5	80	11	0	11	0
9:30 – 9:45	18	1	19	5.3	7	3	10	30	15	4	19	21.1
9:45-10:00	18	2	20	10	4	2	6	33.3	2	3	5	60

STADIUM JUNCTION INTERSECTION

Table A5: Turning movement counts at 15-minute interval for STADIUM Approach

	Left Turns				Right Turns			
Time	Veh	HV	Total	% H	Veh	HV	Total	% H
7:00 – 7:15	20	0	20	0	27	1	28	3.6
7:15 – 7:30	20	3	23	13.0	22	2	24	8.3
7:30 – 7:45	28	1	29	3.4	18	1	19	5.3
7:45 – 8:00	25	0	25	0	10	0	10	0
8:00 – 8:15	20	0	20	0	12	1	13	7.7
8:15 – 8:30	20	2	22	9.1	15	6	21	28.6
8:30 – 8:45	20	1	21	4.8	16	3	19	15.8
8:45 – 9:00	28	0	28	0	17	4	21	19.0
9:00 – 9:15	24	1	25	4.0	16	4	20	20
9:15 – 9:30	32	0	32	0	22	1	23	4.3
9:30 – 9:45	14	2	16	12.5	10	1	11	9.1
9:45-10:00	17	4	21	19.0	19	3	22	13.6

Table A6: Turning movement counts at 15-minute interval for AMAKOM Approach

	Through Traffic				Right Turns			
Time	Veh	HV	Total	% H	Veh	HV	Total	% H
7:00 – 7:15	173	7	180	3.9	17	0	17	0
7:15 – 7:30	188	6	194	3.1	7	1	8	12.5
7:30 – 7:45	168	4	172	2.3	7	1	8	12.5
7:45 – 8:00	183	4	187	2.1	7	1	8	12.5
8:00 – 8:15	160	9	169	5.3	12	2	14	14.3
8:15 – 8:30	177	8	185	4.3	5	0	5	0
8:30 – 8:45	144	4	148	2.7	6	0	6	0
8:45 – 9:00	205	5	210	2.4	12	1	13	7.7
9:00 – 9:15	192	10	202	4.95	8	0	8	0
9:15 – 9:30	189	7	196	3.6	9	2	11	18.2
9:30 – 9:45	189	8	197	4.1	6	1	7	14.3
9:45-10:00	206	10	216	4.6	5	0	5	0

Table A7: Turning movement counts at 15-minute interval for ANLOGA Approach

	Left Turns				Through Traffic			
Time	Veh	HV	Total	% H	Veh	HV	Total	% H
7:00 – 7:15	103	5	108	4.62	162	0	162	0
7:15 – 7:30	97	4	101	3.96	225	0	225	0
7:30 – 7:45	110	1	111	0. 9	235	2	237	0.84

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7:45 – 8:00	103	9	112	8.03	262	1	263	0.38
8:00 – 8:15	100	7	107	6.5	240	2	242	0.83
8:15 – 8:30	94	3	97	3.1	195	5	200	2.5
8:30 – 8:45	96	1	97	1.03	241	7	246	2.82
8:45 – 9:00	103	3	106	2.8	205	6	211	2.84
9:00 – 9:15	81	4	85	4.7	214	2	216	0.93
9:15 – 9:30	99	3	102	2.94	213	1	214	0.47
9:30 – 9:45	89	5	94	5.3	197	3	200	1.5
9:45-10:00	79	9	88	10.2	207	0	297	0

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AMAKOM INTERSECTION

Table A8: Turning movement counts at 15-minute interval for STADIUM Approach

	Left Turns				Through Traffic				Right Turns			
Time	Veh	HV	Tot	%H	Veh	HV	Tot	%H	Veh	HV	Tot	%H
7:00 – 7:15	5	0	5	0	27	3	30	10	0	1	1	100
7:15 – 7:30	20	0	20	0	32	2	34	5.9	1	0	1	0
7:30 – 7:45	9	0	9	0	24	3	27	11.1	1	0	1	0
7:45 – 8:00	8	0	8	0	33	3	36	8.3	0	3	3	100
8:00 – 8:15	10	1	11	9.1	28	0	28	0	1	0	1	0
8:15 – 8:30	13	1	14	7.1	27	1	28	3.6	0	4	4	100
8:30 – 8:45	12	1	13	7.7	31	1	32	3.1	0	1	1	100
8:45 – 9:00	15	0	15	0	33	4	37	10.8	1	0	1	0

9:00 – 9:15	8	0	8	0	29	4	33	12.1	0	0	0	0
9:15 – 9:30	8	0	8	0	34	7	41	17.1	0	0	0	0
9:30 – 9:45	8	0	8	0	37	5	42	11.9	0	1	1	100
9:45-10:00	19	1	20	5	28	1	29	3.4	0	2	2	100

