THE IMPACT OF LOCATION OF FUEL SERVICE STATIONS ON THE PERFORMANCE OF SIGNALIZED INTERSECTIONS IN KUMASI

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Master of Science
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(College of Engineering)

By

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SEPTEMBER, 2009

DECLARATION

I hereby declare that this submission is my own work towards the Master of Science 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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DEDICATION

This piece of work is dedicated to

Mr. Edmund Kwame Debrah (*my father*)

and

Mrs. Cecilia Darkowah Debrah (*my mother*)

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ABSTRACT

The overall performance of a given road network is to a large extent dependent on the levels of service of the intersections. For signalized intersections the number of lanes, lane widths, traffic composition, grade, speed, distribution of green times among conflicting movements, significantly affect both the capacity and performance of the intersections. In Ghana, it is commonplace to locate a Fuel Service Station (FSS) within the vicinity of an intersection. The presences of these facilities invariably alter the roadway conditions of the intersections, particularly with the opening of accesses onto the main roadways leading into the intersections, thus rendering the signalized intersections to perform at sub-optimal levels.

This study assessed the impact of the location of FSS on the performance of four (4) signalized intersections in Kumasi. Traffic and road geometric data were collected at these sites and simulated using the *Synchro 6.0 plus Sim trafficware*. The simulation results of the existing situations were further compared with that of hypothetical traffic scenarios representing the non-existence of the FSS (i.e. the accesses to the FSS are absent). The overall assessments show that the location of FSS within the vicinity of a signalized intersection result in the reductions of saturation flow rates; as marginal as 2% through in excess of 50% for some of movements depending on the orientation of accesses of the FSS to the intersection roads. Again, there are associated increases in lost times, up to 7sec/movement. These ultimately contribute to about 8% to 30% of the overall intersection delays. The field studies also revealed that for certain movements, especially right turning, the accesses to the FSS may facilitate by-passing of the intersections which promotes ease of congestion at the signalized intersections.

Finally, improvements of the existing situations by optimizing the signal timings and enhancement of the geometric characteristics of some of the signalized intersections were explored and recommendations put forward. The findings of the study can be a useful guide to transportation professionals, and officials of authorities responsible for the issuing permits for the establishment of the FSS and other similar facilities.

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LIST OF ABBREVIATIONS AND ACRONYMS

CBD	Central Business District
CORSIM	An Acronym of a Traffic Software
FSS	Fuel Service Station(s)
GHA	Ghana Highway Authority
HCM	Highway Capacity Manual
HCS	Highway Capacity Software
HCM (2000)	Highway Capacity Manual, 2000 Edition
ICU	Intersection Capacity Utilization
KMA	Kumasi Metropolitan Authority
KNUST	Kwame Nkrumah of Science and Technology
LOS	Level of Service
MOE	Measure of Effectiveness
PHF	Peak Hour Factor
R/A	Round About (Intersection)
SIDRA	An Acronym of a Traffic Software
STC	State Transport Company, Ghana
SYNCHRO	An Acronym of a Traffic Software
TIA	Traffic Impact Assessment
TIS	Traffic Impact Statement
TRANSYT-7F	An Acronym of a Traffic Software
TRB	Transport Research Board
TRL	Transport Research Laboratory, U.K
TRRL	Transport Research Laboratory, England
hr	hour
m	meter
sec	second
vphgpl	vehicles per hour of green time per lane

1.0 INTRODUCTION

1.1 Background

The overall performance of a given road network is to a large extent dependent on the levels of service of its intersections. Therefore analyses procedures that provide for the determination of the measures of effectiveness of the performance of intersections are of great importance.

According to Freeman et al (2000) the amount of traffic which can be handled by an intersection depends on characteristics of the vehicle and pedestrian stream, traffic control measures, various physical and operating characteristics of the roadway, and the environmental conditions which have a bearing on the actions of the driver. Consequently, factors that affect the level of service at signalized intersections include the flow and traffic distributions, the geometric characteristics, and the signal control system in place.

The traffic flows measured in vehicles per hour at a given intersection is dependent on the saturation flows at the intersection. Saturation flow is an important element for the operational analysis of a signalized intersection (Nevers, 2000). This parameter is a basic input for the determination of performance indicators such as average vehicle delay, level of service, capacity, and queue formation for an intersection. The most commonly used guide for estimating saturation flow is the Highway Capacity Manual (HCM).

1.2 Problem Statement

In Ghana, it is commonplace to locate a Fuel Service Station (FSS) within the vicinity of an intersection. At an intersection there exist a number of conflicting movements. The establishment of a FSS requires opening accesses onto the roadways which introduces additional conflict points. Thus the location of these facilities invariably alters the roadway conditions of the intersections particularly with the opening of accesses onto the intersection's main roadways.

Apart from the main fuel service rendered, the FSS usually offers other ancillary services such as vulcanization, lubrication, as well as grocery shops. The presence of such additional services induces other traffic activities which ultimately impact on the overall traffic operations of the intersections.

The saturation flow rates of some traffic movements are reduced as a result of the induced traffic activities of the FSS among other factors (Adams and Obiri-Yeboah, 2008). These introduce impedances to the free flow of vehicular traffic at the intersections making them operate at sub-optimal levels. Hence, the intended performance of the signalized intersections may be significantly affected leading to queue formations and associated delays on the intersection approaches.

The study therefore looks at the impact of location of FSS on the performance of signalized intersections.

1.3 Justification of Study

The findings of this study are expected to offer an insight to repercussions of locating FSS and other facilities such as shopping malls which require opening of accesses onto roads at the approaches of signalized intersections. Hence with this study transportation professionals can make the necessary modifications in the design and implementation of traffic schemes to efficiently cater for such facilities in the vicinity of signalized intersections. In addition, agencies and authorities responsible for issuing permits for the establishment of such facilities requiring the opening of accesses onto the roads of signalized intersections could make informed decisions.

1.4 Study Goal And Objectives

The goal of this study was to understand and quantify the nature of the impact the location of FSS in the vicinity signalized intersections have on the performance of the intersections. The specific objectives were to:

- Establish the impact of location of FSS on the observed saturated flow rates of identified lane groups at the signalized intersections.
- Estimate the actual lost times at the intersections.
- Assess the changes, if any, in delays at the signalized intersections using the simulation results from the *Synchro 6.0 plus SimTraffic* software.

2.0 LITERATURE REVIEW

2.1 History of Performance Appraisal of Intersections

Since the advent of motorized road transport, the performance of intersections has generally been subjected to critical analysis for various reasons. A detailed literature search conducted by Freeman et al (2000) revealed that signalized intersections capacities in particular have been subjected to a great deal of study since the mid 1950's. Mousa (2002) and Fambro and Rouphail (1997) allude to the fact that the appraisals of the performance of signalized intersections have been conducted in many jurisdictions around the world and recommended improvements implemented.

Presently, manual ways of assessing the performance of signalized intersection system have given way to the use of computer based programs. For instance, Lu et al (2004) assessed the performance of signalized intersections in two Asian cities; Tokyo and Beijing using filmed traffic and desk analysis with a software package. The study included the assessment of the contributory factors leading to lower saturation flow rates in Beijing relative to that in Tokyo. Some of the interventions in their recommendation for enhancing the saturation flow rates in Beijing included provision of channelization facilities at the intersections, improvement of the guidance markings at the intersections, and strengthening of education of road users on traffic rules and traffic safety consciousness.

2.2 Lane Groups

The Highway Capacity Manual (HCM, 2000) describes a lane group as a single movement, a group of movements, or an entire approach that is defined by the geometry of the intersection and the distribution of movements over the various lanes. According to Garber and Hoel (1994) a lane group consists of one or more lanes that have a common stopline, carry a set of traffic streams, and whose capacity is shared by all vehicles in the group. The authors also put forward the following guidelines for the identification of lane groups of a given signalized intersection:

1. Separate lane groups should be established for exclusive left-turn lane(s), unless the approach also contains a shared left-turn and through lane. In such a case, consideration should also be given to the distribution of traffic volume between the movements. These same guidelines apply for exclusive right-turn lanes
2. When exclusive left-turn lane(s) and/or exclusive right-turn lane(s) are provided on an approach, all other lanes are generally established as a single lane group
3. When an approach with more than one lane also has a shared left-turn lane, the operation of the shared left-turn lane should be evaluated to determine whether it is effectively operating as an exclusive left-turn lane because of the high volume of left-turn vehicles on it.

2.3 Saturation Flow Rates

2.3.1 Introduction to Saturation Flow Rates

Welsh (2003) defined saturation flow as the rate at which traffic will flow given an infinite reservoir of road space. Saturation flow rates of a given lane group by Garber and Hoel's (1994) definition is the flow rate in vehicles per hour of green time per lane (vphgpl). They further explain that the saturation flow rate of a given lane group is the number of vehicles that could be carried by that lane given one continuous hour of green time without interruptions. Ideal Saturation flow rate (S_o), therefore, is that flow in the absence of all traffic, geometric, and other external impedances. This is usually taken as 1900 vphgpl. However, in real situations there could be a number of factors that could alter the ideal saturation flow rate. Thus the saturation flow rate of any given lane group is usually adjusted to reflect the desired conditions or existing traffic situation.

It is, arguably, the single most important element of a signalized intersection operational analysis, according to Nevers (2000). This assertion is buttressed by the fact that the basic parameters for the design and analysis of signal controlled intersections are saturation flow, lost time and traffic composition (Hossain, 2001).

2.3.2 Common Factors Affecting Saturation Flow Rates

For a given lane group there may exist a number of factors that could influence the ideal saturation flow which result in the real and observed saturation flow. However, empirical studies conducted by transportation professionals reveal a number of common factors. These have been identified by Garber and Hoel (1994) following extensive field studies, to include:

- lane widths of the intersection approaches,
- heavy vehicles representation in traffic stream,
- approach grade,
- adjacent parking areas along travel lanes ,
- area type – typically of the landuse,
- bus blockage – stoppages by buses and other commercial vehicles,
- lane utilization,
- right turnings, and
- left turnings.

