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Evaluation of the Effects of Signalisation on Roundabout Capacity Using VISSIM:

A Case Study of the Suame Roundabout in Kumasi, Ghana

By

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DECLARATION

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

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ABSTRACT

Increasingly, many roundabouts in Ghanaian cities are becoming problematic due to capacity challenges resulting from rapid traffic growth. Signalisation is known to improve capacity, shorten queue length, and reduce delay at roundabouts but this option has not been used in Ghana. In this study, three model options for signalising roundabouts, namely Approach-by-Approach Control, Metered Approach, and Full Signalisation, were explored for analysis and comparison of their effects on roundabout in Kumasi as a case study. The aim of the study was to establish the model option that best addresses traffic problems at the Suame Roundabout in terms of improved capacity, reduced delays and queue lengths. Field traffic volume studies for the entire roundabout's approaches were performed. Travel time and delay studies together with queue lengths measurement for the subject approach (South East Approach) were also undertaken. Geometric data as well as data from the field study described earlier were used to calibrate a model for the existing situation. Capacity, delay, queue length and degree of saturation were observed for the signalised options and existing un-signalised. The results indicated that the Full Signalisation Model produced the best parameter results and the Approach-by-Approach Model the worst among all the models including the calibrated existing model. Full signalisation of the Suame roundabout is recommended to improve capacity and reduce vehicular delays and queue lengths. However, under budgetary constraints, the Metered Approach option may provide more consistency in operations at the roundabout as compared to the current situation where movements at peak periods are traffic-warden controlled.

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LIST OF ABBREVIATIONS

CORSIM – (CORridor SIMulation) software program
EA – East Approach

FHWA – Federal Highway Administration

HCM – Highway Capacity Manual

MOE – Measures of Effectiveness

NCHRP – National Cooperative Highway Research Program

NEA – North-East Approach

NWA – North-West Approach

ODOT – Oregon Department of Transportation
PARAMICS – (PARAllel MICroscopic Simulation) software program

PTV- *Planung Transport Verkehr AG*

SIDRA – Signalised and Un-signalised Design and Research Aid

SEA – South East Approach

VISSIM - *Verkehr In Stadten – Simulationsmodell* (Traffic in cities – simulation models)

WA – West Approach

WSDOT – Washington State Department of Transportation

CHAPTER 1: INTRODUCTION

1.1 Background

Increasingly, many roundabouts in Ghanaian cities are becoming problematic due to capacity challenges resulting from rapid traffic growth. As traffic grow and congestion continue to outpace the capacity of the road network the roundabouts will remain bottlenecks. Therefore the need to find ways of improving the capacity and performance of roundabouts cannot be overemphasised. Significantly, such improvement will go a long way to enhance flow and safety of an entire road network.

A Roundabout is a type of intersection which primarily serves as a location within the road network for change of direction. In order to increase capacity, improve delay, safety and other performance parameters of roundabouts, engineers in the United Kingdom introduced the concept of signalizing roundabouts. Signalisation of a roundabout can be described as the use of a traffic signal system to control traffic flow at a roundabout.

The idea of combining traffic signals and roundabout to improve capacity and performance may sound contradictory to many and therefore researching into such a concept with simulation could not have been any more appropriate.

Thus, this study focused on the effect of signalisation on roundabout capacity and other measures of effectiveness such as delay and queues.

1.2 Problem Statement

In Ghana, when the capacity of a roundabout is exceeded or the performance parameters become poor on more than one of the approaches, it is common to find any of the following interventions being adopted to solve the problem especially when there are more than four legs:

1. Police personnel or traffic wardens or private individuals controlling movement of traffic, which is inefficient, labour intensive and unreliable
2. Change of control by replacing with a traffic signal and change of intersection layout.
3. Grade-separation, which is usually expensive.

These three measures have differing costs and timelines to come on stream. Police personnel can be deployed and often redeployed even when traffic is still very high. Additionally, in bad weather and situations where the lighting is poor it is unsafe to deploy Police personnel or traffic wardens. Controlling roundabouts by this method is associated with long queues and delays because it is too arbitrary. Figure 1.1 below shows two Police personnel controlling movement of traffic at the Suame Roundabout.

Figure 1.1. Two Police Personnel Controlling Traffic Movement at Suame Roundabout

In the case of traffic signal control and grade separation, huge civil and infrastructure construction is involved. When signals are used, accidents may increase, also, poor light and unreliable power sources are some of the challenge to signalisation. The situation is compounded when the number of legs exceed four as is the case of Suame. Interchanges are very expensive and take very long time to come in stream. Suame has had proposed interchange design for over a decade. Road users continue to suffer delays and frustrations resulting from queuing traffic. The environmental condition brought about by the pollution from vehicles is terrible. The intersection suffers from gridlock making queues block intersection within 1km of the approaches. In the medium term, an intervention must be initiated to ease the congestion and improve throughput especially at the AM peak and PM peak conditions.

1.3 Research Objectives

i. To model and simulate the existing traffic situation at the roundabout and assess its capacity, delay and queue lengths with a calibrated VISSIM software. ii. To investigate various signalised roundabout options to improve throughput iii. Study the effect of signalisation on queuing delay and queue length of traffic

1.4 Justification of the Study

It is rare to find research on signalised roundabouts in Ghana. This study will therefore contribute significantly to signalisation of roundabout design and implementation in the following ways:

This study will reveal more efficient ways of controlling traffic in saturated conditions at roundabouts as compared to the deployment of Police personnel or Traffic wardens to undertake same activity.

The research will also show a relatively less expensive means of improving short term capacity and delay at roundabout as compared to grade-separation. Conclusions and recommendation from this research will be useful in subsequent studies and inform policy direction regarding roundabouts and signalisation.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Intersections, and for that matter roundabouts, are critical to the performance of a transport corridor and therefore enhancing their capacity and other flow parameters are important to the smooth flow of traffic. This chapter focuses on various aspects of this research in literature and also seeks to highlight their relevance. The sections below discusses roundabouts, signalisation, and tools for roundabout performance analysis as have been reported in literature.

2.2 Roundabouts

The NCHRP (2010) defines a roundabout as a form of circular intersection in which traffic travels counter clockwise around a central island and in which entering traffic must yield to circulating traffic. Roundabouts also function as an alternative form of intersection traffic control.

There are specific defining characteristics that differentiate roundabouts from other circular intersections. These include: Yield control at each approach, Separation of conflicting traffic movements by pavement markings or raised islands, Geometric characteristics of the central island that typically allow travel speeds of less than 50km/h, parking not usually allowed within the circulating roadway (Garber & Hoel, 2009).

In order to work efficiently, sufficient gaps must appear in the circulating flows on the roundabout that drivers then accept. Traffic on the entry arms can thus enter, circulate and then leave at their desired exit arm. Its operation has, therefore, certain similarities to that of a priority intersection. Dealing with a roundabout intersection is more complex as there is no clear identifiable major road traffic flow that can be used as a basis for designing the junction, with the circulating flow depending on the operation of all entry arms to it.

Apart from their ability to resolve conflicts in traffic as efficiently as possible, roundabouts are often used in situations where there is a) a change in road functional class. b) a clear alteration in the direction of the road; c) a change from an urban to a rural environment (Garber & Hoel, 2009).

2.3 Overview of Signalised Roundabouts

Roundabouts have merits such as capacity enhancement and increased safety over regular signalized intersections. However, they have disadvantages when there are unbalanced traffic conditions. This occurs when entering and circulating traffic are not spread evenly, typically during peak periods when traffic flows have predominantly directional characteristics (Natalizio, 2005). In order to enhance the capacity of roundabout and check unbalanced flow roundabouts may be signalised.

Signalisation of roundabouts was first experimented with in 1959 in the UK to prevent circulating traffic from blocking entering traffic during peak periods. With the introduction of the offside-priority rule in roundabout operation in the mid-1960s, various operation and geometric layout improvements were implemented, usually aimed at smooth operation as well as improving the performance and capacity (Department for Transport, 2009).

2.4 Categories or Types of Signalising Roundabouts

The NCHRP (2010) broadly categorizes roundabout signalization under metering signals and full signalization of the circulatory roadway. However, there are many options for signalization and metering a roundabout. These could be classified under Method/Means of control, Time of operation, and Extent of control / Approach control (The Highway Agency, 2004; Stevens, 2005; Natalizio, 2005; Department for Transport, 2009).

The means of control at a signalized or metered roundabout describes how the signal system controls entering and exiting vehicles. There are two main means of control at a signalized or metered roundabout: direct control and indirect control. A direct means of control affects both external and internal approaches, influencing traffic entering the roundabout as well as vehicles leaving from within the roundabout. For a metered approach, a direct means of control usually only affects vehicles entering the roundabout.

Indirect control affects external traffic at a distance from the entry point of the roundabout. (Natalizio, 2005; Department for Transport, 2009).

The signal system installed at a roundabout may be designed to operate full time or part time. For full-time operation, the installed signals operate permanently and do not turn off during non-peak times. For part-time operation, the installed signal does not operate at all times. The signal is activated by time of day or by detectors (Natalizio, 2005; Department for Transport, 2009).

Approach control describes the number of approaches controlled with a signal or meter. There are two main types of approach control: full control and part control. Full approach control oversees all approaches of the roundabout. Part approach control at a signalised or metered roundabout is defined as control of one or more but not all legs of the roundabout while remaining approaches operate under yield control. Roundabout metering signals which usually control a single approach are considered part control (Stevens, 2005).

In this research three types of signalised roundabouts were analysed. The chosen three incorporated the various options under the categories described above. These are discussed in the sub-sections below.

2.4.1 Fully Signalised Roundabout

Full signalisation refers to the situation where all approaches (external and circulating roadway inclusive) are signalised. By means of control it is direct and can operate full time or part time.

This type fully controls all movements entering and leaving the roundabout (The Highway Agency, 2004; Stevens, 2005; Department for Transport, 2009).

2.4.2 Metered Approach Roundabout

This refers to an indirect means of control of a roundabout where external traffic are controlled but not the circulatory flow. It is also under the partial control of a roundabout.

This implies that one or more of the approaches but not all are signalised (The Highway Agency, 2004; Department for Transport, 2009). Regarding time of operation this type can operate full time or part time. According to the Department of Transport (2009) it is often employed where delays do not occur on all arms. Additionally, it is also possible to use queue detectors on an uncontrolled approach to change stages at a preceding node on the circulating carriageway to produce gaps.

2.4.3 Approach-by-Approach Control

This refers to the situation where right-of-way is assigned to movement from a particular approach at a time. This type indirectly controls circulating traffic. It can also operate either full time or part time (The Highway Agency, 2004; Stevens, 2005; Department for Transport, 2009).

2.5 Characteristic Features of Signalised Roundabouts

a. Fully Signalised Roundabouts

The key parameters for determining the capacity of a fully signalised roundabout model includes traffic flow, critical gap, follow-up time, length of green time and cycle time (Cheng et al., 2016). The critical issue in safeguarding the efficiency of a roundabout with traffic signals is the storage areas dedicated to left-turning movements (Tracz & Chodur, 2012). Tracz and Chodur (2012) further asserted that the internal storage areas can provide varying capacity of left-turning movement depending on the signal cycle length and size of this area, which depends on the central island diameter and the number of traffic lanes within the area.

b. Metered Roundabouts

Generally roundabouts with metering signals assist to create gaps in the circulating traffic to check the challenge of extreme queuing and delays caused by unbalanced flow patterns and high demand flow levels (Akçelik, 2011). Metered Roundabouts tend to increase capacity on roundabouts (Martín-gasulla et al., 2016).

c. Approach-by-Approach Control

A roundabout provided with leg-by-leg traffic control should not be viewed as a robust solution, because of the accrual of unnecessary waiting time (Fortuijn and Salomons, 2015).

2.6 Reasons for Signalising a Roundabout

According to Stevens (2005) unbalanced flow and high circulating speeds are the problems signalising a roundabout check. Benefits of signalising roundabouts are as follows: shorter delays, reduced queue lengths, increase in capacity, reduced accidents (Natalizio, 2005; Stevens, 2005; Chard et al., 2009). In addition, they reported that imbalance of flow and high speed flow of traffic on the circulating carriageway are often responsible for lack of entry capacity in roundabouts. Signalising roundabouts can aid roundabout function more easily and assist entry from approaches.

Furthermore, the Department for Transport (2009) asserts that delay and queue length on a specific approach are as a result of lack of capacity. Hence signalisation can alter the balance of flow to decrease delays and queues on some entries while increasing them on others. Signals do reduce overall delay and queue to the whole roundabout when it is functioning at high degrees of saturation by ensuring efficiency in operation with all lanes and approaches being used to its full potential.

2.7 Relevant Traffic Operation Parameters

2.7.1 Traffic Volume/ Throughput

Traffic volume is the number of vehicles that pass a point on a highway, or a given lane or direction of a highway, during a specified time interval. The unit for volume is vehicle per unit time (HCM, 2000).

2.7.2 Traffic Demand

Traffic demand can be described as the fundamental measure of the amount of traffic using a given facility under some set of conditions. When not constrained by a highway's capacity, the actual flow rate (volume) measured on the highway will equal its demand. However, in cases where highway demand exceeds capacity, some queuing will occur and actual measured flow rates will be less than the demand (HCM, 2000).

2.8 Measures of Effectiveness

Generally, traffic signals create vehicle queues at signalized approaches. Therefore, in order to characterize traffic performance at signalized approaches, several measures of effectiveness (MOEs) can be evaluated. Key among these MOEs used are discussed below.

2.8.1 Delay

Delay measures are usually used for roads and signalized intersections to assess the benefits of operational improvements. Roundabout delay is defined separately for each entry approach. The delay for any entry approach is composed of two distinct components: queuing and geometric delay. Queuing delay occurs when drivers are waiting for an appropriate gap in the circulating traffic. Geometric delay results from vehicles slowing down, when traversing the roundabout (Sofia et al., 2012).

Control delay is defined by the Highway Capacity Manual (HCM, 2000) as the time that a driver spends decelerating to a queue, queuing, waiting for an acceptable gap in the circulating flow while at the front of the queue, and accelerating out of the queue. Delay can be applied to evaluate benefits of signal timing improvements for individual intersections (FHWA, 2007).