Table A9: Turning movement counts at 15-minute interval for ASAWASE Approach

	Left Turns				Through Traffic				Right Turns			
Time	Veh	HV	Tot	%H	Veh	HV	Tot	%H	Veh	HV	Tot	%H
7:00 – 7:15	33	5	38	13.1	55	3	58	5.2	24	0	24	0
7:15 – 7:30	30	4	34	11.8	52	6	58	10.3	31	2	33	6.1
7:30 – 7:45	21	3	24	12.5	46	3	49	6.1	29	0	29	0
7:45 – 8:00	23	1	24	4.2	44	6	50	12	39	3	42	7.1
8:00 – 8:15	26	4	30	13.3	48	3	51	5.9	37	0	37	0
8:15 – 8:30	20	1	20	4.8	51	0	51	0	43	3	46	6.5
8:30 – 8:45	13	1	14	7.1	48	7	55	12.7	24	0	24	0
8:45 – 9:00	35	0	35	0	59	6	65	9.2	32	0	32	0
9:00 – 9:15	17	1	18	5.6	38	4	42	9.5	29	2	31	6.5
9:15 – 9:30	29	3	32	9.4	40	4	44	9.1	32	0	32	0
9:30 – 9:45	27	4	31	12.9	43	7	50	14	28	0	28	0
9:45-10:00	28	2	30	6.7	47	5	52	9.6	18	0	18	0

Table A10: Turning movement counts at 15-minute interval for ANLOGA Approach

	Left Turns				Through Traffic				Right Turns			
Time	Veh	HV	Tot	%H	Veh	HV	Tot	%H	Veh	HV	Tot	%H
7:00 – 7:15	13	1	14	7.1	243	0	243	0	22	2	24	8.3
7:15 – 7:30	37	1	38	2.6	285	3	288	1.04	26	2	28	7.1
7:30 – 7:45	20	2	22	9.1	264	1	265	0.4	12	3	15	20
7:45 – 8:00	32	2	34	5.9	312	3	315	0.95	42	0	42	0
8:00 – 8:15	18	5	23	21.7	243	5	248	2.0	22	3	25	12
8:15 – 8:30	27	3	30	10	304	12	316	3.8	20	1	21	4.8
8:30 – 8:45	12	0	12	0	227	11	238	4.6	19	6	25	24
8:45 – 9:00	15	1	16	6.3	293	10	303	3.3	24	3	27	11.1
9:00 – 9:15	16	1	17	5.9	243	6	249	2.4	21	3	24	12.5
9:15 – 9:30	29	3	32	9.4	302	8	310	2.6	14	3	17	17.6
9:30 – 9:45	28	2	30	6.7	289	2	291	0.7	25	4	29	13.8
9:45-10:00	17	0	17	0	226	7	233	3.0	22	3	25	12

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Table A11: Turning movement counts at 15-minute interval for KEJETIA Approach

	Left Turns				Through Traffic				Right Turns			
Time	Veh	HV	Tot	%H	Veh	HV	Tot	%H	Veh	HV	Tot	%H
7:00 – 7:15	~~10~~	1	~~11~~	~~9.1~~	165	4	169	2.4	5	0	5	0
7:15 – 7:30	20	0	20	0	210	4	214	1.9	15	1	16	6.3
7:30 – 7:45	8	0	8	0	161	1	162	0.62	13	0	13	0

7:45 – 8:00	19	1	20	5	192	4	196	2.0	17	0	17	0
8:00 – 8:15	9	1	10	10	152	6	158	3.8	10	0	10	0
8:15 – 8:30	18	0	18	0	171	5	176	2.8	18	0	18	0
8:30 – 8:45	14	0	14	0	150	0	150	0	12	0	12	0
8:45 – 9:00	21	0	21	0	209	5	214	2.3	27	0	27	0
9:00 – 9:15	13	1	14	7.1	172	5	177	2.8	12	0	12	0
9:15 – 9:30	21	0	21	0	185	6	191	3.1	13	0	13	0
9:30 – 9:45	19	1	20	5	198	5	203	2.5	9	0	9	0
9:45-10:00	20	1	21	4.7	201	5	206	2.4	9	0	9	0