However, in Ghana, some of the factors established to affect saturation flow rates at signalized intersections include driver behavior, presence of FSS, pedestrian activity, and presence of lay-byes among the commonly known factors (Adams and Obiri-Yeboah, 2008). Thus, the ideal saturation flow (S_o) is usually adjusted for the prevailing conditions to obtain the adjusted saturation flow (S′) for a given lane group.

2.3.3 Bus Blockage at Signalized Intersections

When a bus stops on a travel lane for some reason usually to discharge or pick up passengers, some of the vehicles immediately behind the bus stop or at best slow down. This results in a decrease in the maximum volume that can be handled by the affected lanes. According to Garber and Hoel (1994), within 250ft (approximately 76m) upstream or downstream from the stop line of an intersection the stoppages of local buses have effect on the flow, and consequently the performance of the intersection. A local bus is one that stops to pick up and/or discharge passengers within the intersection at either a near or a far side bus stop. Stopped buses disrupt the flow of other vehicles and influence the saturation flow rate of the affected lane group.

This effect when establishing the adjusted saturation flow rate of affected lane groups is corrected by applying the recommended adjustment factor (f_{bb}) which is related to the number of buses in an hour that stop on the travel lane. Equation 2.1 gives the relation:

$$f_{bb} = \frac{N - \frac{14.4 N_b}{3600}}{N} \qquad \text{Equation (2.1)}$$

where

f_{bb} = bus blockage adjustment factor

N = number of lanes in lane group

N_b = number of vehicles (bus) stopping in an hour

2.3.4 Vehicle Maneuvers Into or Out of Drive Lanes

As vehicles move in or out of a drive lane to a side parking space or other side areas the resultant effect is a reduction of the maximum flow rate that the lane group can handle. At signalized intersections, turning vehicles often use the same share lane together with the through traffic (Wu, 2009). This effect is corrected for by using an adjustment factor on the base saturation flow rate. According to Garber and Hoel (1994), this factor of correction for side parking (f_p) is dependent on the number of lanes in the lane group (N) and the number of vehicle maneuvers per hour (N_m). From field studies it has been shown that each vehicle maneuver (either in or out) blocks traffic on the adjacent lane group for an average of 18 seconds. The relevant relation for the corrective factor is as presented in Equation 2.2

$$f_p = \frac{N - 0.1 - \frac{18N_m}{3600}}{N} \qquad \text{.......Equation (2.2)}$$

2.3.5 *Field Determination of Saturation Flow Rates*

There are a few ways of determining the saturation flow rates of a particular lane group in the field. For instance there is the manual investigation method which involves the counting of vehicles passing through the intersection per unit time during the green time. This method was introduced by TRRL in England (Lu et al, 2004). The method suggested by Garber and Hoel (1994), however, employs two persons- a timer with stop watch and a recorder using a simple field sheet.

A visible mark coinciding with the stop line of the lane group is used as a reference point. While there is a queue of vehicles on the lane group of interest, the timer starts the stop watch on the beginning of green phase for that lane group. The recorder is notified as the rear wheels of each vehicle cross the reference point. The timer shouts the time for the 4th, 10th and last (N^{th}) vehicle crosses the reference points for the recorder to note them for the greentime of that particular cycle. The recorder notes these times on the standard field sheet. The procedure is repeated for about five more times for the establishment of averages of the required variables; vehicle cross times and number of vehicle passes over the reference point.

Since the flow just after the start of the green time is usually less than saturation flow, the time considered for calculating the saturation flow is that between the time the rear axle of the fourth (4^{th}) vehicle crosses the reference point (t_4) and the corresponding crossing time (t_n) of the last (N^{th}) vehicle. The saturation flow (S) is then determined from Equation 2.3 below:

$$S = \frac{3600(N-4)}{t_n - t_4} \quad \text{.......................................Equation 2.3}$$

It must be noted, however, that the above formula is more applicable in developed cities since traffic characteristics and driver behaviour are significantly different in developing cities. Traffic stream of developing cities such as Dhaka, Bangladesh comprises of both motorised and non-motorised vehicles sharing the same carriageway whereas lane discipline is not the best (Hossain, 2001).

2.4 Delays At Signalized Intersections

The HCM (2000) defines delay as the difference between the travel time actually experienced and the reference travel time that would result during ideal conditions; in the absence of traffic control, geometric delay, or any incidents, and when there are no other vehicles on the road. At signalized intersections, according to Click (2003) delay is associated with the time lost to a vehicle and/or driver because of the operation of the signal and the geometric and traffic conditions present at the intersection.

associated with the time lost to a vehicle and/or driver because of the operation of the signal and the geometric and traffic conditions present at the intersection.

There are several different types of delay that can be measured at an intersection, and each serves a different purpose to the transportation engineer. The signalized intersection capacity and level of service (LOS) estimation procedures are built around the concept of average control delay per vehicle. Control delay is the portion of the total delay attributed to traffic signal operation for signalized intersections (TRB, 2000). Various components of vehicular delay at signalized intersection, including control delay used in the HCM (2000) are shown below (Quiroga and Bullock, 1999).

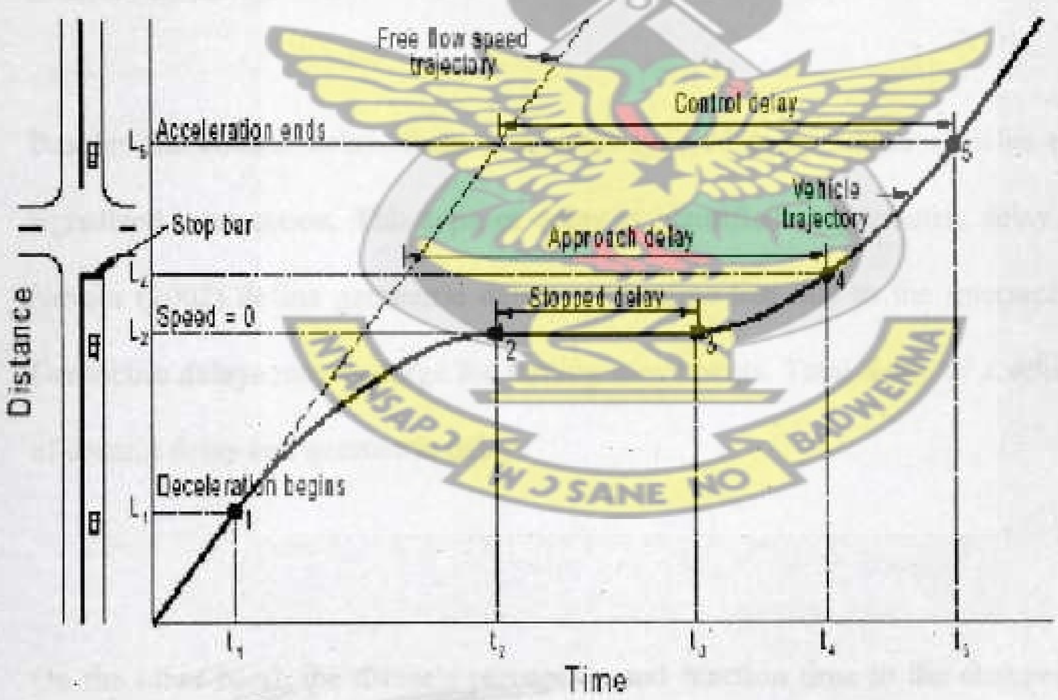


Figure 2-1 Delay Components at Signalized Intersection

Control delay can be categorized into deceleration delay, stopped delay and acceleration delay. Stopped delay is easier to measure, while overall delay reflects better the efficiency incurred by a decelerating or accelerating vehicle is categorized as deceleration and acceleration delay, respectively.

In Figure 2-1, it is first observed that several vehicles reaching the intersection come to a complete stop. These vehicles need to stop either as a consequence of their arrival during the red interval or during the green interval when the queue of vehicles that had formed during the previous red interval has not yet fully dissipated. While it is further observed that the rest of vehicles only experience deceleration and acceleration delay, as these vehicles reach the intersection when all previously queued vehicles have already started to move and therefore only need to slow down to maintain a safe distance with the vehicles ahead of them.

Besides the control delay, there is another type of delay which vehicles experienced at signalized intersection. This type of delay is identified as geometric delay. Luttinen and Nevala (2002) define geometric delay as the time lost due to the intersection geometry. Geometric delays may be large for turning movements. Total delay of a vehicle is the sum of control delay and geometric delay.

On the other hand, the driver's perception and reaction time to the changes of the signal display at the beginning of the green interval and during yellow interval to mechanical constraints and to individual driver behavior also contribute to the traffic delay at signalized intersection. Husch and Albeck (2004) explain that during simulation process using SimTraffic micro simulation software, there are input parameters termed driver parameters. These parameters involve yellow deceleration, yellow reaction time, green reaction time, headways and gap acceptance factor.

Another element that may affect the delays incurred at intersection approaches is the randomness in vehicle arrivals. If vehicles were to arrive at uniform intervals, the delays incurred by vehicles within successive signal cycles would be identical, as there would then be an exact replication of the arrival and departure patterns. However, under random arrival patterns, the number of arrivals may fluctuate from one cycle to the other, thus resulting in different queue lengths. This may in turn result in arrival demands that occasionally exceed the approach capacity, and therefore, in higher delays. Finally, platoon arrivals may also occur in coordinated traffic signal systems. In this case, the delay incurred by vehicles will depend on the degree to which the signals at successive intersections are timed to provide a green indication during the periods of high arrival flow rate (Dion et al, 2004).