2.8.2 Queue Length

Queue measurement are useful for the purpose of detecting hot spots, operations problems at points on a road. At signalized intersections, queue lengths are very important for determining capacity and dimensioning the length of lanes (FHWA, 2007).

According to the NCHRP (2010), length of a queue can also provide further appreciation of the functional performance of a roundabout in contrast with other types of intersections.

2.8.3 Travel Time

The FHWA (2007) describes travel time as the total time for a vehicle to complete a designated trip, over a section of road or from a specified origin to a specified destination.

2.9 Micro-simulation Software Packages

The increasing use of roundabouts to solve traffic flow problems at intersections has produced a great number of models which are able to predict operational performances. Each of these methods allows many important roundabout performance parameters such as capacity, average delay and queue length to be estimated, by the use of empirical or analytical formulations (Kutz, 2003).

Micro-simulation is used in cases where one is interested in the dynamics of the traffic system or if information on microscopic traffic measures is needed. A traffic microsimulation model consists of sub-models that describe driver behaviour. Important behaviour models include; gap-acceptance, speed adaptation, lane-changing, ramp merging, overtakes, and car-following.

Various methodologies and micro-simulation software packages are available for roundabout capacity analysis. These include, but are not limited to, the 2010 Highway Capacity Manual (HCM), SIDRA Intersection, RODEL, Synchro, and VISSIM.

In assessing software packages for analysing roundabout performance, Deshpande and Eadavalli (2011) suggested that most models for analysing roundabout performance are not sensitive to the effects of imbalance in approach volumes. However, SIDRA and VISSIM software seem to account for the effect of one approach volume dominating other approaches. The effect of imbalance is accounted for by reducing the approach capacity. Reduction in the approach capacity would result in the increase in the average delay on the affected approach. Between the two, VISSIM was chosen over SIDRA for this study due its simulation ability.

2.10 VISSIM

As already stated in the section above several s are available for roundabout capacity analysis. VISSIM which is a micro-simulation software package is appropriate for analysing roundabout capacity when a complex network of intersections, driveways, and other factors in the vicinity of the roundabout may impact roundabout operations (Aghabayk et al., 2013). The Suame Roundabout is considered a complex one.

The name VISSIM was derived from *Verkehr In Städten - SIMulationsmodell* (German for Traffic in cities - simulation model) (Aghabayk et al., 2013). It offers a wide variety of urban and highway applications integrating public and private transportation. It also uses a car-following and lane-change logic which allow drivers from multiple lanes to react to each other (Li et al., 2013).

VISSIM gives a flexible platform that allows the user to more realistically model a roundabout. According to the 2000 Highway Capacity Manual methodology, the level of service at an intersection is based upon average control delay on the approaches. The output files for CORSIM provide the control delay for each network link and the total delay for each turn movement. PARAMICS only provides total delay, not control delay, for each link and not for each turn movement. CORSIM and PARAMICS are microsimulation software programs developed by the Federal Highway Administration in the United States and Quadstone Limited, a Scottish company respectively. VISSIM provides two ways to measure delay. The total delay can be measured between any two points in the network, or can be measured for each intersection turn movement using the node evaluation process (Choa et al., 2004).

2.10.1 Coding in VISSIM

It is worth noting that adjustments to the default behavioural parameters are essential to effectively simulate heterogeneous traffic conditions (Siddharth & Ramadurai, 2013). The software has been designed on a link-connector instead of a link-node structure and it is capable of creating an intersection, a corridor or a complete network. Furthermore, VISSIM has the ability to import various image files such as google maps, CAD layout and to set it as a background on which links can be drawn. To ensure that all measurements are in the same units, a suitable scale is assigned. This ensures that the geometric elements of a roundabout (splitter islands, lane width, number of lanes, entry width, etc.) are accurately drawn (Gallelli, 2008). Gallelli (2008) further asserted that, there are three principal features which are very important to set in order for a correct simulation: 1) approach speed, reduced speed zones and circulatory speed; 2) priority rules; and finally, 3) traffic assignment.

2.11 Model Calibration

The calibration is the method applied to achieve acceptable reliability of the model by creating suitable parameter values to enable an accurate replication of real world traffic conditions (ODOT, 2011). In other words, the simulation model has to be calibrated through adjusting model parameters and be validated through comparison with field data before the model can be used for analysis. Model calibration is performed by comparing user experimental conditions from simulation results with observed data from field counts (Kutz, 2003).

WSDOT (2014) recommends the use of traffic volumes and speed/travel times as calibration goals for all traffic models. This is because the mentioned Measures of Effectiveness have strong effect on many other operational characteristics of the transportation network.

Queue lengths are used to ensure correct operations at intersections in the calibration process (ODOT, 2011). Additionally, both ODOT (2011) and WSDOT 2014) played down the need for quantitative comparison of queue lengths with real world conditions. The following parameters were chosen for the purpose of calibration in this study; traffic volume (throughput), delay, queue lengths.

2.12 Gap Acceptance Theory

Usually drivers at an un-signalised intersection are able to join the flow or manoeuvre by evaluating and using an available suitable gap. A gap may be accepted by some drivers and rejected by others. This phenomenon is known as gap acceptance. The parameter most often used in gap acceptance is the critical gap, defined as the minimum time headway between successive major street vehicles, in which a minor street vehicle can make a manoeuvre. If the gap accepted is larger than minimum, then more than one driver can enter the intersection: the time required for an additional vehicle to utilize the same gap in traffic, is defined as follow-up time (Gallelli, 2008). The evaluation of available gaps and the decision to carry out a specific manoeuvre within a particular gap are inherent in the concept of gap acceptance.

2.13 Car Following Models

A car-following model controls a driver's behaviour with respect to the preceding vehicle in the same lane. A vehicle is classified as following when it is constrained by a preceding vehicle and driving at the desired speed will lead to a collision. When a vehicle is not constrained by another vehicle, it is considered free and travels, in general, at its desired speed. In the end, a car-following model should deduce both which regime or state a vehicle is in and what actions it applies in each state. Most car-following models use several regimes to describe the follower's behaviour. VISSIM is based on a particular kind of car-following model known as the Wiedmann Model. Wiedmann Model uses dynamic speeds and stochastic car-following models and, hence, models driving behaviour close to that detected in the field (Park & Qi, 2005).

2.14 Methodology for Traffic Data Collection

The use of video technology for data collection in roundabout research works have become popular. Gazzarri et al. (2012) and Li et al. (2013) have all indicated how they used video technology in their roundabout field data collection for further analysis.

A variant form of the license-plate observation was used in the determination of field travel time and delay data needed for the calibration. The license-plate method requires that observers be positioned at the beginning and end of the test section. Each observer records the last three or four digits of the license plate of each car that passes, together with the time at which the car passes. The reduction of the data is accomplished in the office by matching the times of arrival at the beginning and end of the test section for each license plate recorded. The difference between these times is the traveling time of each vehicle. The average of these is the average traveling time on the test section (TRL,1993;.Garber & Hoel, 2009).

2.15 Capacity

Central to this study is the assessment of the capacity of the Suame Roundabout. The capacity of a facility is the maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions. This is usually expressed as vehicle per hour, or persons per hour (HCM, 2000).

2.16 Conclusion

Citations discussed in this chapter have shown the importance researchers elsewhere in the world have attached to studies relating to roundabouts in general. The Highways Agency (2004) asserts that most signalised roundabouts are not alike and, as a consequence general guidance only can be provided on the design for various forms of signalised roundabouts. Again, Gap acceptance models are seriously affected by driver behaviour and local habits (Gazzarri et al., 2012). Considering the fact that the culture of driving in Ghana and that in United Kingdom are not the same, results of similar studies in these countries may differ. Hence it is imperative for such studies to be undertaken locally so that the necessary adaptations will be taken to suit the Ghanaian environment. It is vital to reiterate that signalization is only one of many solution options available. Therefore further dedicated research will reveal several other solutions.

CHAPTER 3: METHODOLOGY

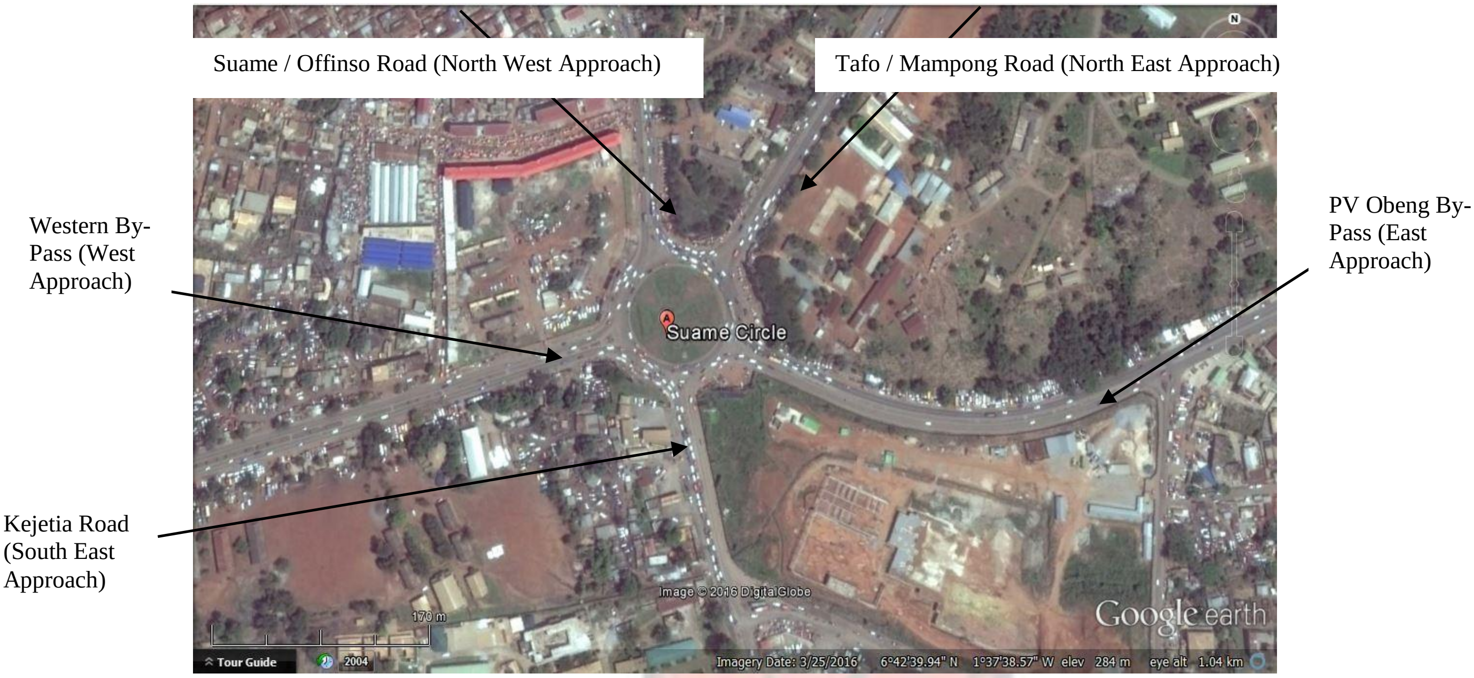
3.1 Site Selection and Description

The Suame Roundabout was selected as the site for the project. This intersection has relatively high traffic volumes and congestion during both morning and evening peak periods. The roundabout is also characterized by long queues and experiences high delays on all approaches during peak periods and is common to observe a gridlock traffic situation. It is very common to find the Motor Traffic and Transportation Police Department (MTTD) staff controlling flows manually.

According to the Highway Agency (2004), physical constraints imposed on the circulating carriageway link lengths between each signal-controlled approach makes small roundabouts unsuitable for signalisation. The Suame roundabout has a 110m inscribed circle diameter and a 15m circulatory roadway with an approximate average of 58m distance between nodes for vehicular storage in the circulatory area making it suitable for this study. Additionally, it is worth noting that the circulatory roadway of the roundabout is not marked, but for this research, it was assumed to have of three lanes.

The roundabout is a five legged intersection with two entry/exit lanes for each leg. The circulatory area is not marked but, often three lanes of traffic are observed using the roundabout. The approaches consist of the Suame / Offinso Road (North West Approach), Tafo / Mampong Road (North East Approach), while the P.V. Obeng By-pass joins the roundabout from Krofrom (East Approach) and the Western By-Pass (West Approach) joins from the Abrepo Junction end. The Kejetia Road is the South East Approach. Figure 3.1 is the aerial view of the roundabout.

Figure 3.1. Google Image of Suame Roundabout



3.2 Data Collection

Preliminary site visit revealed a difficulty in finding an appropriate elevated observer position to mount a video camera for filming the entire roundabout. Therefore field data collection methods described below were used.

3.2.1 Traffic Volume and Classification Count

A manual traffic volume and classification survey was carried out to collect the required approach volume data at the roundabout. Trained observers positioned at all the approaches tallied (counting and classifying) vehicles as they passed in a time interval of 15 minutes. Queued vehicles on each approach after every 15 minute count were recorded and added to the volume to determine the traffic demand. The vehicles were classified into three; 1.Small (saloon cars, taxis, pick-ups, cross country vehicles/ sport utility vehicles), 2. Medium Vehicles (mini-buses, buses), and 3. Trailers / Heavy (trucks, large buses and other heavy goods vehicles) (GHA, 1991).

This exercise was undertaken during the morning peak period between 6:30am and 9:30am on the 14th of April 2016. Evening peak values were not taken because of poor lighting at the site and presence of police which interfered with the flows.

3.2.2 Travel time and delay studies

The purpose of the travel time studies was to be able to determine average approach delay and free flow travel time for calibrating the VISSIM model. A vehicle license-plate technique was used to determine the travel time over a section of length 140m on the Kejetia approach. The section of road was from the yield line to a point upstream. Care was taken to ensure that this did not interfere with the nearest intersection. The observer measured the time it took for a randomly selected vehicle to travel from the start (upstream point) to the end point (yield line) during the morning peak period while the traffic volume count was on-going.