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APPENDIX B

HEADWAY AND SATURATION FLOW RATES

Table B1: Headway and Saturation Flow Rate Values for ANLOGA Intersection

FROM TECH				TO TECH			
Headway	Saturation Flow Rates	Headway	Saturation Flow Rates	Headway	Saturation Flow Rates	Headway	Saturation Flow Rates
2.53	1423	2.23	1614	3.11	1158	4.47	805
1.69	2130	7.71	467	2.68	1343	4.03	893
2.14	1682	2.93	1229	4.42	814	1.38	2609
3.28	1098	3.13	1150	1.32	2727	4.46	807
1.82	1978	2.46	1463	2.04	1765	2.66	1353
0.93	3871	1.86	1935	8.51	423	2.3	1565
2.48	1452	4.7	766	1.19	3025	4.37	824
2.59	1390	2.03	1773	0.92	3913	1.36	2647
6.55	550	2.84	1268	1.75	2057	1.56	2308
2.92	1233	3.83	940	2.38	1513	1.47	2449
0.86	4186	4.38	822	2.12	1698	2.21	1629
1.88	1915	2.06	1748	2.02	1782	1.48	2432
2.32	1552	1.96	1837	2.69	1338	1.58	2278
2.2	1637	2.93	1229	4.28	841	2.43	1481
2.59	1390	1.77	2034	8.93	403	1.57	2293
1.48	2432	2.59	1390	1.76	2045	1.87	1925
2.26	1593	2.23	1614	1.75	2057	2.87	1254
2.2	1637	2.92	1233	1.18	3051	2.39	1506

1.53	2353	2.32	1552	4.52	796	1.57	2293
1.80	2000	1.80	2000	1.28	2813	1.98	1818
5.69	633	2.52	1429	2.01	1791	1.60	2250
3.23	1115	1.99	1809	2.03	1791	4.43	813
2.51	1434	1.27	2835	2.35	1532	2.17	1659
3.52	1023	2.56	1406	2.25	1600	3.39	1062
1.83	1967	1.73	2081	3.01	1196	2.49	1446
1.08	3333	3.21	1121	1.28	2813	2.18	1651
1.84	1957	1.03	3495	2.49	1446	2.05	1756
1.59	2264	3.27	1101	2.05	1756	2.64	1364
2.78	1295	4.18	861	3.50	1029	1.92	1875
1.87	1925	3.37	1068	1.45	2483	1.47	2449

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Table B2: Headway and Saturation Flow Rate Values for STADIUM Intersection

FROM TECH				TO TECH			
Headway	Saturation Flow Rates	Headway	Saturation Flow Rates	Headway	Saturation Flow Rates	Headway	Saturation Flow Rates
2.53	1423	3.57	1008	1.32	2727	1.94	1856
2.45	1469	1.63	2209	1.97	1827	2.78	1295
4.89	736	10.0	360	2.14	1682	3.16	1139
5.41	665	1.94	1856	1.63	2209	2.85	1263
1.33	2707	3.95	911	2.86	1259	3.06	1176
2.04	1765	2.75	1309	2.55	1412	2.02	1782

2.14	1682	3.47	1037	1.53	2353	1.83	1967
7.85	459	1.63	2209	1.73	2081	5.31	678
3.78	952	3.06	1176	2.10	1714	1.53	2353
2.96	1216	6.94	519	1.84	1957	2.24	1607
2.35	1532	2.04	1765	2.55	1412	4.28	841
2.35	1532	2.65	1358	2.13	1690	2.04	1765
3.06	1176	4.81	748	2.95	1220	3.13	1150
4.99	721	5.29	681	1.45	2483	2.93	1229
3.98	905	5.2	692	3.60	1000	1.65	2182

Table B3: Headway and Saturation Flow Rates Values for AMAKOM Intersection

FROM TECH				TO TECH			
Headway	Saturation Flow Rates	Headway	Saturation Flow Rates	Headway	Saturation Flow Rates	Headway	Saturation Flow Rates
3.78	952	2.35	1532	6.30	571	2.75	1309
1.12	3214	2.65	1358	8.67	415	2.04	1765
3.87	930	1.12	3214	3.27	1101	2.66	1353
3.78	952	2.55	1412	5.10	706	1.63	2209
3.36	1071	3.68	978	7.03	512	3.24	1111
4.59	784	5.50	655	3.16	1139	2.93	1227
2.45	1469	1.23	2927	3.07	1173	3.13	1150
2.65	1358	4.18	861	3.96	909	2.47	1457
2.66	1353	2.90	1241	4.25	847	3.25	1108

3.55	1014	3.07	1173	3.36	1071	2.16	1667
1.32	2727	2.14	1682	4.10	878	3.26	1104
6.74	534	1.64	2195	2.30	1565	14.08	256
2.53	1423	2.30	1565	2.63	1369	2.04	1765
3.16	1139	1.98	1818	3.94	/ 914	2.14	1682
3.24	1111	3.46	1040	1.69	2130	9.59	375
2.76	1304	1.84	1957	1.83	1967	1.33	2707
2.53	1423	5.82	619	4.36	826	3.77	955
4.63	778	3.67	981	3.78	952	3.18	1132
3.13	1150	1.52	2368	5.10	706	2.43	1481
3.29	1094	1.32	2727	3.29	1094	2.93	1229
1.83	1967	1.43	2517	2.45	1469	3.36	1071
2.14	1682	1.94	1856	2.25	1600	4.37	824
1.94	1856	1.12	3214	2.03	1773	7.24	497
3.47	1037	7.24	497	1.23	2927	5.64	638
1.84	1957	5.25	686	3.77	955	4.63	778
1.55	2323	2.43	1481	2.55	1412	2.67	1348
4.37	824	1.94	1856	1.13	3186	3.33	1081
5.91	609	3.76	957	1.93	1865	1.98	1818
3.16	1139	3.56	1011	2.35	1532	2.60	1385
1.22	2951	4.24	849	2.55	1412	2.16	1667