2.5 Lost Time

Lost time is the time during which no vehicles are able to pass through an intersection despite the traffic signal displaying a green (go) signal. The total lost time is the sum of two separate elements: *start-up lost time* and *clearance lost time*. Start-up lost time occurs when a traffic signal changes from red (stop) to green. Some amount of time elapses between the signal changing from red to green and the first queued vehicle moving through the intersection. There is then an additional amount of time for the next vehicle to begin moving and pass through the intersection, and so on. The total time taken for all waiting drivers to react and accelerate is the start-up lost time. Clearance lost time on the other hand is the time lost to stopping a line of vehicles at the end of a green phase. The clearance time depends on the clearance distance and average traverse speeds through the intersections. Lost time is always measured in seconds (Roess et al, 2004).

Start-up lost time can be calculated as the sum of the differences between the headways for the first cars in line and the average headway through the intersection at a theoretical maximum flow, the *saturation flow rate*. When no observations have been made, the start-up lost time is assumed to be 2.0 seconds as a default value HCM (2000). The default value for clearance time is 2.0 seconds, thus the default value for lost time is 4.0 seconds.

In design, the lost time for a particular movement is taken as;

Lost time = Max (4s; 2s + D_i/S_i; Y + AR)……………………Equation 2.4

where D_i = intersection clearance distance (m) for the i^{th} movement

S_i = average speed of clearance (m/s) for the i^{th} movement

Y+AR = yellow and all red time (s)

2.6 Capacity and Performance Analysis of Signalized Intersections

In the 2000 Highway Capacity Manual approach, capacity at intersections is defined for individual lane groups and for the intersection as a whole. Capacity of a lane group is calculated as the maximum rate of flow that may pass through the intersection under prevailing traffic, roadway, and signalization conditions.

The rate of flow is generally measured or projected for a 15-minute period and capacity is stated in vehicles per hour. Capacity analysis of intersections involves the computation of volume-to-capacity (v/c) ratios for each lane group, from which an overall intersection v/c ratio may be derived.

Generally, when two opposing flows are moving during a single phase, one of the lane groups will require more green time than another to process all of its volume. This would be defined as the "critical" lane group for the subject signal phase. The concept of a critical v/c ratio is used to evaluate the intersection as a whole, considering only the critical lane groups or those with the greatest demand for green time within each signal phase. This procedure assumes that green time has been appropriately allocated. Thus, it is possible to have an overall intersection v/c of less than 1.00 (under capacity), but still have individual movements be over-saturated within the signal cycle if the green time has not been appropriately allocated to the various approaches.

2.7 Simulation Model Software

The use of computerized traffic simulation models and analysis programs has developed from a strictly research device to a very valuable tool in modern traffic engineering practice (Taylor and Wolshon, 2001). Today there is a variety of engineering analysis tools/softwares available for analysis and modeling of an existing or proposed signalized intersection system. For instance, there are packages such as HCS, TRANSYT-7F, SIDRA, SYNCHRO, Sim Traffic, CORSIM, etc (Freeman et al, 2000). While the majority of the analysis tools provide the engineer with valuable information pertaining to intersection system delays, queue length, saturation flow, levels of service, etc, there is no single analysis software that could accurately predict all this valuable information for an actual or proposed field condition. For example, when signalized intersections are closely spaced, some software analysis techniques are good at predicting delays only and weak at predicting other valuable information such as queue length or queue spill back.

In this study the Synchro 6 Trafficware plus Sim Traffic with the attached Intersection Capacity Utilization spreadsheet were used for the analyses of the various traffic scenarios of the selected signalized intersections. The choice of this package is based on its appropriate abilities among the others as revealed by the work of Freeman et al, (2000).

Synchro uses HCM techniques and considers the same factors as the HCS (Highway Capacity Software). HCS is a program based on the HCM and its primary function is to analyze capacity and provide level of service for isolated intersections. In addition, traffic signal offsets and random traffic variations are factored into the computational procedure. Synchro uses traffic signal optimization procedures that can evaluate existing traffic signal timing conditions and optimize proposed signal timing conditions. The procedure provides output that includes average and 95th percentile queues, delays, stops, fuel consumption, and percent of time that queues exceed available storage.

Some basic characteristics and capabilities of the Synchro 6 Trafficware plus SimTraffic package are described in the following sections.

2.7.1 Synchro 6 Trafficware plus Sim Traffic

Synchro 6 Trafficware plus SimTraffic is a complete software package for modeling and optimizing traffic signal timings. Among the numerous features is its Intersection Capacity Analysis. Synchro 6 implements the Intersection Capacity Utilization (ICU 2003) method for determining intersection capacity. This method compares the current volume to the intersections ultimate capacity. The method is very straightforward to implement and can be determined with a single page worksheet.

Synchro 6 also implements the methods and guidelines of the HCM (2000), especially for Chapters 15, 16, and 17; Urban Streets, Signalized Intersections, and Unsignalized Intersections (Hush and Albeck, 2004). Synchro provides an easy-to-use solution for single intersection capacity analysis and timing optimization.

Synchro 6 also includes a term for queue interaction blocking delay. A new Total Delay will include the traditional control delay plus the new blocking delay. Delay calculations are an integral part of the optimization objective in Synchro. In addition to calculating capacity, Synchro can also optimize cycle lengths and splits, eliminating the need to try multiple timing plans in search of the optimum.

2.7.2 Intersection Capacity Utilization (ICU)

Husch and Albeck (2003) have developed a simple yet powerful tool for measuring an intersection's capacity dubbed 2003 Intersection Capacity Utilization, ICU (2003). The ICU is based on the principles and guidelines of HCM. It can be calculated using a single page worksheet, that is both easy to generate and easy to review. The ICU is a useful tool for planning applications such as roadway design and traffic impact studies.

The method sums the amount of time required to serve all movements at saturation for a given cycle length and divides by that reference cycle length. This method is similar to taking a sum of critical volume to saturation flow ratios (v/s), yet allows minimum timings to be considered. The ICU tells how much reserve capacity is available or how much the intersection is overcapacity.

The ICU is timing plan independent, yet has rules to insure that minimum timing constraints are taken into account. This removes the choice of timing plan from the capacity results.

According to the developers, one of the key applications is for traffic impact studies, future roadway design, and congestion management programs. The primary output from ICU (2003) is similar to the intersection volume to capacity ratio. Some of the benefits to using ICU (2003) over delay-based methods include greater accuracy, and a clear image of the intersection's volume to capacity ratio.

The ICU Level of Service (LOS) gives insight into how an intersection is functioning and how much extra capacity is available to handle traffic fluctuations and incidents. The ICU gives a good meaning on the conditions that exist or can be expected at the intersection. Categorizations from letters A to H are used to describe the level of an Intersection's Capacity Utilization. Note that the LOS for ICU (2003) includes additional levels past F to further differentiate congested operation.

Table 2-1 Level of Service Criteria for ICU Analysis

ICU	Level of Service
0 to 55%	A
>55% to 64%	B
>64% to 73%	C
>73% to 82%	D
>82% to 91%	E
>91% to 100%	F
>100% to 109%	G
>109%	H

A brief description of the conditions expected for each ICU LOS follows:

LOS A, [ICU <55%]: The intersection has no congestion. A cycle length of 80 seconds or less will move traffic efficiently. All traffic should be served on the first cycle. Traffic fluctuations, accidents, and lane closures can be handled with minimal congestion. This intersection can accommodate up to 40% more traffic on all movements.

LOS B, [55%< ICU< 64%]: The intersection has very little congestion. Almost all traffic will be served on the first cycle. A cycle length of 90 seconds or less will move traffic efficiently. Traffic fluctuations, accidents, and lane closures can be handled with minimal congestion. This intersection can accommodate up to 30% more traffic on all movements.

LOS C, [64%< ICU< 73%]: The intersection has no major congestion. The majority of traffic should be served on the first cycle. A cycle length of 100 seconds or less will move traffic efficiently. Traffic fluctuations, accidents, and lane closures may cause some congestion. This intersection can accommodate up to 20% more traffic on all movements.

LOS D, [73%< ICU< 82%]: The intersection normally has no congestion. Most of the traffic should be served on the first cycle. A cycle length of 110 seconds or less will move traffic efficiently. Traffic fluctuations, accidents, and lane closures can cause significant congestion. Sub optimal signal timings can cause congestion. This intersection can accommodate up to 10% more traffic on all movements.

LOS E, [82%< ICU< 91%]: The intersection is right on the verge of congested conditions. Many vehicles are not served on the first cycle. A cycle length of 120 seconds is required to move all traffic. Minor traffic fluctuations, accidents, and lane closures can cause significant congestion. Sub-optimal signal timings can cause significant congestion. This intersection has less than 10% reserve capacity available.

LOS F, [91%<ICU< 100%]: The intersection is over capacity and likely experiences congestion periods of 15 to 60 consecutive minutes. Residual queues at the end of green are common. A cycle length over 120 seconds is required to move all traffic. Minor traffic fluctuations, accidents, and lane closures can cause increased congestion. Suboptimal signal timings can cause increased congestion.

LOS G, [100%< ICU< 109%]: The intersection is up to 9% over capacity and likely experiences congestion periods of 60 to 120 consecutive minutes. Long queues are common. A cycle length over 120 seconds is required to move all traffic. Motorists may be choosing alternate routes, if they exist, or making fewer trips/during the peak hour. Signal timings can be used to distribute capacity to the priority movements.

LOS H, [109%<ICU]: The intersection is 9% or greater over capacity and could experience congestion periods of over 120 minutes per day. Long queues are common. A cycle length over 120 seconds is required to move all traffic. Motorists may be choosing alternate routes, if they exist, or make fewer trips during the peak hour. Signal timings can be used to distribute capacity to the priority movements.