To guide this research in terms of minimum sample size, confidence interval for the means of travel times and delay required, an initial pilot sampling was undertaken. Six morning peak period travel time and five free flow travel time data were collected along the same section on the Kejetia leg as the main travel time and delay studies. The pilot survey was carried out on 12th April, 2016 prior to the main work. The morning peak period travel time data was collected between 7:00am and 9:00am while that for the free flow travel time was undertaken between 1:00pm and 2:00pm.

Based on the analysis of the pilot sampling data, twelve vehicles were sampled for the travel time measurement. A similar exercise was carried out during an off peak period when traffic was free flowing. Average approach delay was determined as follows:

$$D = T_p - T_o \qquad (3.1)$$

Where D = Average delay

T_p=Average Travel time during peak period

T_o=Average Travel time during off-peak period

3.2.3 Queue Length Measurement

Queue Length was measured at the upstream of the yield line on the Kejetia Approach at intervals of 15minutes. Markers were made along the edge of the road at intervals of 30m for the queue measurement used for the calibration.

3.3 Geometric Data

Geometric data such as approach road widths (7m for all approaches), width of circulatory roadway (15m), inscribed circle diameter (110m) and the diameter of Central Island (80m) were also measured at the site and verified from computer aided design drawing derived from google earth data.

3.4 Data Reduction and Analysis

The traffic volume data collected were converted into passenger car units (pcu) using passenger car equivalent (PCE) values of 1.0 for Car, 1.7 for Medium and 2.5 for Heavy vehicles from traffic studies undertaken in Kumasi (Adams and Obiri-Yeboah, 2008). Turning movement proportions for trip assignment were obtained from a previous traffic study conducted by BCEOM/ACON (2005). Even though the data from the study is old, preliminary studies at the site showed that though volumes were not the same the proportions were similar. Data obtained from the travel time, delay and queue length measurements were used in the calibration process.

As a quality control measure various tests were conducted to assess the statistical relationships between field and calibrated data. These included GEH statistic for the throughput and queue length, and Analysis of Variance test for delay and travel time data.

3.5 Calibration

A Google layout image of the roundabout was imported into the VISSIM software and set as background on which the VISSIM links and connectors were drawn. Suitable scale and units were entered so that all the measurements were in the same units. The number of lanes, and lane widths were specified while drawing the links and connectors. Desired speeds on the approaches were set at 50km/h at start of the network.

Reduced speed and priority rule parameters which are also vital for modelling a roundabout in VISSIM were coded. Figure 3.2 below shows locations within the VISSIM model where reduced speeds and priority rules were applied.

Figure 3.2. VISSIM model showing Reduced Speed Areas and Priority Rules

Sections in Figure 3.2 indicated in yellow markings show the reduced speed areas. The reduced speed areas on the circulatory roadway were 20km/h, 15km/h and 12km/h for the inner, middle and outer lanes, respectively, except sections in the outer lanes near the Suame and Tafo exit lanes, which were assigned 5km/h. The change was done in order to mimic the congestion which occurs due to the persistent bus (*trotro*) stopping along those exit lanes. The 5km/h reduced speed areas coding extended 40m and 60m into the Tafo and Suame outer exit lanes respectively meanwhile 40m into the inner lane of the latter was also assigned 15km/h reduced speed. The subject approach (Kejetia) lanes were assigned 12km/h reduced speed, however, the remaining approaches were assigned 20km/h reduced speed areas near their respective yield lines.

VISSIM priority rule consists of two basic parameters – minimum gap time and minimum headway. The priority rule consisted of one stop line (red) and corresponding two conflict markers (green marks) that are associated with the stop line (see Figure 3.2). Table 3.1 shows the priority rule data used for the calibration. T_{gi} and H_{mi} represent minimum gap time and minimum headway, respectively, which were the priority rule parameters used. These data were obtained by trial and error.

Table 3.1. Priority Rule Data Used in the Calibration

	Minimum Gap Time (seconds)				Minimum Headway (metres)			
Approach	T_{g1}(inner lane)	T_{g2}(inner lane)	T_{g3}(outer lane)	T_{g4}(outer lane)	H_{m1} (inner lane)	H_{m2} (inner lane)	H_{m3} (outer lane)	H_{m4} (outer lane)
Kejetia (SEA)	2.6	2.6	2.6	2.6	3.6	3.6	3.6	3.6
Krofrom (EA)	2.6	2.6	2.4	2.4	3.5	3.5	3.5	3.5
Tafo (NEA)	2.5	2.5	2.6	2.6	3.6	3.6	3.6	3.6
Suame (NWA)	2.3	2.3	2.3	2.3	3.5	3.6	3.5	3.5
Abrepo (WA)	2.6	2.6	2.6	2.6	3.9	3.9	3.7	3.7

Vehicle routes which were essential for traffic assignment and movements were coded. The processed traffic volume demand data were entered as well.

3.5.1 Error Checking

The error checking process included a verification of VISSIM inputs, a review of the animation, and a correction of the VISSIM error files.

3.5.2 VISSIM Model Runs

Microscopic simulation models are stochastic models whose results tend to change depending on the random seed number used. It was, therefore, important to run each model multiple times and subsequently find the average of the results. For these models, each scenario was run 12 times in order to match results of the initial sampling and confidence interval.

3.6 Simulated Output Measurement and Analysis

The students' version of VISSIM 7.0 obtained from PTV was used for this study. This version has the limitation of being able to run a simulation for only a maximum period of 600 seconds. There was a warm up period of 300 seconds prior to data measurement. The data collection tool was used to mark the yield lines on all approaches to enable the collection of throughput. The throughput was calculated by converting the counts obtained during second period of 300 seconds to an hourly volume.

With the help of the evaluation tab on the menu toolbar the relevant output attributes such as travel time and delay, throughput, were selected and recorded for further analysis.

3.6.1 Measurements from Calibrated Model

As described earlier, the existing traffic situation at the roundabout was modelled and calibrated on the following Measures of Effectiveness (MOE); travel time approach delay, free flow travel time, throughput and queue length. Travel time and delay measurements were done by the creation of a travel time section from the yield line to a point 140m upstream of the Kejetia Approach to mimic the field measurement. Queue counters were also placed at the yield lines to measure queue length.

3.6.2 Measurements from Signalised Models

Throughput was measured in the same manner as was it in the calibrated model. The average queueing delays were measured with travel time sections marked from the following distances upstream of their corresponding yield line: Kejetia – 140m; Krofrom – 230m; Tafo – 165m; Suame – 154m; Abrepo – 180m. Queue lengths of the approaches in the various modelled scenarios were also measured. The results obtained from the calibrated model is also referred to as Existing Model results in the discussion section of this document.

3.7 Proposed Signalised Roundabout Models

The North-West (Suame) and South-East (Kejetia) approaches were categorized as major approaches since traffic demand from them were much higher than those from the other approaches. Consequently, the other approaches, namely; North-East Approach (NEA), East Approach (EA), and West Approach (WA) were considered as minor.

The following guided the determination of signal timing for the various options with modifications where necessary.

- According to NHCRP (2015) and FHWA (2013), long cycle lengths tend to increase delay and queueing.
- Department of Transport (2009) recommends a minimum and maximum cycle lengths of 60s and 90s, respectively, to ensure optimum flow.
- In order not to violate expectations of drivers at a signalised intersection, NHCRP (2015) recommends minimum green times for major arterials (≤ 65km/h) ranging between 7s and 15s.

- Furthermore, yellow time of 3s were used where applicable (FHWA, 2013).

- In designing the various scenarios, the cycle and green times were obtained by optimizing the simulation results through many trials.

Three options for signal phasing were explored; Approach-by-Approach Control Model, Metered Approach Model and Full Signalisation Model.

3.7.1 Approach-By- Approach Control Model (Leg by Leg Control)

This was aimed at assigning the right-of-way to movement from a particular approach at a time. The VISSIM model for this type was designed using one signal controller with five signal groups. Traffic signal lights were assigned to only the five approaches (indirect). Figure 3.3 shows labelled signal groups and numbered signal heads at the various entry approaches.

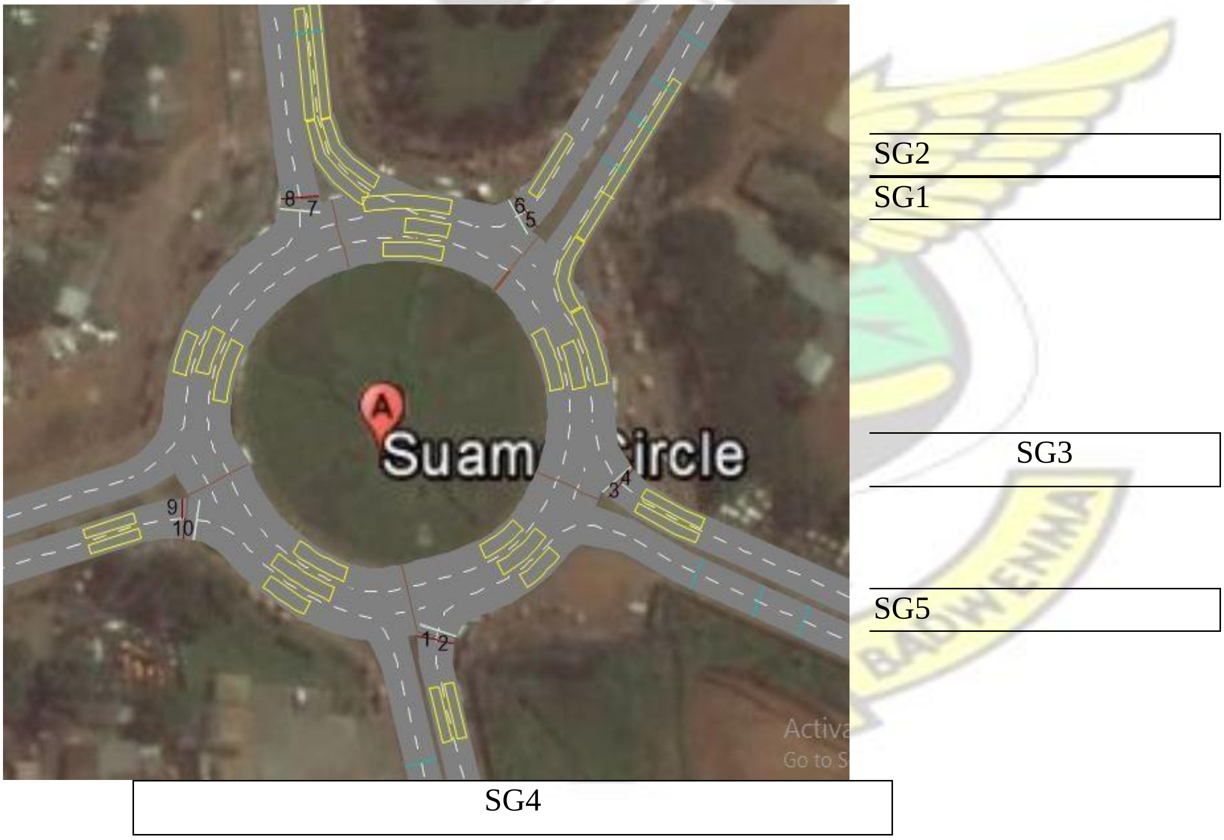


Figure 3.3. Approaches with Signal Groups Labelled SG1 to SG5

The phasing plan was such that one approach movement was allowed at a time with the sequence of movement being clockwise.

A cycle length of 60s was used. The NWA, SEA and EA were assigned 12s, 11s and 8s green times respectively while the green times for the WA and NEA were 7s each. Details of estimation of signal timing have been provided in Appendix B, Section B-1. Figure 3.4 displays VISSIM's signal controller interface showing the cycle time and other time allocations for the various phases.

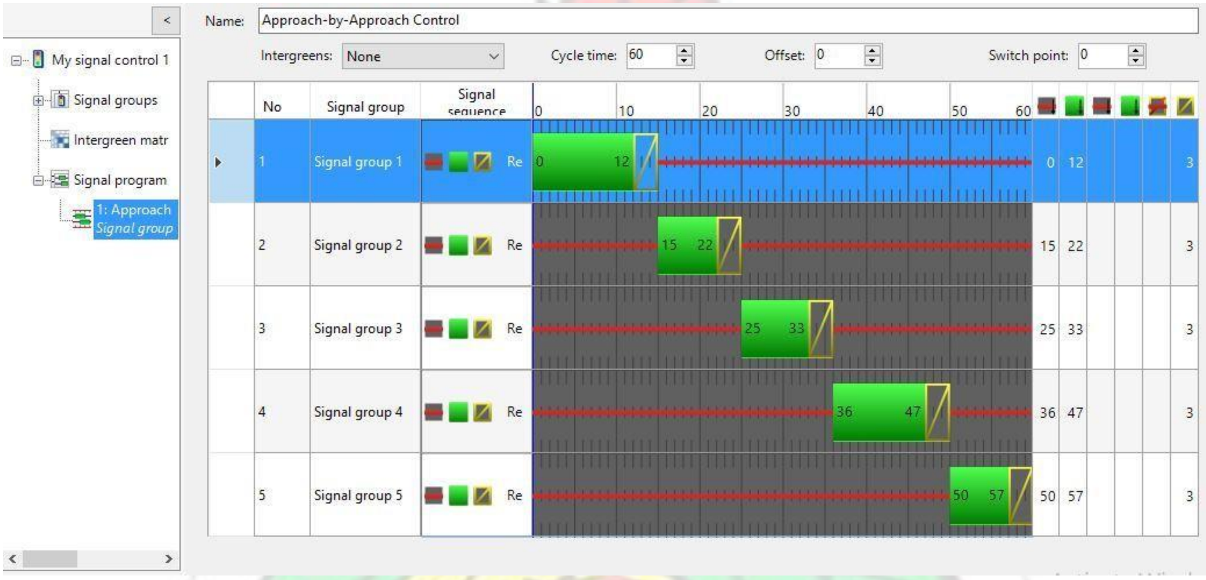


Figure 3.4. Signal Controller Interface for the Approach-by-Approach Control Model

Additionally, Figure 3.4 also shows the sequence of movement for each approach. The Suame, Abrepo, Kejetia, Krofrom and Tafo approaches were assigned to signal groups 1 to 5 and also allocated the right-of-way in the order. Details of the signal groups with corresponding signal heads and approaches are in Table 3.2.