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APPENDIX C

INTERSECTION SPEED VALUES

Table C1: Speed Values Obtained from KNUST Approach at Anloga Intersection

THROUGH TRAFFIC			RIGHT TURNS		
SPEED	SPEED	SPEED	SPEED	SPEED	SPEED
20	24	26	19	25	28
20	24	26	20	25	29
20	24	26	20	25	29
21	24	26	21	25	30
21	24	26	22	25	30
21	24	26	23	25	30
21	24	27	23	25	30
21	24	27	23	25	30
21	24	27	23	25	30
21	24	27	23	25	30
21	24	27	23	25	30
22	24	27	23	26	30
22	25	28	23	26	30
22	25	28	24	26	30
22	25	28	24	26	30
22	25	28	24	26	31
22	25	28	24	26	31
22	25	28	24	26	31
22	25	28	24	26	31

23	25	30	24	26	32
23	25	30	24	26	32
23	25	30	24	26	32
23	25	30	24	27	32
23	25	30	25	27	33
23	25	31	25	27	33
24	25	31	25	27	33
24	25	31	25	28	34
24	25	32	25	28	35
24	25	35	25	28	36
24	25	36	25	28	41

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Table C2: Speed Values Obtained from Amakom Approach at Anloga Intersection

THROUGH TRAFFIC			LEFT TURNS		
SPEED	SPEED	SPEED	SPEED	SPEED	SPEED
20	25	29	20	26	29
20	25	29	21	26	29
21	25	30	21	26	29
21	26	30	23	26	29
22	26	30	24	26	30
22	26	30	24	26	30
22	26	30	24	26	30
23	26	30	24	26	30

23	26	31	24	27	30
23	26	31	25	27	30
24	26	31	25	27	30
24	26	31	25	27	30
24	26	31	25	27	30
24	26	31	25	27	30
24	26	31	25	27	31
24	27	32	25	27	31
24	27	32	25	27	31
24	27	32	25	28	31
24	27	34	25	28	31
24	27	36	25	28	31
25	27	38	25	28	31
25	27	38	25	28	32
25	28	39	25	28	32
25	28	39	26	28	32
25	28	40	26	28	33
25	28	40	26	28	34
25	28	44	26	28	34
25	28	46	26	28	34
25	28		26	28	
25	28		26	29	

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25	29		26	29	

Table C3: Speed Values Obtained from KNUST and Amakom Approach at Stadium Intersection

FROM TECH				TO TECH			
Speed	Speed	Speed	Speed	Speed	Speed	Speed	Speed
22	25	28	29	22	24	25	26
23	25	28	29	22	24	25	26
23	25	28	29	23	24	25	26
23	26	28	30	23	24	25	26
24	27	28	30	23	24	25	27
24	27	28	30	23	24	25	27
24	27	28	31	24	25	25	27
24	28	29	31	24	25	26	27
25	28	29	32	24	25	26	28
25	28	29	33	24	25	26	28

KNUST

Table C4: Speed Values Obtained from KNUST Approach at Amakom Intersection

THROUGH TRAFFIC				LEFT TURNS			
Speed	Speed	Speed	Speed	Speed	Speed	Speed	Speed
23	24	26	28	21	23	24	25
23	24	26	28	21	23	24	25
23	25	27	28	21	23	24	26

23	25	27	29	22	23	24	26
23	25	27	29	22	23	24	26
23	25	27	30	22	23	24	26
23	25	27	30	22	23	25	27
24	26	28	31	23	23	25	27
24	26	28	32	23	24	25	27
24	26	28	32	23	24	25	27

KNUST

Table C5: Speed Values Obtained from Kejetia Approach at Amakom Intersection

THROUGH TRAFFIC				LEFT TURNS			
Speed	Speed	Speed	Speed	Speed	Speed	Speed	Speed
22	25	27	28	21	22	23	24
23	25	27	28	22	23	23	24
24	25	27	28	22	23	23	24
24	25	27	28	22	23	23	24
24	25	27	29	22	23	24	25
24	26	28	29	22	23	24	25
25	26	28	29	22	23	24	25
25	26	28	29	22	23	24	25
25	26	28	30	22	23	24	25
25	26	28	30	22	23	24	25

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