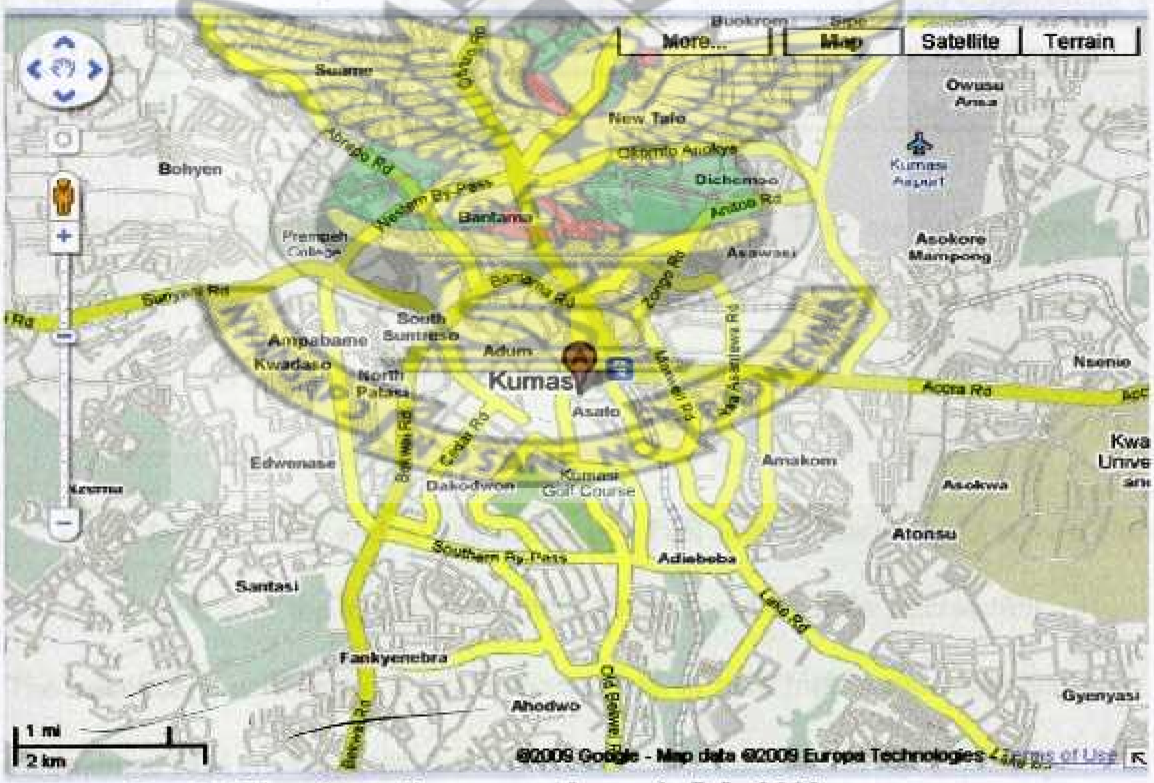
If intersections have LOS E or worse, queues between intersections can lead to blocking problems and spillbacks. Such problems could be addressed by indepth analyzes of the signal timing plans with microscopic simulation (Husch and Albeck, 2003).

3.0 RESEARCH METHODOLOGY

3.1 Description of Study Area

Kumasi Metropolis, the second city of Ghana, is the study area. The road network of the area is radial. There are all kinds of intersections including roundabouts, unsignalized and signalized. Currently, there are some 27 active signalized intersections in the Kumasi Metropolis which are serving various classes of traffic. These are scattered all over the road network and are mainly located at 4- and 3-legged intersections.

Figure 3-1 shows the road network of the Kumasi Metropolis.



Source: Google Maps (http://maps.google.com), July 2009

Figure 3-1 The central portion of the Road Network of Kumasi Metropolis

3.2 Site Selection

Nine (9) of the existing signalized intersections have at least one Fuel Service Station located within 50 meter radius of the intersection. These were considered candidate sites for the study as listed in Table 3-1.

Table 3-1 List of Candidate Sites

Intersection Local Name	Type	FSS Company	Land Use
Anloga	╬	Total	High-Commercial
Amakom	╬	Shell and Shell	Low-Commercial
KMA	╦	Shell	Non-Commercial
Aboabo	╬	O'ando	Low-Commercial
Abrepo Jct	╬	Shell	High-Commercial
Krofrom	╬	Shell	High-Commercial
Neoplan Asafo	╦	Other	High-Commercial
Texas	╦	Other	Low-Commercial
STC-Opoku Transport	╬	Top	Low-Commercial

Four (4) of the candidate sites were selected for this study. They are Anloga, Krofrom and Aboabo crossroads and the 3-legged KMA signalized intersections. The selection of these sites was based on the following criteria:

- Intersection legs are typically of 3- or 4- legs at fairly right angles
- At least an Entry and/or Exit access of a Fuel Service Station links at least one of the legs of the intersection within 50 meter radius of the intersection

The selection was done for a fair representation of both high-commercial and low-commercial road environment. A brief description of the selected sites follows:

3.3 Description of Selected Sites

3.3.1 The Anloga Signalized Intersection

The Anloga signalized intersection is located on one of the main arterials of Kumasi, the 24th February Road. The surrounding area is characterized by mixed landuse patterns including light industrial wood processing shops and diverse on-street commercial activities. The intersection's four legs are; the KNUST, Adum, Airport R/A and Anloga.

A TOTAL Fuel Service Station (FSS) is located between the Airport R/A and Adum legs. Services offered by the fuel station include fuel, lubrication, vulcanization, as well as a grocery shop. This FSS has an access on the Airport R/A approach and two accesses onto the exit lanes for the KNUST approach. A taxi rank is found right in the vicinity of the Fuel service station as shown in the Figure 3-2.

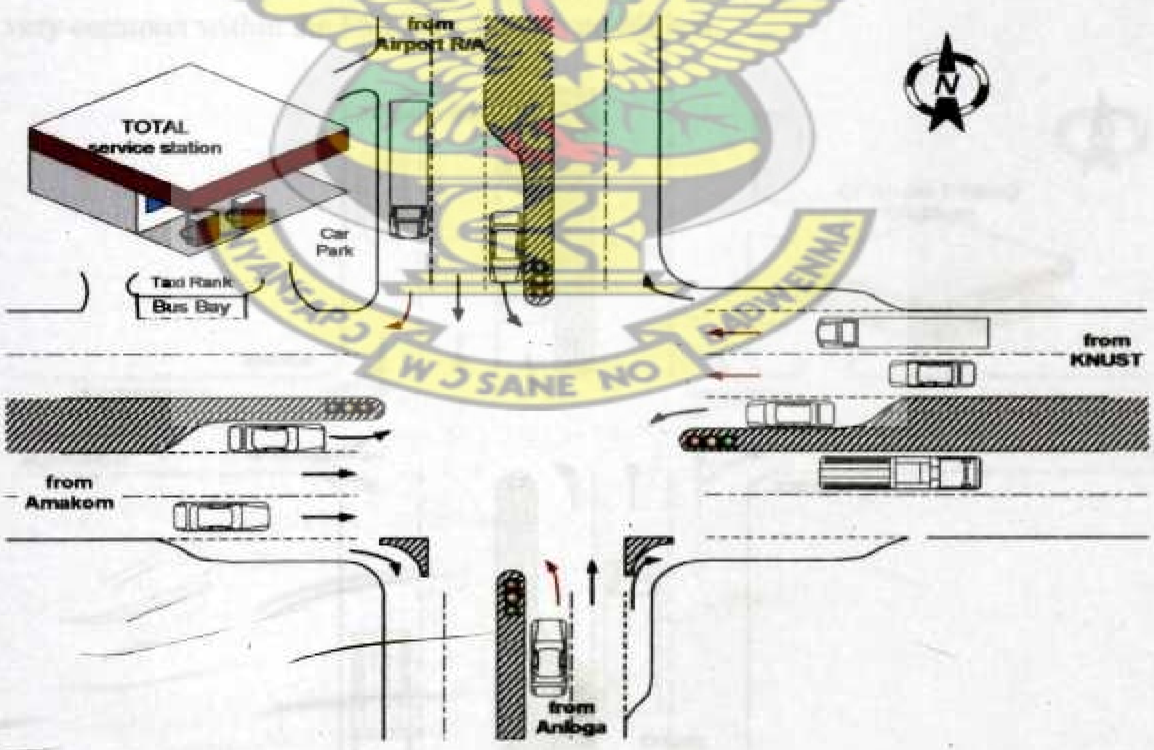


Figure 3-2 General Layout of Anloga Signalized Intersection

3.3.2 The Aboabo Signalized Intersection

The Aboabo signalized intersection has 4-legs and relatively low roadside commercial activity. It is located in a residential area. The intersection is relatively on a high ground with its Airport R/A leg sloping down. The other three legs, namely Anloga, Aboabo, and Asokore Mampong slope from the intersection more gently.

An O'ANDO Fuel Service Station which mainly offers only the fuel and vulcanizing services is located in the area between Asokore Mampong and Airport R/A legs. This FSS has an access on the Asokore Mampong approach and two accesses onto the exit lanes for the Anloga approach. Currently one of the latter two accesses has been blocked with bollards as shown in Figure 3-3. Passenger loading by taxi cab drivers is very common within the FSS area, though prohibited.