Table 3.2. Signal Groups with their Corresponding Signal Heads

Approach	Signal Group	Signal Head
Suame	SG 1	7,8
Abrepo	SG 2	5,6
Kejetia	SG 3	3,4
Krofrom	SG 4	1,2
Tafo	SG 5	9,10

3.7.2 Metered Approach Model

Its design involved the partial control of the roundabout (signalisation of only three approaches). In VISSIM, the model was designed with the traffic signal lights positioned at a location 40m at the upstream of the entry yield lines. In this case only the minor approaches were signalized. The modelled metered design is as shown in Figure 3.5.

Figure 3.5 Model with Metered Approaches (West, East and North East)

Only two signal aspects (green and red) were used. The phasing plan design involved the use of one signal controller with one signal group assigned to all three approaches (see Figure 3.6 below).

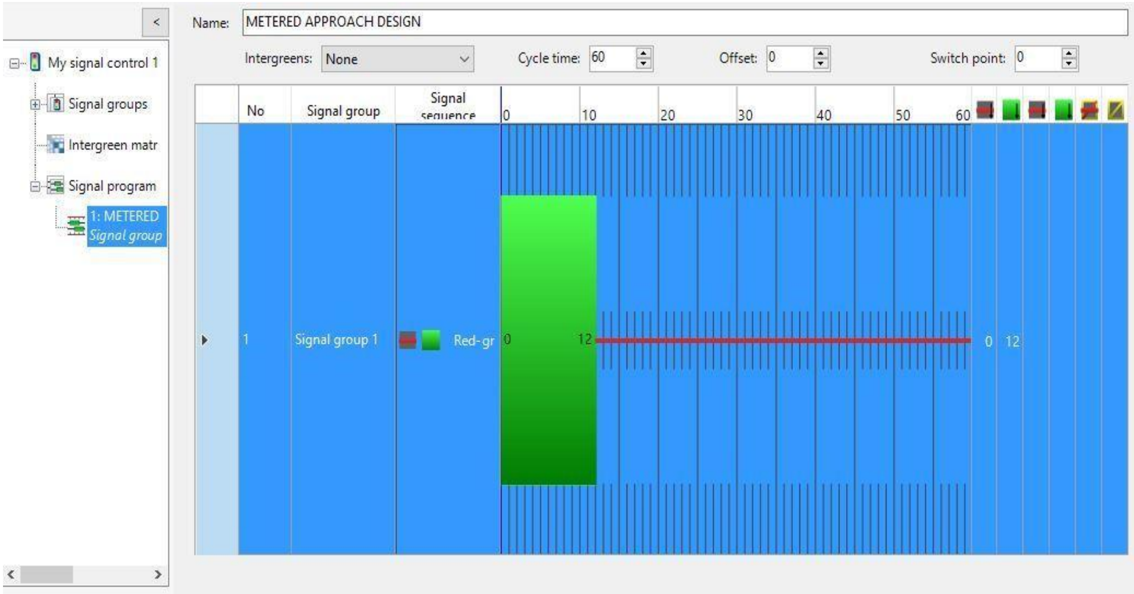


Figure 3.6 Signal Controller Interface of the three Metered Approach Design

In this single phased signal timing design, all three approaches were assigned with 12s of green time in a cycle time of 60s. Even though vehicular traffic from three approaches were signal controlled, entry into the circulatory roadway was controlled by priority rule (gap acceptance). Vehicles in the circulating roadway had priority. This meant that traffic passing through the signals was still required to give way to traffic in the circulatory roadway, see Appendix B for estimation of green time.

3.7.3 Full Signalisation Model

The full signalization model was designed to ensure co-ordinated traffic movements within the roundabout by the provision of signals on all approaches as well as the circulatory carriageway. The cycle time was 60s and green time for the major and minor approaches were 15s and 10s respectively. The signal controller interface displaying signal groups with green time, amber and red times as well as cycle time is shown in Figure 3.7.

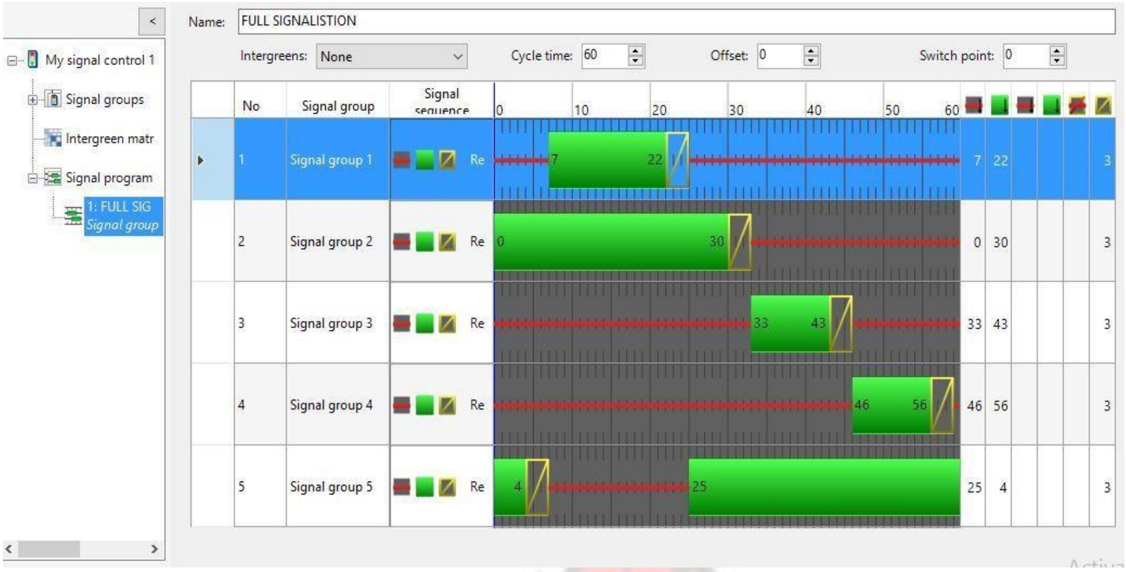


Figure 3.7 Signal Controller Interface of the Full Signalisation Design

Movements from the major approaches were assigned to Signal Group 1, while movements from West (Abrepo) and North East (Tafo) Approaches were assigned to Signal Group 3. Signal Group 4 controlled movement from the East (Krofrom) Approach. The sequence of the movements was such that traffic discharge from the major and minor approaches was done consecutively. The overlapping phases assigned to Signal Groups 2 and 5 were provided to move traffic stored within the circulatory roadway area. The green times assigned to Signal Groups 2 and 5 were 30s and 39s, respectively. Vehicles in the circulating lanes could also use the immediate exit on red. Figures 3.10 to 3.12 and Table 3.3 show the sequence of movement within the roundabout while Table 3.2 displays signal groups together with the signal heads they control.

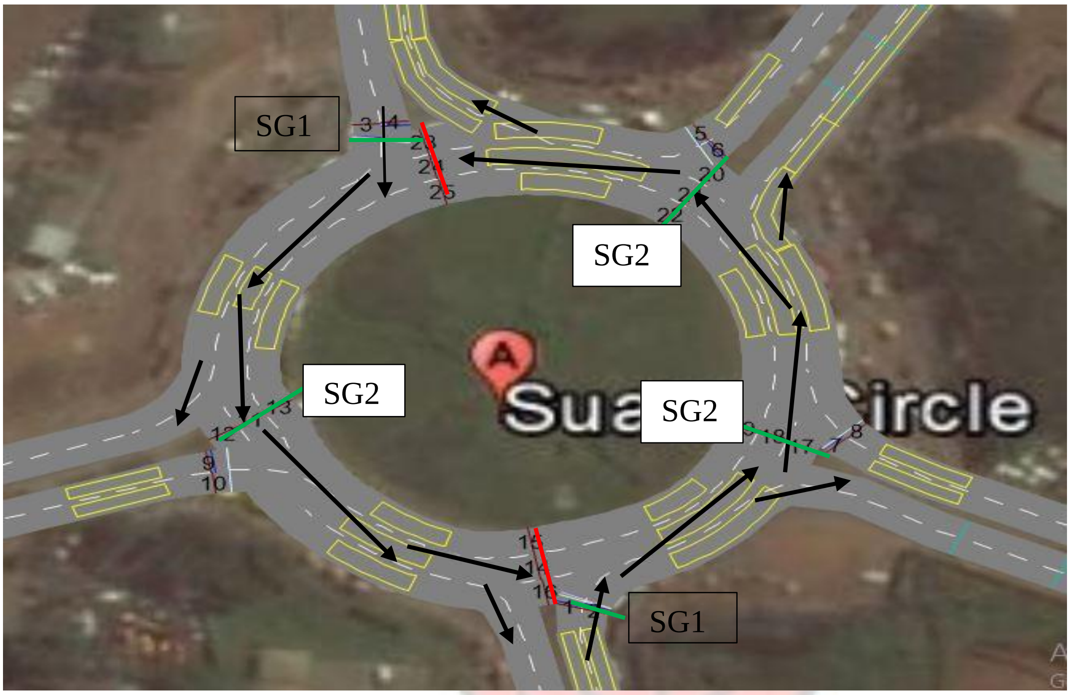


Figure 3.8. Movement of Major Approaches Traffic

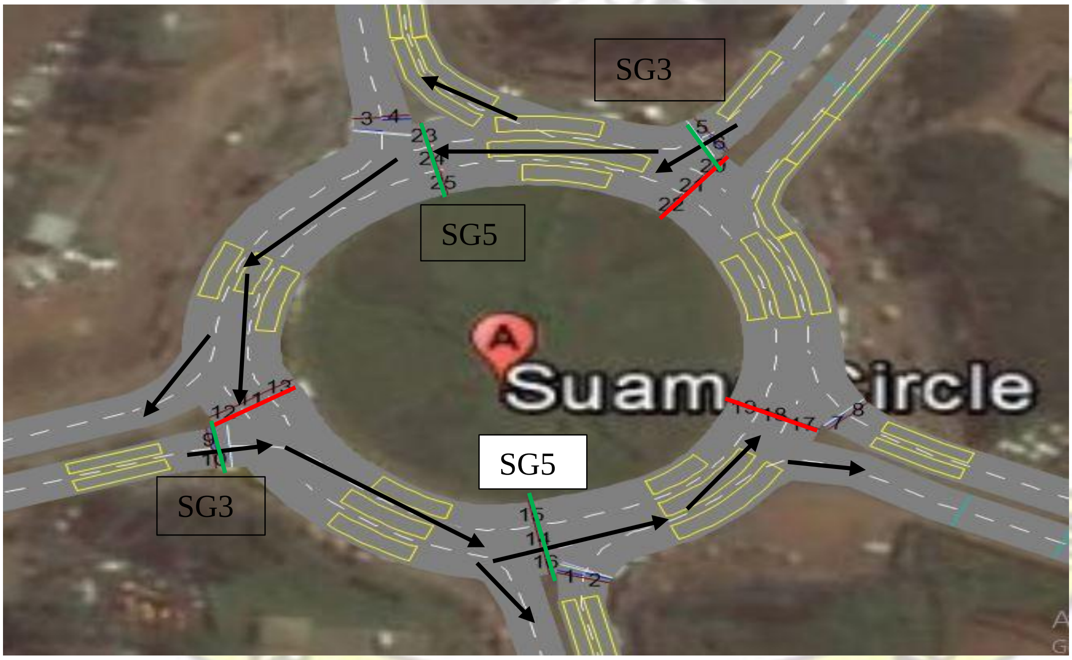


Figure 3.9. Movement of Minor (East and North West) Approaches Traffic

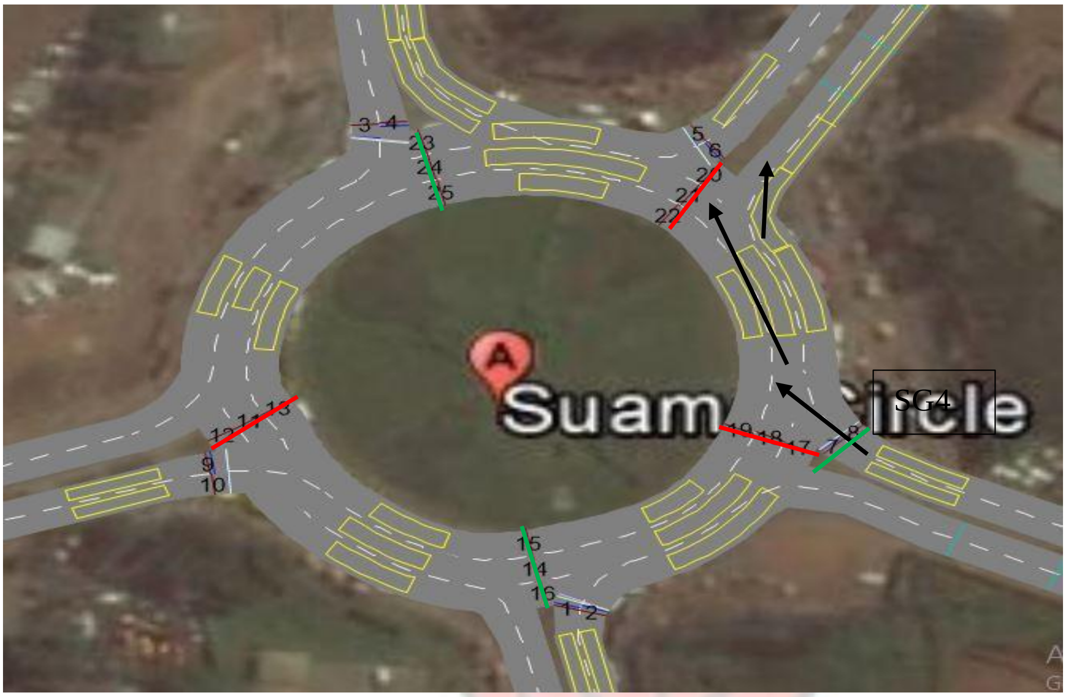


Figure 3.10. Movement of Minor (West) Approach Traffic

Table 3.3. Signal Groups with Corresponding Signal Heads

Item No.	Signal Group (SG)	Signal Head
1	SG 1	1,2 / 3, 4
2	SG 2	11, 12, 13 / 17, 18, 19 / 20, 21, 22
3	SG 3	5, 6 / 9, 10
4	SG 4	7,8
5	SG 5	14, 15, 16 / 23, 24, 25

CHAPTER 4: RESULTS AND DISCUSSION

4.1 Traffic Demand

The results of the field traffic demand and throughput are presented in Table 4.1.

Table 4.1. Field Traffic Demand and Throughput

Approach	Throughput (pcu/h)	Traffic Demand (pcu/h)	Degree of saturation (v/c)
Kejetia (SEA)	843	1069	1.3
Krofrom (EA)	715	879	1.2
Tafo (NEA)	644	778	1.2

Suame (NWA)	977	1211	1.2
Abrepo (WA)	649	769	1.2

An assessment of the field demand to maximum flow rate (throughput) as shown in Table 4.1 indicates that the roundabout is oversaturated making traffic flow very unstable (v/c>1.00).

4.2 Confidence Intervals and Model Runs

In order to improve statistical accuracy in the comparison of the results of the model runs and the field data, pilot sampling was undertaken and analysed. The results for the off peak or free flow travel time are presented in Table 4.2.

Table 4.2. Pilot Free flow Travel Time

Number	Free flow Travel Time (s/pcu)
1	15
2	14
3	18
4	13
5	16

Standard Deviation = 2.01s/pcu

Mean = 15 s/pcu

Considering 95% confidence level and confidence interval of ±1.65 s/pcu the expected mean free flow travel time lies between **13.5** s/pcu and **16.8** s/pcu. The minimum sample size/ model runs was determined by evaluating Equation (4.1) (FHWA, 2004) below:

$$CI = 2 \times t_{(1-\alpha/2),N-1} \times S/\sqrt{N} \qquad (4.1)$$

N= 11.46 ≈ **12**

Where;

CI is the Confidence interval

N is Minimum sample size/ model runs S is Standard Deviation $t_{(1-\alpha/2),N-1}$ is the Student's t-statistic for the probability of a two-sided error summing to alpha with N-1 degrees of freedom.

The results of pilot travel time and delay for morning peak are presented in Table 4.3. This was performed to determine the confidence interval for the mean approach delay. **Table 4.3. Pilot Delay and Travel Time**

Number	Travel Time (s/pcu)	Delay (s/pcu)
1	88	73
2	159	144
3	157	142
4	141	126
5	144	129
6	150	135

Where;

Mean Approach delay =125 s/pcu

Standard Deviation = 26.35 s/pcu

Using Equation (4.1), CI for approach delay = 39.1≈ 40s/pcu i.e. ±20 s/pcu.

The mean of the approach delay was expected to lie between **105** s/pcu and **145** s/pcu.

4.3 Calibration Results

4.3.1 Throughput

The FHWA (2004) recommends the use of the GEH statistic to compare field volume data with that of simulated outputs. It also recommends GEH statistic acceptability threshold of less than 5 for individual links (FHWA, 2004).

The data in Table 4.4 show the simulated throughput.

Table 4.4. Results of the Simulation of Traffic Demand data

Simulation Runs	Throughput (pcu/h)				
	Kejetia	*Krofrom*	*Tafo*	*Suame*	*Abrepo*
1	774	594	546	696	624
2	744	576	366	912	552
3	726	576	360	864	585
4	648	570	426	810	582
5	894	420	342	852	588
6	762	552	417	726	606
7	804	461	488	886	504
8	840	546	420	714	582
9	724	580	515	778	540
10	708	573	480	872	470
11	714	506	426	980	418
12	678	510	306	906	618
MEAN	***751***	***539***	***424***	***833***	***556***

A comparison was made between the mean simulated throughputs and the field data based on their GEH statistic. The results obtained is presented in Table 4.5.

Table 4.5. Comparison of Field Throughput data with Simulated Output

	Kejetia	***Krofrom***	***Tafo***	***Suame***	***Abrepo***
Field (pcu/h)	843	715	644	977	649
Simulation (pcu/h)	751	539	424	833	556
GEH	***3.14***	*7.04*	*9.50*	***4.79***	***3.80***

The results shown in Table 4.5 indicate that the GEH statistic for all the approaches, except the North East Approach (Tafo) and East Approach (Krofrom), met the acceptability threshold (GEH <5). For the other approaches, GEH<10, since these were not the target approaches for the calibration (FHWA, 2004).

4.3.2 Delay and Travel Time for Target Approach

The mean field and simulated approach delay on the Kejetia Approach (SEA) were calculated as 128 s/pcu and 122 s/pcu, respectively. Table 4.6 shows results both field and simulated travel time and delay.

Table 4.6. Field and Simulated Travel Time and Delay Data (Kejetia Approach)

Sample Size/ Runs	Field Data	Simulation Data		
	Approach Delay (s/pcu)	Travel Time (s/pcu)	Free Flow Travel Time (s/pcu)	Delay (s/pcu)
1	94	134	14	120
2	113	121	14	107
3	141	144	14	130
4	128	175	14	161
5	124	99	14	86
6	122	121	14	107
7	146	168	14	154
8	136	181	14	167
9	125	169	14	155
10	119	150	14	136
11	141	120	14	107
12	149	42	14	28
MEAN	***128***	***135***	***14***	***122***

A test was undertaken to determine whether there was a significant difference between the simulated and field delays.

Table 4.7 shows the one-factor Analysis of Variance test results for the field and simulated approach delays.

Table 4.7. One- Factor Analysis of Variance (ANOVA) Test Results for Field and Simulated Approach Delay Data

Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	283.6627	1	266.6667	0.302146	0.588078	4.30095
Within Groups	19416.67	22	886.5758			
Total	19683.33	23				

From Table 4.7, p-value (0.588) > than the significance level (0.050) while F (0.3021) < Fcrit. (4.301). This implies that the difference between the means of the field and simulated approach delays was not significant. This lead to the failure to reject the fact that means of the approach delays are equal. Furthermore, both the field and simulated approach delay were within the confidence interval (105 s/pcu and 145 s/pcu) calculated at the pilot sampling stage. Additionally, the mean of the simulated free flow travel time was 14s/pcu. This value was considered acceptable since it was within the confidence interval (13.5s/pcu to 16.8s/pcu) estimated from the pilot sample analysis. The model was therefore considered calibrated with respect to approach delay and travel time.

4.3.3 Queue Length

The measured average queue length of the subject approach in the field and calibrated existing model were 126m and 106m, respectively. Using the GEH statistic to compare the two, the result was 1.86. The model was, therefore, considered calibrated with respect to queue length of the subject approach. In this case, a GEH statistic threshold value of less than or equal to 3 was applied. Results of queue measurement for the target approach in the field has been displayed in Table A -6 in Appendix A.

4.2.4 Summary of Results from Existing Model

A Summary of results from the existing situation model used for the discussion has been presented in Table 4.8.

Table 4.8. Summary of Results from Modelled Existing Situation at the Roundabout

Approach	Throughput (pcu/h)	Average Queueing Delay (s/pcu)	Average Queue Length (m)
Kejetia (SEA)	751	122	105.93
Krofrom (EA)	539	157	98.06

Tafo (NEA)	424	230	104.14
Suame (NWA)	833	118	126.7
Abrepo (WA)	556	123	100.51

Data in Table 4.8 was used in comparing with results of the signalised roundabout options in the sub-section below.

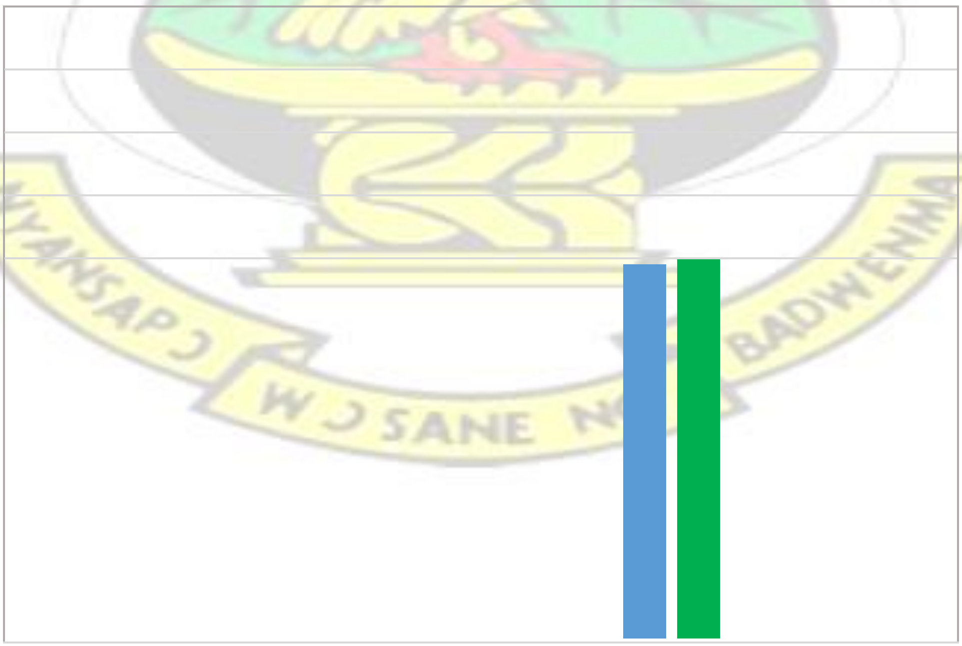
4.4 Results from the Proposed Signalised Roundabout Scenarios

This section discusses the results of the proposed signalized roundabout models as compared with the calibrated existing model. The results are presented in three sections for capacity, delay and queue length for the different options and compared to the unsignalised condition.

Data for plotting the graphs and discussions in the sub-sections below can be found in Appendix B (Table B-1, Table B-2, Table B-3, and Table B-4)

4.4.1 Capacity

This sub-section discusses the maximum throughput results obtained from running the various models in VISSIM. Throughput of the Approach-by-Approach Control Model as compared with that of the existing model are as shown Figure 4.1.



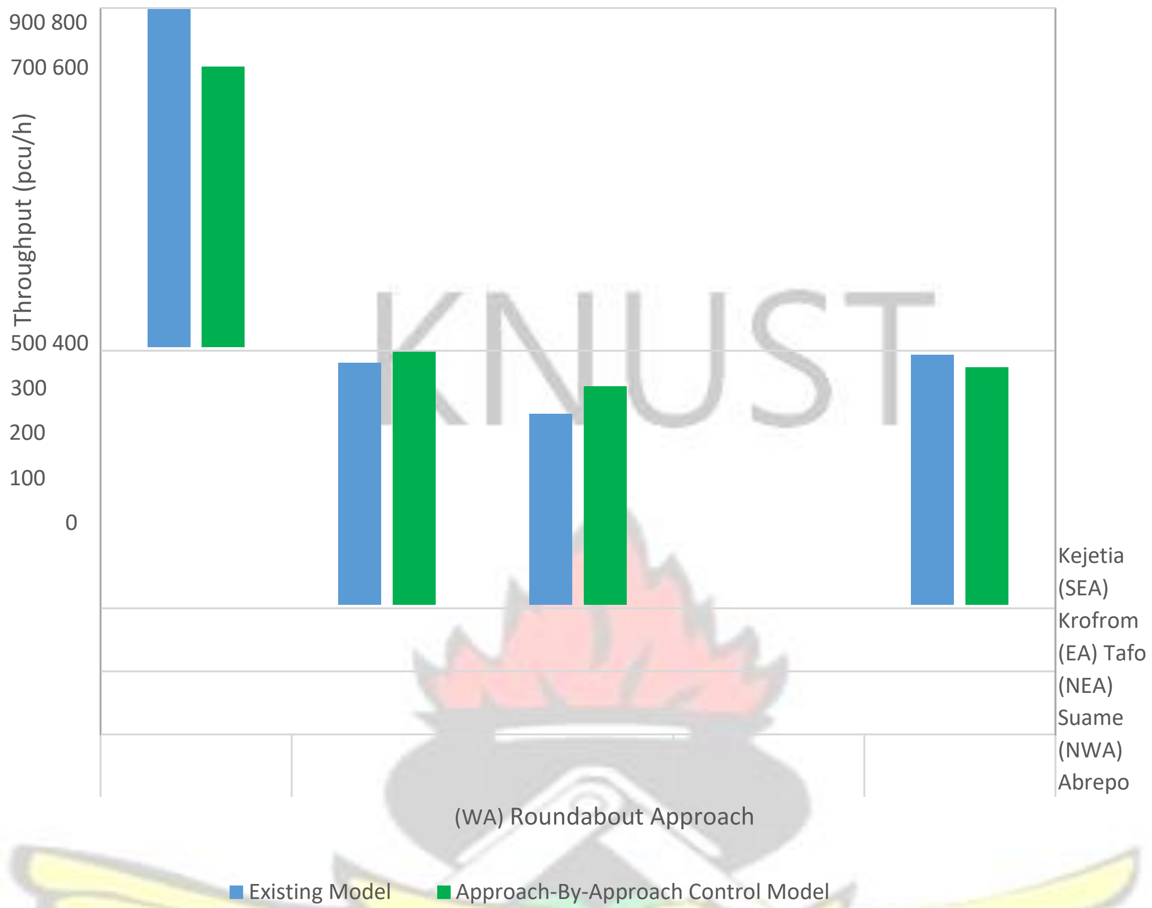


Figure 4.1.Comparison of Throughput from Existing Model and Approach-by-Approach Control Model

From Figure 4.1, the results show some reduction in approach throughput for the major approaches (Kejetia and Suame) from the Approach-by-Approach Control Model as compared to the existing situation. The throughputs for the minor approaches from the Approach-by-Approach option are marginally better than those of the Existing Model. In effect, the Approach-by-Approach Control Model does not provide improvement in capacity over the existing situation for a cycle length of 60s.

The throughput values from the Metered Approach Model option compared with that of the Existing Model are displayed in Figure 4.2 below.