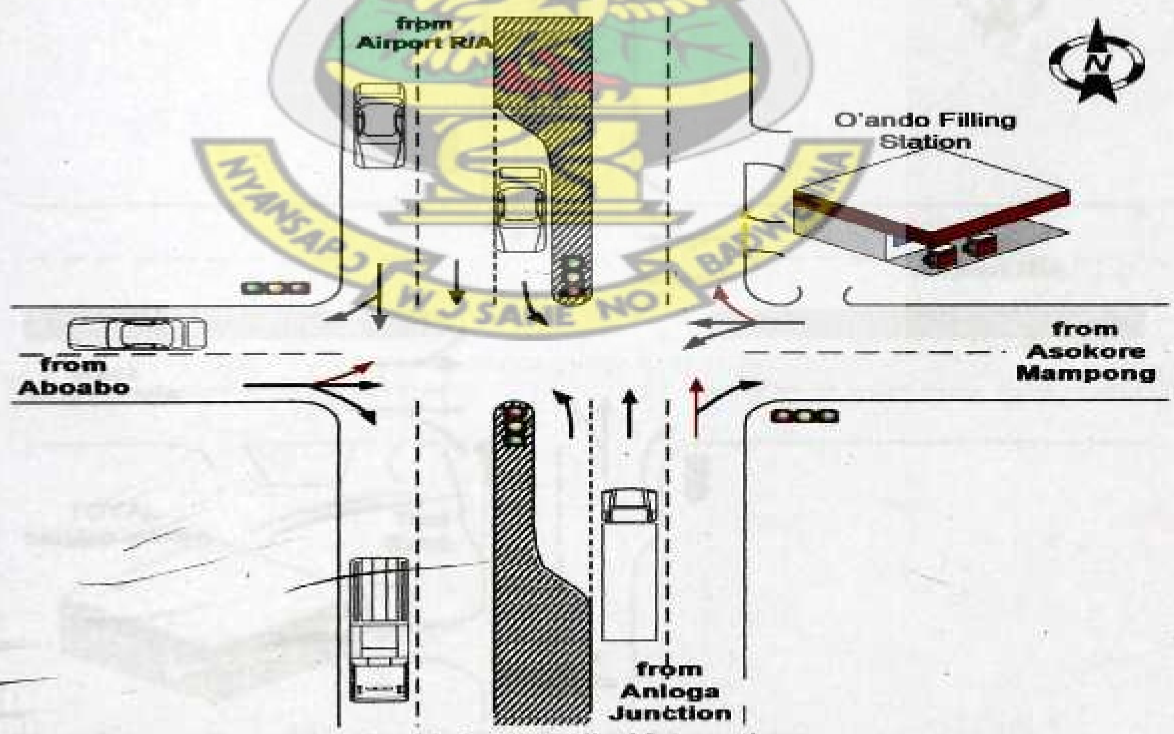


Figure 3-3 General Layout of Anloga Signalized Intersection

3.3.3 The Krofrom Signalized Intersection

Located in a mixed residential cum commercial area, the Krofrom signalized intersection is a 4-leg junction on the Western Ring aeterial of Kumasi. The signalized intersection's four legs are Suame R/A, New Tafo, Airport R/A, and Kejetia.

A TOTAL Fuel Service Station is located between the Suame R/A and Kejetia legs. Unlike the Aboabo and Anloga FSS, this station has two accesses on the Suame R/A approach and an access onto the exit lane for the New Tafo approach. The businesses of this station include fuel and lubrication, vulcanization, and a grocery shop. A taxi rank is found right in the confines of the Fuel service station as shown in Figure 3-4.

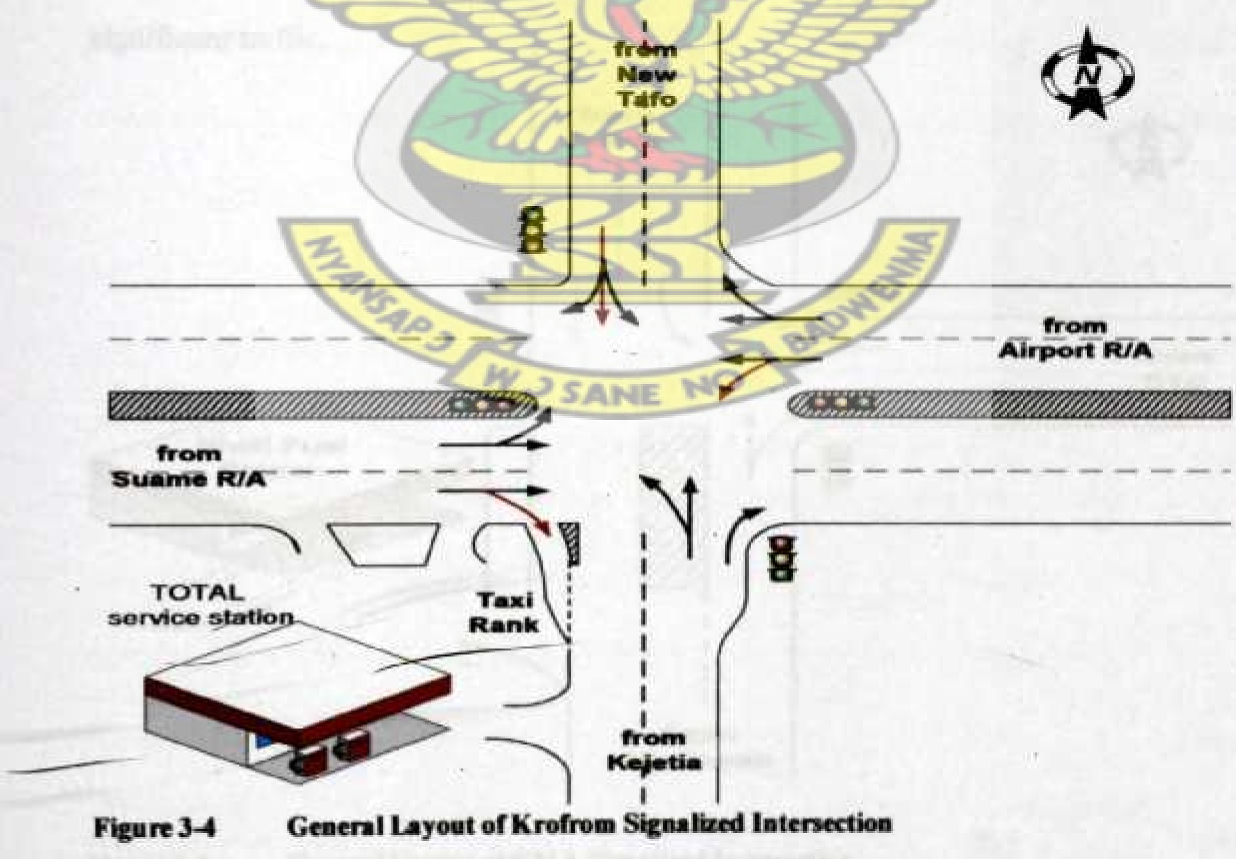


Figure 3-4 General Layout of Krofrom Signalized Intersection

3.3.4 The KMA Signalized Intersection

The KMA signalized intersection has 3-legs and located close to the administrative area of the Kumasi Metropolis. Thus there is minimal commercial activity around except for a few street hawkers. The three legs which typically form a 'tee' are the Adum and Nhyiaeso legs forming the main road and STC leg as the minor road.

A SHELL Fuel Service Station is located on the side of the main road with two accesses. Unlike the other three signalized intersections there is no taxi rank located within the confines of the fuel service station and hence there is almost no commercial vehicle based traffic activity at the accesses. However, the station offers well patronized grocery shop besides the fuel and lubrication services which attract significant traffic.

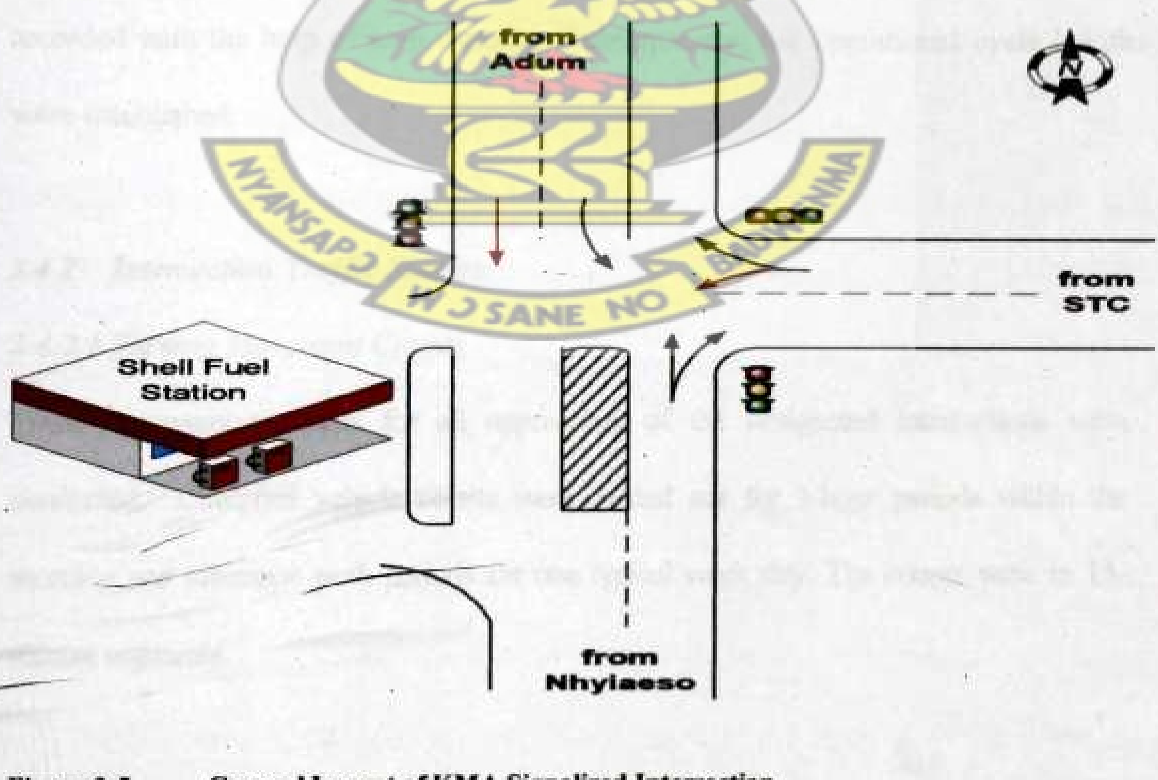


Figure 3-5 General Layout of KMA Signalized Intersection

3.4 Field Studies and Observations

3.4.1 Inventory of Intersections

Using the guidelines put forward by Garber and Hoel (1994), lane groups were identified on all the approaches of the study sites. Vehicle movements thought to be potentially affected by the traffic activities on the FSS' accesses have been highlighted in red on the geometric layouts (refer to Figures 3-2 to 3-5). Lane widths were also measured using cyclometer and tape measures. The approach grades were also estimated from the differences in ground levels and their corresponding horizontal distances apart.

Each intersection's signal system was identified and noted with the corresponding phase sequences. The split times of the signal phases were further observed and recorded with the help of stop watches. Subsequently, the operational cycle lengths were established.

3.4.2 Intersection Traffic Studies

3.4.2.1 Turning Movement Counts

Turning movement counts for all approaches of the designated intersections were conducted. Classified vehicle counts were carried out for 3-hour periods within the morning and afternoon peak periods for one typical week day. The counts were in 15-minute segments.

3.4.2.2 Saturation Flow Rates of Selected Lane Groups

To establish the field saturated flow rate of particular lane groups, the number of vehicles crossing the stop line while there was a queue were counted in accordance with the Garber and Hoel's procedures (see section 2.3.5 of the Literature Review). The actual saturated flow rates were then computed using the formula of Equation 2.3 [page 10].

3.4.2.3 Vehicular Traffic at the Fuel Service Stations

Vehicular traffic to and from the fuel service stations were captured on video and later analyzed in the playback mode on a computer. These videos were captured in 15-minute slots and were conducted concurrently with the vehicle turning movement counts. This was done to establish the average vehicle maneuvers per hour at the accesses of the FSS.

3.4.2.4 Vehicles (local buses and taxis) Stoppages at FSS Accesses

From the videos the average vehicle stoppages per hour on areas of the accesses that were being used as bus bays were also extracted. These stoppages tend to impede the traffic flows.

3.4.2.5 Intersection Delay Studies: - Lost Times and Control Delays

To establish the actual lost times the respective clearance distances and average speeds of travel through the intersections for the respective movements were taken. This was accomplished with the use of cyclometer and radar speed gun for the instantaneous speeds of vehicles as they traverse the central area of the intersections. The assumption is that the instantaneous speed of a vehicle at the central intersection area is representative of the average speed of travel through the intersections. The actual lost times were subsequently computed using the formula $(2 + D_i/S_i)$ as contained in Equation 2.4 [page 14].

The delays at an intersection; total average delay, average delay per stopped vehicle, average delay per approach, and percent of vehicles stopped were computed from intersection delay studies. This involved the counting of vehicles stopped in the intersection approach at successive intervals. A typical duration for these intervals was 30-seconds and was selected so as not to be a multiple of the traffic signal cycle length, as recommended by guidelines for intersection delay study contained in the Manual on Uniform Traffic Studies (2000). An observer counts and records the number of vehicles stopped on the approach for each sampling interval. It should be noted that a vehicle was counted more than once if it was stopped during more than one sampling time. A second observer performed a separate tabulation of the approach volume for each time period by classifying the vehicles as either stopped or not stopping.

3.5 Performance Assessment of Intersections

The collected field data were organized to form the basic input parameters for the partial calibration of the *Synchro 6 Trafficware plus Sim Traffic* software. The data were then used to simulate the existing situation for analysis. Hypothetical traffic scenarios representing the cases of the absence of the FSS were also simulated for comparison with the base (existing) situations. The impact on the saturation flow rates and lost times were assessed. Delays were the main measures of effectiveness (MOE) used for the assessments in this study. The assessments were conducted by comparing the MOE from the hypothetical traffic scenarios with the existing traffic situations at the signalized intersections.

4.0 ANALYSIS OF RESULTS

4.1 Inventory of the Intersections

4.1.1 Anloga Signalized Intersection

All the four legs of the Anloga Signalized intersection have dedicated turning lanes with the respective storage lengths as indicated in Table 4-1. The KNUST and Amakom approaches have dual lanes serving the through traffic. All other lanes are single. With the exception of the Anloga approach which is relatively level with the intersection central area, the other three legs slope into the intersection.

Table 4-1 Geometric Data of Anloga Signalized Intersection

From	To	Movement Code	Approach Grade (%)	No. of Lanes	Lane Width (*m*)	Storage Length (*m*)
KNUST	Anloga	WBL	- 5.0	1	3.0	75.0
	Amakom	WBT		2	3.4	N/A
	Airport R/A	WBR		1	3.0	75.0
Amakom	Airport R/A	EBL	- 3.0	1	3.4	86.0
	KNUST	EBT		2	3.3	N/A
	Anloga	EBR		1	4.7	54.0
Anloga	Amakom	NBL	0	1	4.8	40.0
	Airport R/A	NBT		1	4.8	N/A
	KNUST	NBR		1	4.7	34.0
Airport R/A	KNUST	SBL	- 1.0	1	3.0	65.0
	Anloga	SBT		1	3.8	N/A
	Amakom	SBR		1	3.8	58.0

Note: N, S, E, and W represent the four cardinal points whereas L, T, and R represent Left turning, Through traffic and Right turning respectively. For instance WBL is read as West Bound Left turn, which for instance, represents the movement from the KNUST approach to Anloga. Thus all movements emanating from KNUST approach are West bound with their respective turns.

The Anloga intersection is a pretimed 4-phase system consisting of protected and split movements. The cycle length is 212 seconds. Figure 4-1 displays the current operational system.

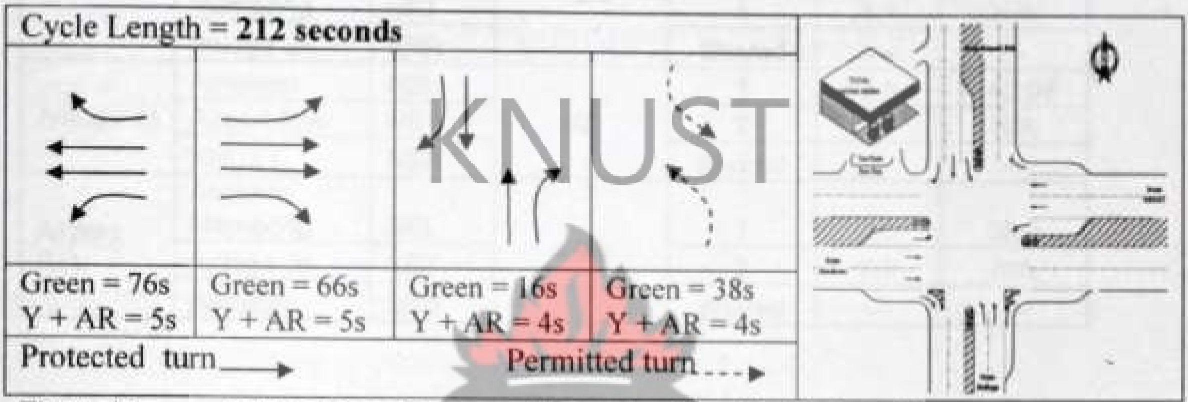


Figure 4-1 Existing Phase Diagram of the Anloga Signalized Intersection

4.1.2 Aboabo Signalized Intersection

The Airport R/A and Anloga junction approaches have left turning filter lanes of storage lengths 38.0m and 40.0m respectively. Whereas the Anloga Junction approach is relatively flat to the intersection, the other three approaches slope away with their grades shown in Table 4-2. The Aboabo and Asokore Mampong approaches have single shared lanes of standard width. The widths of the lanes on the major road are less than the recommended standard of 3.65m

Table 4-2 Geometric Data of Aboaba Signalized Intersection

From	To	Movement Code	Approach Grade (%)	No. of Lanes	Lane Width (*m*)	Storage Length (*m*)
Asokore Mampong	Anloga Jn	WBL	+ 1.0	Shared	---	---
	Aboabo	WBT		1	3.7	N/A
	Airport R/A	WBR		Shared	---	---
Aboabo	Airport R/A	EBL	+ 2.0	Shared	---	---
	Asokore Mampong	EBT		1	3.8	N/A
	Anloga Jn	EBR		Shared	---	---
Anloga Jn	Amakom	NBL	0	1	2.5	40.0
	Airport R/A	NBT		2	3.0	N/A
	KNUST	NBR		Shared	---	---
Airport R/A	Asokore Mampong	SBL	+ 5.0	1	2.5	38.0
	Anloga Jn	SBT		2	2.9	N/A
	Aboabo	SBR		Shared	---	---

This intersection currently operates as a pretimed 3-phase system with both protected and permitted movements. Its current cycle length is 120 seconds. Figure 4-2 displays the current phase diagram.