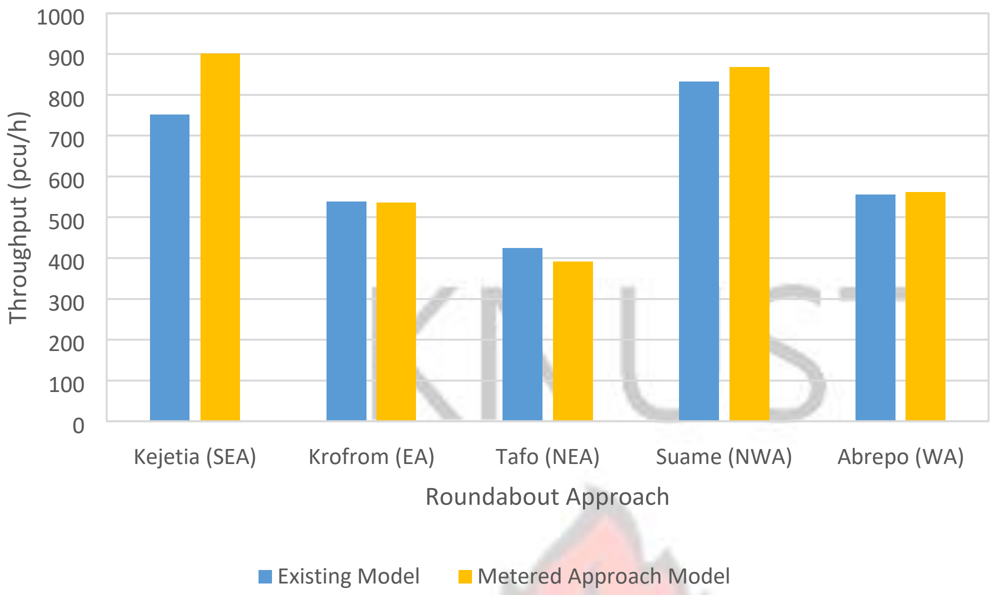


Figure 4.2. Comparison of Throughput Results from Existing Model and Metered Approach Model

In this option, only the minor approaches were signalised. Each traffic signal head had green time of 12s and 48s of red indication.

The results show some improvement in approach throughput for all the major approaches (Kejetia and Suame) from the Metered Approach Model as compared to the Existing situation. This could be attributed to the fact that in the Metered Model option major approaches flows were unimpeded by signalisation. Compared to the Existing Model, the reduction in throughput from the Krofrom and Tafo approaches in the Metered Approach Model was ascribed to the metering on those approaches while the slightly higher throughput from the Abrepo Approach was due to change in the balance of flow resulting from the metering. The metering of the minor roads means that more traffic could traverse the intersection from the two major flows from Kejetia and Suame. Consequently, when the signal turned green, the minor approaches traffic could find gaps relatively easily which resulted in improved throughput compared to the Approach-by-Approach Control option. Additionally, the 60s cycle length was long enough for them to utilise gaps in the circulating traffic.

In the case of the full signalisation option, the cycle length was 60 s. There were three main phases and five signal groups. The results are presented in Figure 4.3.

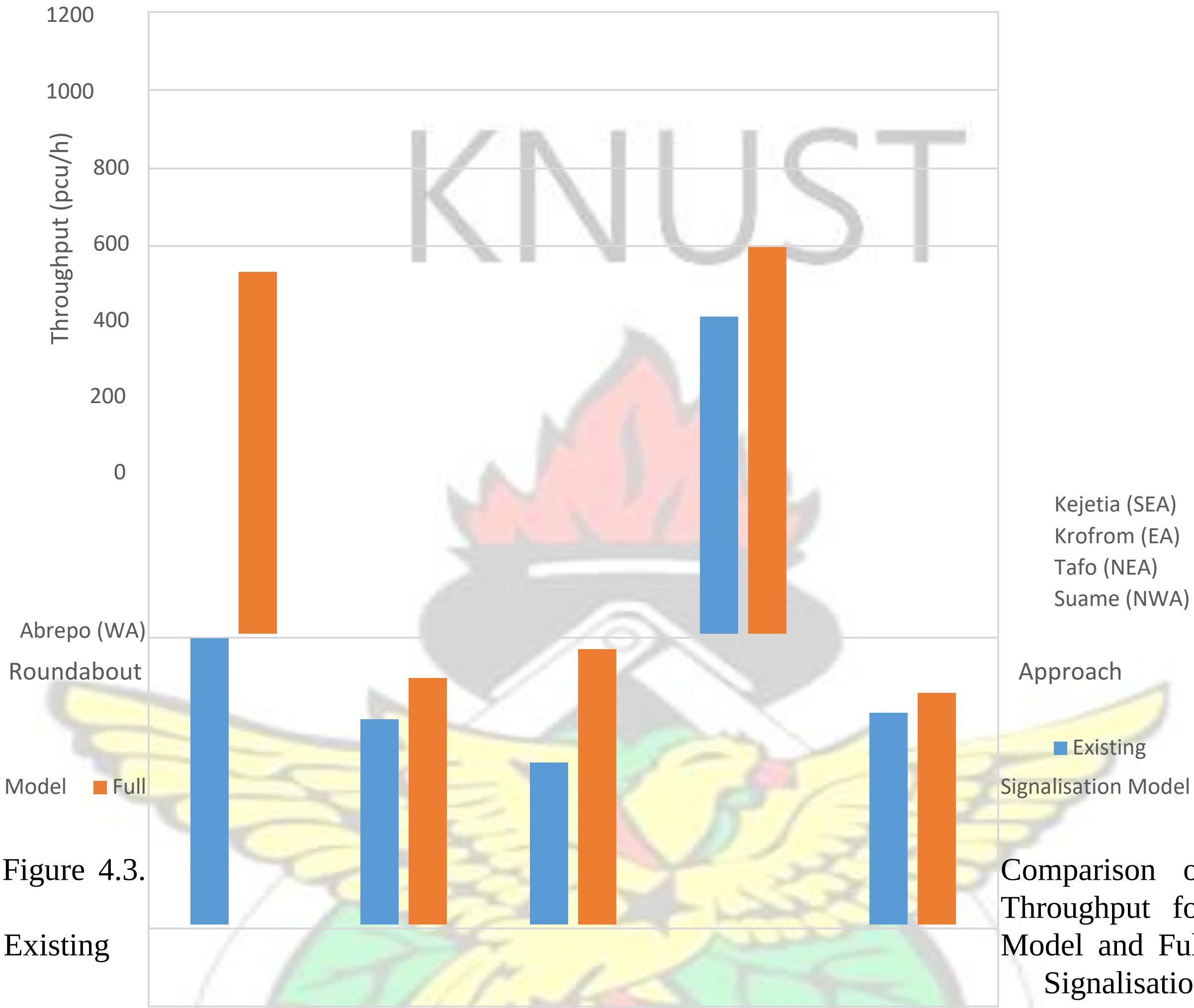


Figure 4.3. Comparison of Throughput for Existing Model and Full Signalisation Model

In this option, all approaches recorded increased flow compared to the un-signalised option. This means that the entire roundabout under a full signalisation option, improves the capacity. A summary of the performance (capacity) of the Existing, Approach-by-Approach and Full Signalisation is presented in Figure 4.4.

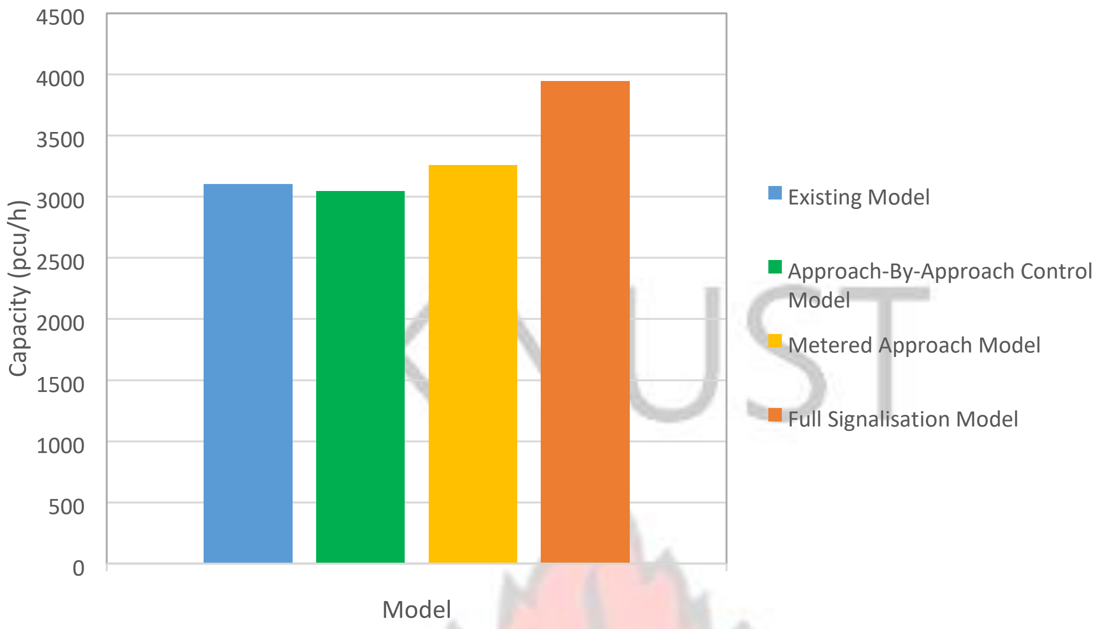


Figure 4.4. Capacity Results from all Models

It is observed that the Full Signalisation Model produced the best result with a capacity increase of 27% over the Existing Model followed by the Metered Approach Model (5% increase in capacity over the Existing Model). The capacity results generated from simulation runs of the Approach-by-Approach Control Model was the worst i.e. it was 2% less than that of the Existing Model.

4.4.2 Average Queueing Delay

This sub-section discusses the results of the average delay from each model for approach. Figure 4.5 displays the average delay results from the Approach-by-Approach Model as compared with displays from the Existing Model.

250

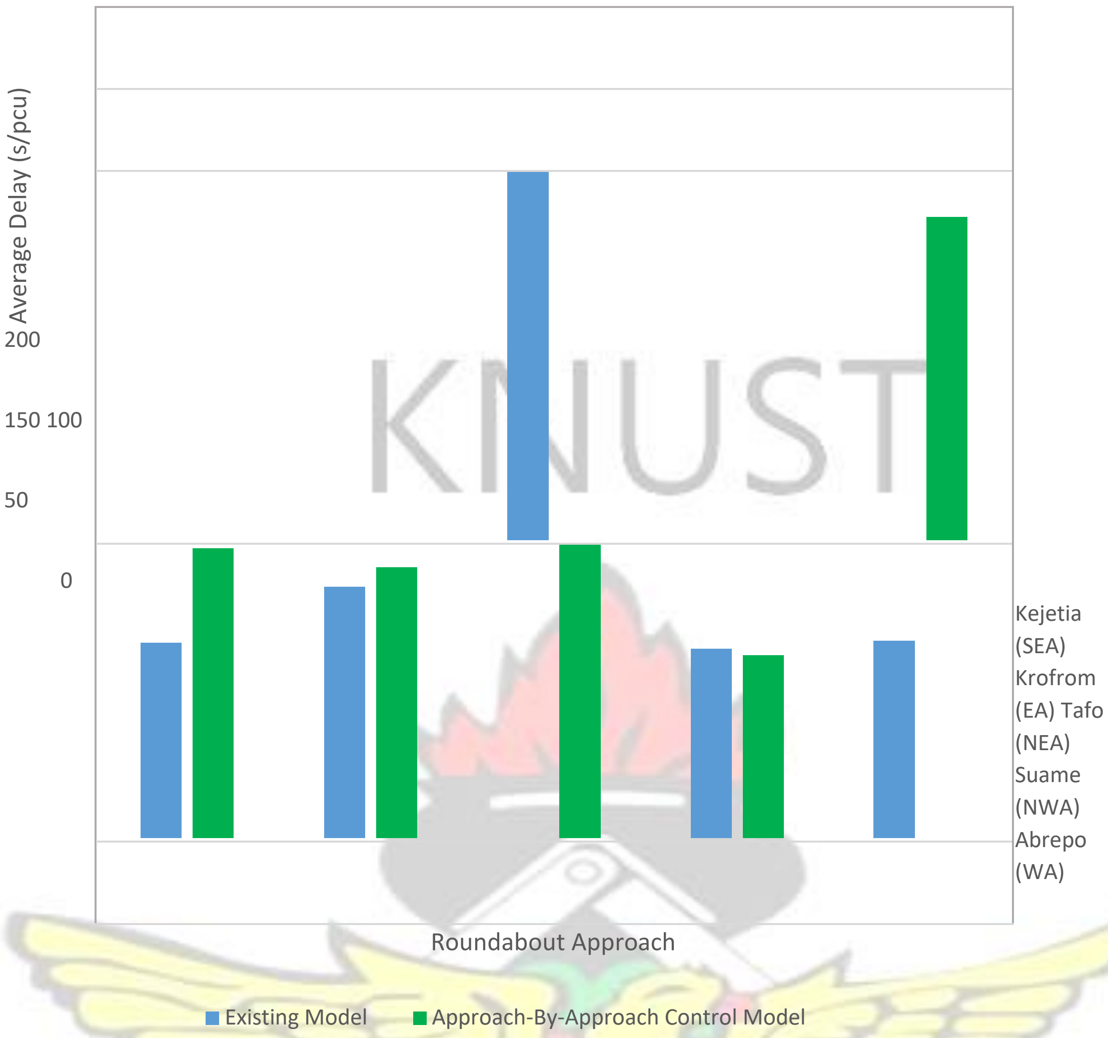


Figure 4.5. Average Approach Delay of Existing Model Compared with that of Approach-by-Approach Model

The trend in Figure 4.5 shows an increase in average delay for each approach from the simulation runs of the Approach-by-Approach Control Model as compared to the Existing Model. Except for the Tafo and Suame approaches which shows the reverse. The excessive delay for queueing vehicles stems from the number of phases and the need to allow circulating flow to clear from any approach before the next approach traffic is released.

A comparison of the average approach delay from the Metered Approach Model with that from the Existing Model is displayed in Figure 4.6.