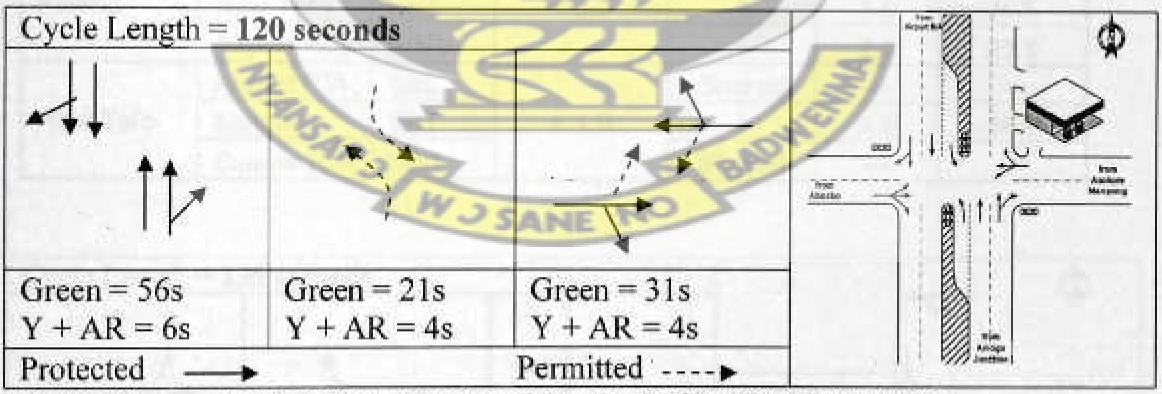


Figure 4-2 Existing Phase Diagram of the Aboabo Signalized Intersection

4.1.3 Krofrom Signalized Intersection

The main road of the Krofrom Signalized intersection consists of the Airport R/A and the Suame R/A approaches, and are relatively flat with no dedicated turning lanes. They have dual shared lanes of widths 3.5m and 3.2m respectively. As shown in Table 4-3, the New Tafo approach has a single shared lane of width 4.8m while the Kejetia approach has a short right turn filter lane of width 2.8m in addition to a 3.7m lane. These two approaches slope away from the intersection. The intersection is currently operating a 3-phase pretimed signal system as shown in Figure 4-3.

Table 4-3 Geometric Data of Krofrom Signalized Intersection

From	To	Movement Code	Approach Grade (%)	No. of Lanes	Lane Width (*m*)	Storage Length (*m*)
Airport R/A	Kejetia	WBL		Shared	---	---
	Suame R/A	WBT	0	2	3.5	N/A
	New Tafo	WBR		Shared	---	---
Suame R/A	New Tafo	EBL		Shared	---	---
	Airport R/A	EBT	0	2	3.2	N/A
	Kejetia	EBR		Shared	---	---
Kejetia	Suame R/A	NBL		Shared	---	---
	New Tafo	NBT	+ 2.0	1	3.0	N/A
	Airport R/A	NBR		1	2.8	23.0
New Tafo	Airport R/A	SBL		Shared	---	---
	Kejetia	SBT	+ 1.0	1	4.8	N/A
	Suame R/A	SBR		Shared	---	---

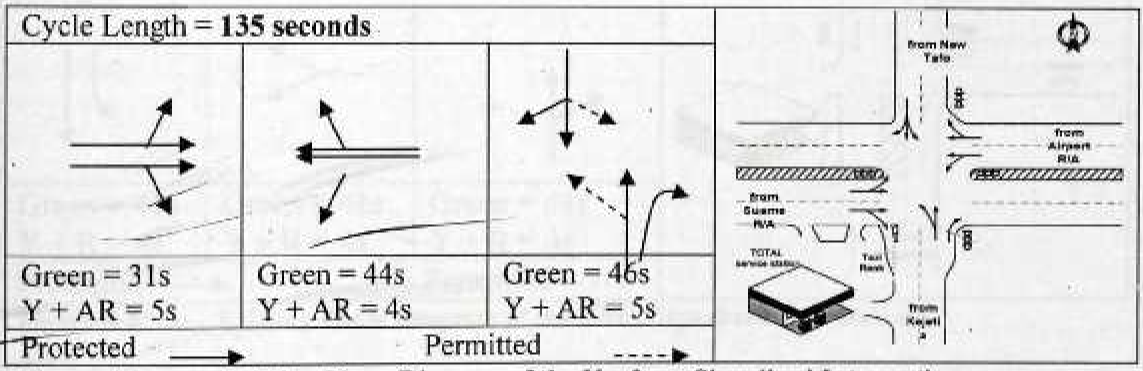


Figure 4-3 Existing Phase Diagram of the Krofrom Signalized Intersection

4.1.4 KMA Signalized Intersection

The Adum approach has a dedicated left turn lane of storage length 21.5m. Nhyieso and the STC approaches have single shared lanes of widths 3.6m and 3.4m respectively. The main road being the Adum and Nhyieso legs are on a slope resulting in the grades as displayed in Table 4-4. The STC leg is relatively flat to the intersection.

Table 4-4 Geometric Data of KMA Signalized Intersection

From	To	Movement Code	Approach Grade (%)	No. of Lanes	Lane Width (*m*)	Storage Length (*m*)
STC	Nhyiaeso	WBL	0	1	3.4	---
	Adum	WBR		Shared	---	---
Nhyiaeso	Adum	NBT	+ 3.0	1	3.6	N/A
	STC	NBR		Shared	---	---
Adum	STC	SBL	- 2.0	1	3.5	21.5
	Nhyieso	SBT		1	3.6	---

The system is a 3-phase system with both the protected and permitted movements. Figure 4-4 displays the current system

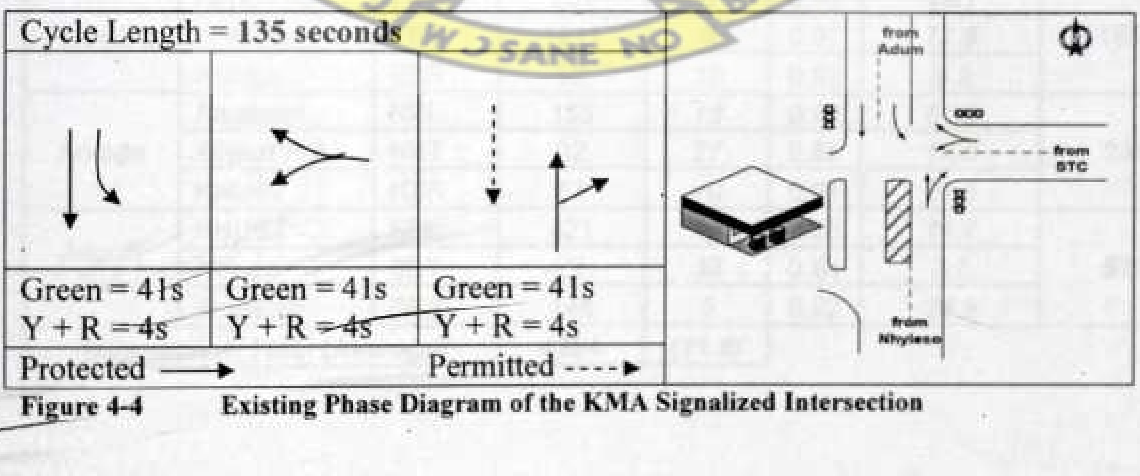


Figure 4-4 Existing Phase Diagram of the KMA Signalized Intersection

4.2 Existing Traffic Characteristics

4.2.1 Turning Movements at the Intersections

4.2.1.1 Anloga Signalized Intersection

The proportion of through traffic on the major approaches- KNUST and Amakom are quite high in excess of 77% of their respective volumes. The discharge of through traffic on the KNUST approach experience some impedance on the access to the FSS. On the other hand the left turners on the minor approaches- Anloga and Airport R/A are high, constituting about 65% and over 70% of their respective approach volumes. It must be noted that the Airport R/A volume of 587 veh/hr excludes traffic that by-passed the intersection through the FSS.

Table 4-5 **Anloga Intersection Turning Movement Counts**

From	To	Move-ment Code	Traffic Flow (Veh/hr)	% Heavy Veh.	PHF	% of Approach Traffic Flow	Approach Traffic Flow (Veh/hr)
KNUST	Anloga	WBL	27	*13*	0.89	*1.4*	**1907**
	Amakom	WBT	1483	*3*	0.96	*77.8*	
	Airport R/A	WBR	397	*4*	0.90	*25.5*	
Amakom	Airport R/A	EBL	291	*8*	0.91	*18.7*	**1554**
	KNUST	EBT	1211	*5*	0.97	*77.9*	
	Anloga	EBR	52	*10*	0.92	*3.3*	
Anloga	Amakom	NBL	153	*13*	0.92	*64.8*	**236**
	Airport R/A	NBT	32	*27*	0.88	*13.6*	
	KNUST	NBR	51	*10*	0.91	*21.6*	
Airport R/A	KNUST	SBL	421	*10*	0.90	*71.7*	**587**
	Anloga	SBT	22	*33*	0.94	*3.7*	
	Amakom	SBR	144	*5*	0.92	*24.5*	
Intersection Total (*Average*)			**4284**	***(11.8)***			

4.2.1.2 Aboabo Signalized Intersection

The Anloga Junction approach has the highest volume (1457 veh/hr) with a proportion of over 80% (1211 veh/hr) being through traffic, as shown in Table 4-6. These through traffickers encounter some impedances as a result of traffic activities on the FSS access downstream the Anloga Junction approach.

The WBR movement (i.e. Asokore Mampong to Airport R/A) of the Asokore Mampong approach has a volume of 149 veh/hr representing 47.3% of the approach. In actual fact this excludes by-passers of the intersection through the FSS access on the Asokore Mampong.

Table 4-6 **Aboabo Intersection Turning Movement Counts**

From	To	Move-ment Code	Traffic Flow (Veh/hr)	% Heavy Veh.	PHF	% of Approach Traffic Flow	Approach Traffic Flow (Veh/hr)
Asokore Mampong	Anloga Jn	WBL	62	*8*	0.92	*19.7*	**315**
	Aboabo	WBT	104	*6*	0.92	*33.0*	
	Airport R/A	WBR	149	*12*	0.92	*47.3*	
Aboabo	Airport R/A	EBL	51	*9*	0.90	*24.9*	**205**
	Asokore Mampong	EBT	99	*13*	0.90	*48.3*	
	Anloga Jn	EBR	55	*11*	0.90	*26.8*	
Anloga Jn	Amakom	NBL	187	*6*	0.92	*12.8*	**1457**
	Airport R/A	NBT	1211	*9*	0.93	*83.1*	
	KNUST	NBR	59	*10*	0.93	*4.0*	
Airport R/A	Asokore Mampong	SBL	103	*13*	0.92	*10.2*	**1012**
	Anloga Jn	SBT	838	*8*	0.95	*82.8*	
	Aboabo	SBR	71	*8*	0.95	*7.0*	
Intersection Total (*Average*)			**2989**	***(9.4)***			

4.2.1.3 Krofrom Signalized Intersection

The Airport R/A and Suame R/A approaches are the main roads of the Krofrom signalized intersection. The relatively low volume of 892 veh/hr on the Suame R/A approach compared with that of Airport R/A (1553 veh/hr), as shown in Table 4-7, is attributed to the high number of right turning through the FSS rather than through the intersection.