250

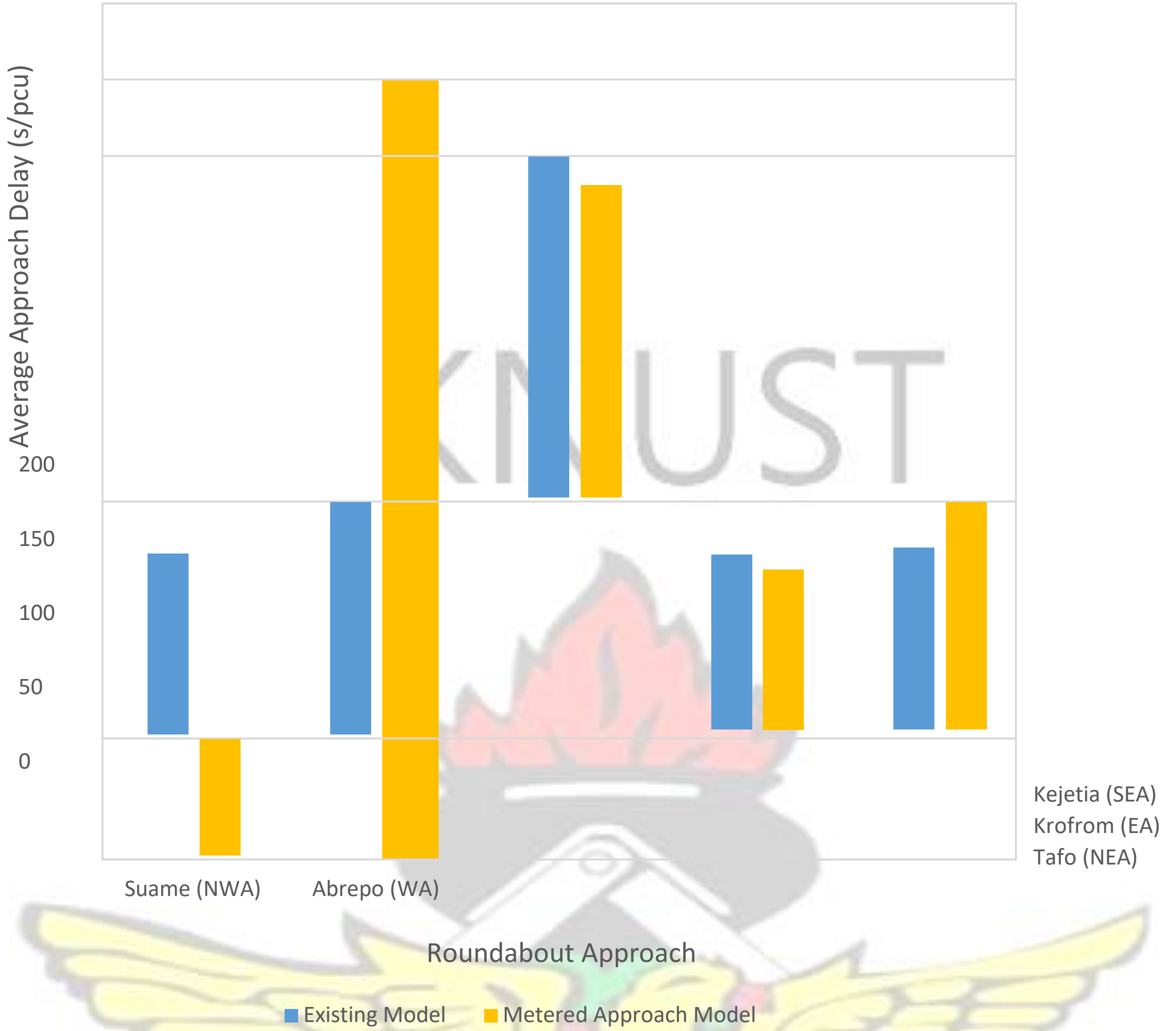


Figure 4.6. Comparison of Average Approach Delay from Existing Model to that from Metered Approach Model

The general trend shown by the figure is a reduction in average approach delay from the major approaches and an increase in the case of two of the minor approaches (Krofrom and Abrepo) in respect of the Metered Approach Model. Signalisation of the two minor approaches (Krofrom and Abrepo) caused the extra delay on those approaches which also created more space for movement from the major approaches. This resulted in reduction of the average queue delay on the major approaches. The reduction in average delay from the Metered Approach regarding the minor Approach (Tafo) is attributable to the general change in the balance of flow due to the metering.

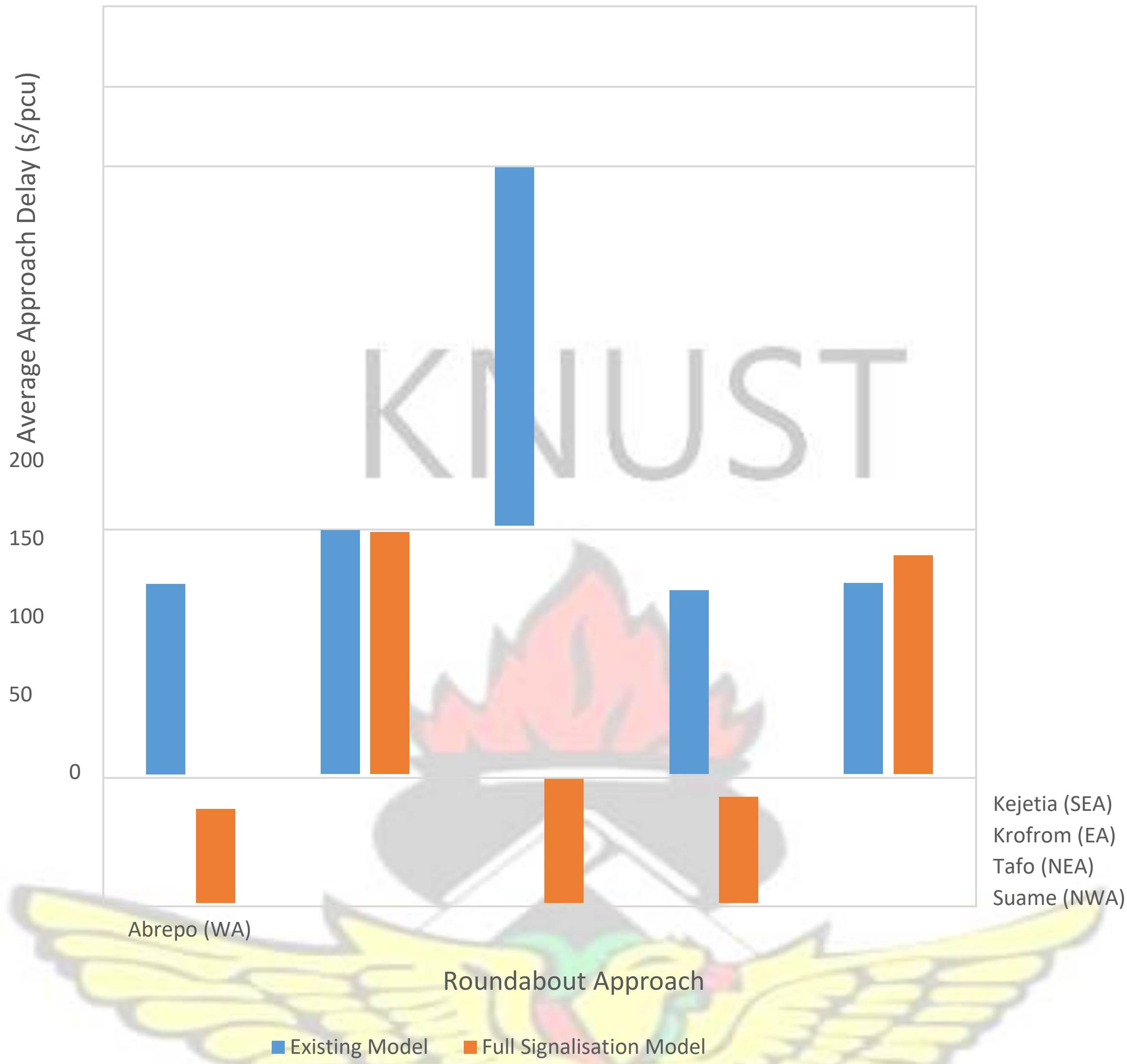


Figure 4.7. Comparison of Average Approach Delay from Existing Model with that from Full Signalisation Model

In this option, almost all approaches recorded low average delay compared to the unsignalised option. However, results for the West Approach (Abrepo) managed to buck the trend. This is attributable to alteration in the balance of traffic flow due to the signalisation on all the approaches as well as the circulatory carriageway. Overall low results imply that the entire roundabout under a full signalisation option, reduced the queueing delay.

4.4.3 Average Queue Length

This sub-section discusses the results of the average queue lengths from each model for each approach. Figure 4.8 display the average queue length for the Approach-by-Approach

Control Model as compared with same for the Existing Model.

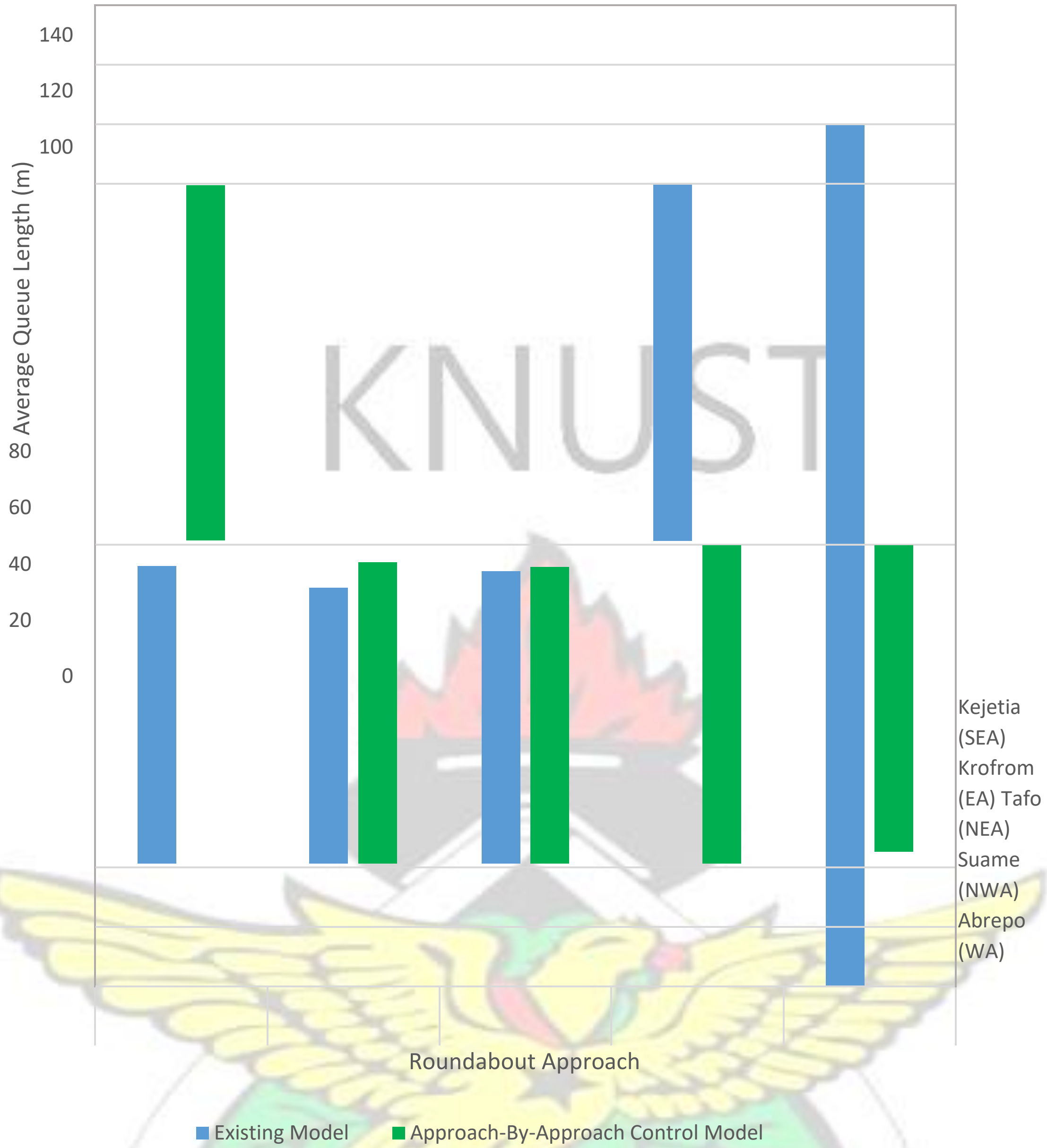


Figure 4.8. Comparison of Average Queue Length of Existing Model to that of Approach-by-Approach Control Model

The average queue lengths exceeded the queue lengths in the un-signalised roundabout condition except for the Suame Approach which shows the reverse. The excessive queuing is attributed to the need to allow circulating flow to clear from any approach before release of the next stream of vehicles.

The comparison of the average queue length for the Metered Approach Model with that of the Existing Model is displayed in Figure 4.9.

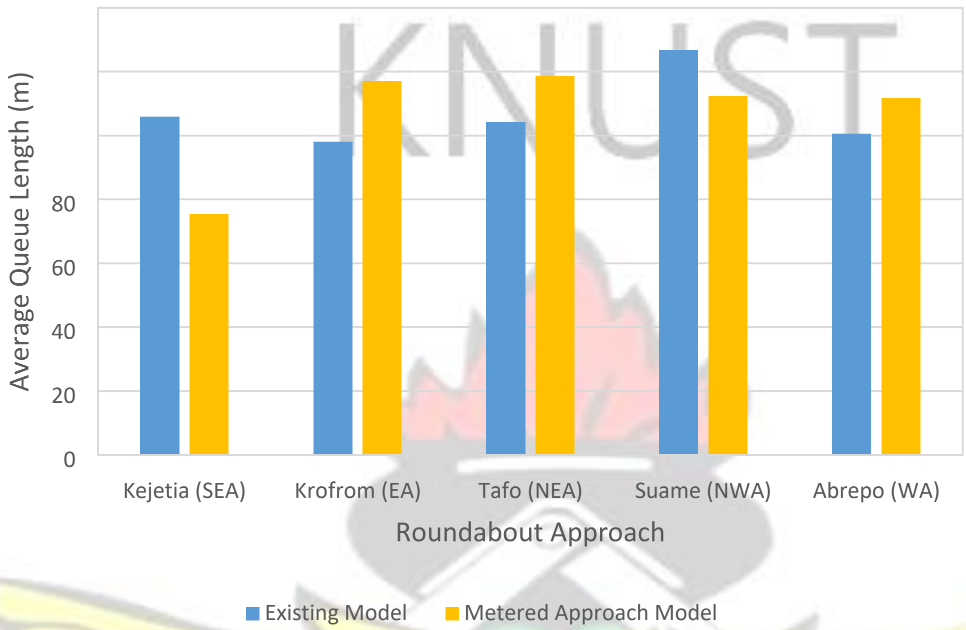


Figure 4.9. Comparison of Average Queue Length from Existing Model with that from Metered Approach Model

The results indicate an increase in average queue length with respect to the metered approaches and a decrease in the case of the un-signalised approaches. This happened because in the metered approach option, traffic movements from the major approaches were unhindered by traffic signal control.

Figure 4.10 shows the average queue lengths from the Full Signalisation Model and the Existing Model. The Full Signalisation Model produced lower average queue lengths compared to those generated from the Existing Model. The Full Signalisation Model introduced new traffic flow balance with proportionate fixed allocation of right of way with a relatively small cycle time of 60s. The changes described above caused the improvements in queue lengths.

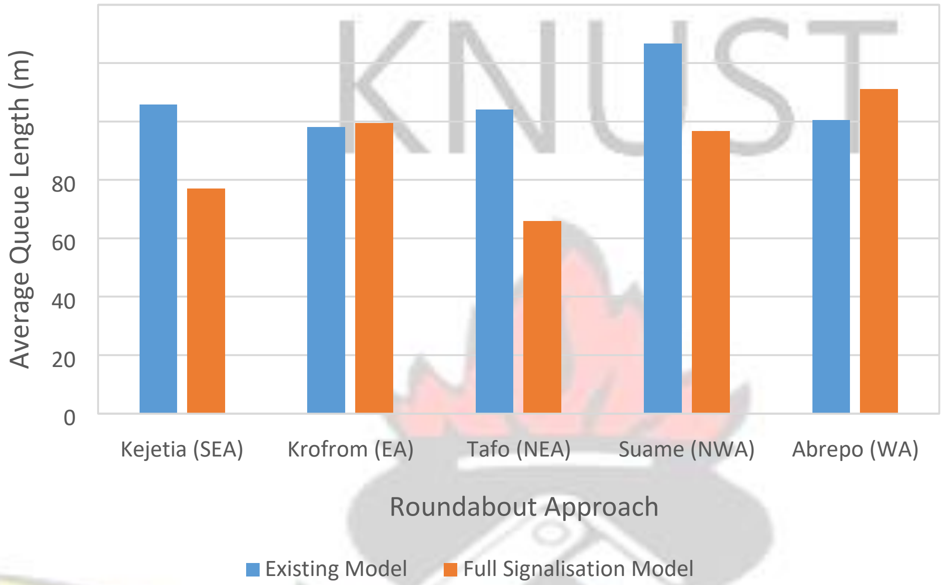


Figure 4.10. Comparison of Average Queue Length from Existing Model with that from Metered Approach Model

4.4.4 Overview of Discussion

Based on the trends in the figures as have been discussed above, the Full Signalisation Model provided the highest improvement in capacity, and lowest average delay and queue lengths followed by the Metered Approach and then Approach-by-Approach control models. The full signalisation model resulted in improvement in almost every parameter measured on virtually all the approaches.

The Metered Approach Model resulted in improvement on that of the existing model but fell short in comparison with that of Full Signalisation. The phenomenon could be explained by the fact that un-signalised roundabouts operate on the principle of yield

140

120

100

control i.e. self- regulating while fully signalised roundabouts are controlled internally (circulatory lanes) and externally (all approaches) by traffic signals. Metered roundabouts are, however, unique because they are partly self-regulating and partially signal controlled hence giving improvements which lie between the Full Signalisation Model and the Existing Model.

As discussed earlier, assignment of the circulating roadway to one approach traffic for each phase resulted in the longest delays and queue lengths as well as the lowest throughput among all the proposed models in respect of the Approach-by-Approach Control Model.

Even with improvement in roundabout performance by the various options, the average degree of saturation remained greater than 1.00 (see Table 4.9). That notwithstanding, full signalisation still provides the best results followed by the Metered Approach Model. The results from the full signalisation model and metered approach model simulation appear to validate the assertion made by Tracz and Chodur (2012) that under circumstances of heavy traffic intensity at an intersection, roundabouts may operate effectively with traffic signal control.

Table 4.9. Assessment of Degree of Saturation

Approach	Traffic Demand (pcu/h)	Existing Model		Approach-by-Approach Control Model		Metered Approach Model		Full Signalisation Model	
		Maximum Entry Throughput (pcu/h)	Degree of saturation	Maximum Entry Throughput (pcu/h)	Degree of saturation	Maximum Entry Throughput (pcu/h)	Degree of saturation	Maximum Entry Throughput (pcu/h)	Degree of saturation
Kejetia (SEA)	1069	751	1.4	623	1.7	902	1.2	950	1.1
Krofrom (EA)	879	539	1.6	563	1.6	536	1.6	648	1.4
Tafo (NEA)	778	424	1.8	486	1.6	392	2.0	723	1.1
Suame (NWA)	1211	833	1.5	845	1.4	868	1.4	1016	1.2
Abrepo (WA)	769	556	1.4	528	1.5	562	1.4	608	1.3
Combined	4706	3103	**1.5**	3045	**1.5**	3260	**1.4**	3944	**1.2**

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

Based on the results of this study the following conclusions have been drawn:

- Full signalization as well as metering could increase the capacity of a roundabout even though the all approaches could be overloaded (v/c >1.0).
- The Approach-by-Approach Model is not an effective and efficient method for improving the performance of the Suame Roundabout because it results in low values of approach capacity, excessive delays, long queues and the worst degree of saturation compared to the existing situation.
- Full signalisation option would be the best for improving capacity, and reducing queue lengths and delays at the Suame Roundabout.
- Metered Approach Model produced marginal improvement over the existing situation but compared to the full signalisation, this option would be less expensive and easy to implement as a short term measure for improving performance of the Suame Roundabout.

5.2 Recommendations

Following from the conclusions drawn, one definitive recommendation is to undertake further studies regarding roundabout signalisation. Areas that could be the focus for further studies may include but not limited to the following:

- Metering as a subject on its own
- Full signalisation (Exploring with alternative roundabout geometries)

The full signalisation of the Suame roundabout is recommended to improve capacity and reduce vehicular delays. However, if the budget is not available, the metered option will also provide more consistency in operations at the roundabout as compared to the current situation where Police Personnel and Traffic wardens control movement.

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APPENDICES

APPENDIX A

-1. SUMMARY OF FIELD TRAFFIC DATA

TIME (AM)	SUAME/OFFINSO APPROACH		
	SMALL	MEDIUM	HEAVY
6.30-6.45	66	91	6
6.45-7.00	81	77	7
7.00-7.15	80	85	9
7.15-7.30	77	83	9
7.30-7.45	78	81	8
7.45-8.00	76	90	10
8.00-8.15	74	83	6
8.15-8.30	74	72	7
8.30-8.45	67	75	16
8.45-9.00	62	53	8
9.00-9.15	52	75	9
9.15-9.30	48	68	14

TABLE A

TABLE A-2. SUMMARY OF FIELD TRAFFIC DATA

TIME (AM)	ABREPO JUNCTION APPROACH		
	SMALL	MEDIUM	HEAVY
6.30-6.45	84	52	4
6.45-7.00	50	60	8
7.00-7.15	58	50	9
7.15-7.30	54	40	3
7.30-7.45	58	35	2
7.45-8.00	56	43	2
8.00-8.15	56	32	5
8.15-8.30	54	48	2
8.30-8.45	54	41	3
8.45-9.00	53	47	2
9.00-9.15	51	48	8
9.15-9.30	80	58	7

-3. SUMMARY OF FIELD TRAFFIC DATA

TIME (AM)	KEJETIA APPROACH		
	SMALL	MEDIUM	HEAVY
6.30-6.45	41	92	5
6.45-7.00	52	94	6
7.00-7.15	40	90	9
7.15-7.30	54	71	6
7.30-7.45	49	83	6
7.45-8.00	44	97	6
8.00-8.15	53	67	7
8.15-8.30	29	68	7
8.30-8.45	60	73	6
8.45-9.00	58	68	9
9.00-9.15	54	68	4

TABLE A

9.15-9.30	56	100	6

TABLE A-4. SUMMARY OF FIELD TRAFFIC DATA

TIME (AM)	KROFROM APPROACH		
	SMALL	MEDIUM	HEAVY
6.30-6.45	37	43	6
6.45-7.00	61	68	9
7.00-7.15	50	45	10
7.15-7.30	52	55	16
7.30-7.45	42	37	10
7.45-8.00	48	64	10
8.00-8.15	52	58	7
8.15-8.30	48	48	11
8.30-8.45	46	64	8
8.45-9.00	59	64	7
9.00-9.15	69	68	5
9.15-9.30	45	39	4

-5. SUMMARY OF FIELD TRAFFIC DATA

TIME (AM)	TAFO / MAMPONG APPROACH		
	SMALL	MEDIUM	HEAVY
6.30-6.45	32	44	3
6.45-7.00	35	50	2
7.00-7.15	36	57	4
7.15-7.30	31	48	5
7.30-7.45	33	56	4
7.45-8.00	28	61	6

TABLE A

8.00-8.15	44	64	3
8.15-8.30	38	63	4
8.30-8.45	36	60	2
8.45-9.00	44	68	5
9.00-9.15	47	67	5
9.15-9.30	45	58	4

TABLE A-6. FIELD QUEUE LENGTH RESULTS FOR THE TARGET APPROACH (KEJETIA APPROACH)

Time (AM)	Queue Length (m)
6:30-6:45	96
6:45-7:00	168
7:00-7:15	144
7:15-7:30	96
Average Queue Length (m)	126

APPENDIX B

TABLE B-1. Approach Capacity Results

	Throughput (pcu/h)			
Approach	Existing Model	Approach-byApproach Control Model	Metered Approach Model	Full Signalisation Model
Kejetia (SEA)	751	623	902	950
Krofrom (EA)	539	563	536	648
Tafo (NEA)	424	486	392	723
Suame (NWA)	833	845	868	1016
Abrepo (WA)	556	528	562	608
Combined Capacity (pcu/h)	**3103**	**3045**	**3260**	**3944**

TABLE B- 2. Average Delay Results

	Average Delay (s/pcu)			
Approach	Existing Model	Approach-byApproach Control Model	Metered Approach Model	Full Signalisation Model
Kejetia (SEA)	122	181	79	60
Krofrom (EA)	157	169	200	155
Tafo (NEA)	230	183	211	79
Suame (NWA)	118	114	108	68
Abrepo (WA)	123	202	153	140

TABLE B- 3. Average Queue Length Results

	Average Queue Length (m)			
Approach	Existing Model	Approach-byApproach Control Model	Metered Approach Model	Full Signalisation Model
Kejetia (SEA)	105.9	126.5	75.2	77.1
Krofrom (EA)	98.1	107.2	117.0	99.4
Tafo (NEA)	104.1	105.5	118.5	65.9
Suame (NWA)	126.7	113.4	112.3	96.8
Abrepo (WA)	100.5	109.1	111.7	111.1

Section B- 1. Signal Timing Design

TABLE B- 5. Critical Demand Analysis for Determination of Green Time / Cycle Time

Approach	Traffic Demand (pcu/h)	Critical Demand Ratio
Suame	1211	0.25
Abrepo	769	0.16
Kejetia	1069	0.23
Krofrom	879	0.19

Tafo	778	0.16
Total	4706	

1. **Approach-by-Approach Control Model**

The signal timing was design as follows:

Number of approaches = 5

Yellow Time = 3 s

Total Yellow Time = 5 x 3 s = 15 s

Use Cycle Time = 60s

Total Green Time = 60s – 15s = 45s

Using the least critical demand ratio in TABLE B-5 Green time for each phase/approach was estimated as follows;

Green Time (Suame Approach) = 0.25 x 45 = 11.25 ≈ 12s

Green Time (Kejetia Approach) = 0.23 x 45 = 11.25 ≈ 11s

Green Time (Krofrom Approach) = 0.19 x 45 = 8.55 ≈ 8s

Green Time (Both Tafo and Abrepo approaches) = 0.16 x 45 = 7.2 ≈ 7s

For this option, Cycle time = total green time + total yellow

= 45s + 15s= 60s

2. **Metered Approach Model**

Choosing 60 s for cycle length

And Using Krofrom approach's critical demand ratio = 0.19

Green time for the metered approaches = 60 x 0.19 = 11.4 ≈ 12 s

3. Full Signalisation Model

Choosing 60 s for cycle length

Using Suame approach's critical demand ratio of 0.19

Green time for the major approaches = 60 x 0.25 = 15 s

And Using Abrepo approach's critical demand ratio of 0.16

Green time for the minor approaches = 60 x 0.16 = 9.6 ≈ 10 s

TABLE B - 4. Percentage Differences between Existing Model and Proposed Models with respect to Measured Parameters

Approach	Percentage Change in Average Delay (%)			Percentage Change in Average Queue Length (%)			Percentage Change in Average Capacity (%)		
	Approach-byApproach Control Model	Metered Approach Model	Full Signalisation Model	Approach-byApproach Control Model	Metered Approach Model	Full Signalisation Model	ApproachbyApproach Control Model	Metered Approach Model	Full Signalisation Model
Kejetia (SEA)	-119	36	51	-19	29	27	-17	20	26
Krofrom (EA)	-62	-28	1	-9	-19	-1	5	0	20
Tafo (NEA)	9	8	65	-1	-14	37	15	-8	70
Suame (NWA)	-81	9	42	11	11	24	1	4	22
Abrepo (WA)	-119	-25	-14	-9	-11	-11	-5	1	9
Combined Percentage Change				-6	-1	15	-2	5	27

Note: (-) Sign in front of any value in the table above indicates a worse result as compared to the existing model

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