The New Tafo approach is a single shared lane which serves a wide range of vehicle classes with relatively high percentage of heavy vehicles that are turning. Especially so with the turning towards the Airport R/A [SBL] direction, heavy vehicles accounting for 22% (98 veh/hr) of volume on that approach as shown in Table 4-7. That therefore explains the relatively low traffic flow of 283 veh/hr on the New Tafo approach.

Table 4-7 Krofrom Intersection Turning Movement Counts

From	To	Move-ment Code	Traffic Flow (Veh/hr)	% Heavy Veh.	PHF	% of Approach Traffic Flow	Approach Traffic Flow (Veh/hr)
Airport R/A	Kejetia	WBL	202	*1*	0.91	*13.0*	**1553**
	Suame R/A	WBT	784	*11*	0.94	*50.5*	
	New Tafo	WBR	567	*20*	0.92	*36.5*	
Suame R/A	New Tafo	EBL	111	*13*	0.90	*12.4*	**892**
	Airport R/A	EBT	684	*11*	0.93	*76.7*	
	Kejetia	EBR	97	*10*	0.88	*10.9*	
Kejetia	Suame R/A	NBL	154	*9*	0.91	*19.1*	**807**
	New Tafo	NBT	249	*7*	0.91	*30.9*	
	Airport R/A	NBR	404	*4*	0.91	*50.1*	
New Tafo	Airport R/A	SBL	98	*22*	0.92	*34.6*	**283**
	Kejetia	SBT	144	*10*	0.92	*50.9*	
	Suame R/A	SBR	41	*6*	0.92	*14.5*	
Intersection Total (*Average*)			**3535**	***(10.3)***			

4.2.1.4 KMA Signalized Intersection

Unlike the other three intersections which are located on arterials of the city, this intersection is located in the central business district (CBD) and is close to the administrative area of the Kumasi metropolis. Thus it serves mainly local traffic. The proportion of heavy vehicles is relatively small with an overall heavy vehicle percentage of 2.3%, as shown from Table 4-8. It must be noted the location of the FSS at the intersection does not facilitate by-passing of the intersection through to the FSS.

Table 4-8 **KMA Intersection Turning Movement Counts**

From	To	Move-ment Code	Traffic Flow (Veh/hr)	% Heavy Veh.	PHF	% of Approach Traffic Flow	Approach Traffic Flow (Veh/hr)
STC	Nhyiaeso	WBL	223	*4*	0.92	*39.1*	**570**
	Adum	WBR	347	*1*	0.92	*60.9*	
Nhyiaeso	Adum	NBT	308	2	0.93	*54.0*	**587**
	STC	NBR	279	3	0.93	*48.9*	
Adum	STC	SBL	337	2	0.92	*59.1*	**725**
	Nhyieso	SBT	388	*1*	0.91	*68.1*	
Intersection Total (*Average)*			**1882**	***(2.3)***			

4.2.2 *Saturation Flow Rates of Selected Lane Groups at the Intersections*

The field saturation flows for the lane groups on the major roads and some selected vehicle movements of interest especially those thought to be affected by the traffic activities of the FSS were estimated using the method recommended by Garber and Hoel (1994). Table 4-9 shows the field measurements and the computed saturated flow rates in vphgpl. The variables N, t_n and t_4 are as contained in Equation 2.2, for the estimation of the saturation flow rates (S).

Table 4-9 Field Estimation Results for Saturation Flow Rates

Intersection	Approach	Movement Code	N	t_n	t_4	S
			Veh (v)	Sec (s)	Sec (s)	vphgpl
Anloga	KNUST	WBT	31	82	10	**1350**
	Amakom	EBT	33	75	9	**1582**
	Anloga	NBL	12	34	9	**1152**
	Airport R/A	SBR	17	41	8	**1418**
Aboabo	Asokore Mampong	WBT	14	33	7	**1385**
	Aboabo	EBT	13	33	8	**1296**
	Anloga Jn	NBT	22	58	8	**1296**
	Airport R/A	SBT	24	58	9	**1469**
Krofrom	Airport R/A	WBT	21	49	8	**1493**
	Suame R/A	EBT	15	34	8	**1523**
	Kejetia	NBT	19	47	9	**1421**
	New Town	SBT	16	43	10	**1309**
KMA	STC	WBL	21	45	7	**1611**
	Nhyiaeso	NBT	21	43	8	**1749**
	Adum	SBT	18	44	7	**1362**

As shown from Table 4-9, the computed saturation flow rates ranged between 1150 and 1750 vphgpl. These field saturation flow rates are quite are comparable with the results of Adams and Obri-Yeboah (2008) on some selected signalized intersections in Kumasi. It is worthy to note that the geometric and traffic characteristics of the approaches had effect on their respective saturation flows. For instance, the New

Town approach of Krofrom signalized intersection is a single shared lane serving movements of significant proportions of heavy vehicles. This resulted in the low field saturation flow rates of 1137 vphgpl for the New Tafo approach, as shown in Table 4-9.

4.2.3 *Vehicle Maneuvers through the Fuel Service Stations*

The orientation of the accesses to the FSS at the Anloga intersection facilitates by-passing of the intersection when travelling from the Airport R/A approach towards the Amakom direction [SBR]. Over 90% of vehicle entries to the FSS (229 veh/h) were those who by-passed the intersection. Of these motorists using the FSS as a by-pass, taxis drivers accounted for more than half (116 entries in an hour) as shown in Table 4-10. Vehicle entries to the FSS for the purpose of doing business is rather relatively low; only 18 in an hour which is less than 10% of the total vehicle entries.

Table 4-10 Vehicle Maneuvers through FSS at Anloga Intersection

Vehicle Class	Passing Through N (Veh/h)	Passing Through %	To Do Business N (Veh/h)	To Do Business %	Total Entries (Veh/h)	Percentage of Total Veh. Entries (%)
Car	53	93.0	4	7.0	57	23.1
Taxi	116	95.9	5	4.1	121	49.0
Bus	52	89.7	6	10.3	58	23.5
Other	8	72.7	3	27.3	11	4.5
Total	**229**	**92.7**	**18**	**7.3**	**247**	**100.0**

Similar to the situation at Anloga Intersection, there are a relatively high percentage of vehicle maneuvers through the O'ando FSS for the purpose of by-passing the Aboabo signalized intersection's central area. These motorists are predominantly those from the Asokore Mampong approach making a right turn to the Airport R/A direction [WBR]. As shown in Table 4-11, taxis account for the most vehicle maneuvers (72 veh/h) representing over 93% of motorists passing through the FSS.

Table 4-11 Vehicle Maneuvers through FSS at Aboabo Intersection

Vehicle Class	Passing Through N (Veh/h)	Passing Through %	To Do Business N (Veh/h)	To Do Business %	Total Entries (Veh/h)	Percentage of Total Veh. Entries (%)
Car	6	66.7	3	33.3	9	8.3
Taxi	72	93.5	5	6.5	77	70.6
Bus	14	70.0	6	30.0	20	18.3
Other	2	66.7	1	33.3	3	2.8
Total	94	86.2	15	13.8	109	100.0

At the Krofrom Signalized intersection, there is the possibility of by-passing the central intersection area for motorists on the Suame R/A approach intending a right turn towards the Kejetia direction [EBR]. As shown in Table 4-12, 170 vehicle entries in an hour out of the 214 are those making such maneuvers. Of these, taxis dominate with 108 maneuvers in an hour.

Table 4-12 Vehicle Maneuvers through FSS at Krofrom Intersection

Vehicle Class	Passing Through N (Veh/h)	Passing Through %	To Do Business N (Veh/h)	To Do Business %	Total Entries (Veh/h)	Percentage of Total Veh. Entries (%)
Car	43	67.2	21	32.8	64	29.9
Taxi	108	88.5	14	11.5	122	57.0
Bus	15	62.5	9	37.5	24	11.2
Other	4	100.0	0	0.0	4	1.9
Total	170	79.4	44	20.6	214	100.0

Unlike the other three signalized intersections, the location of the FSS does not facilitate by-passing of the intersection's central area from any of its approaches. All vehicle maneuvers through the FSS are therefore for some business. Table 4-13 shows the distribution of vehicle maneuvers to the FSS.

Table 4-13 Vehicle Maneuvers to (through) FSS at KMA Intersection

Vehicle Class	Vehicle Entries (*Veh/h*)	Percentage of Vehicle Entries (%)
Car	24	*42.1*
Taxi	17	*29.8*
Bus	6	*10.5*
Other	10	*17.5*
Total	**57**	***100.0***

4.2.4 Vehicle Stoppages (Bus Blockage) at Accesses to Fuel Service Stations

The average number of vehicles per hour which stopped on the accesses to the FSS thereby impeding the free flow of traffic through the intersections was collected.

It was observed that vehicles stopping on the accesses thereby impeding traffic flow were predominantly local buses and taxis, and they do not entirely block the lane. Hence these stoppages did not have the full effect of bus blockage as described in the literature search. For the purpose of this study, the number of vehicle stopping was thus adjusted based on the amount of lane space blocked and the vehicle class to reflect the full effect of bus blockage. On the average the local buses take one-half (0.5) of the lane as they stop whereas the taxis blocked about one-third (0.3). Lane blockage by other vehicle classes was ignored based on their high variability and marginal representation.

Equation (4.1) was then developed to make the necessary adjustments.

$$N_b = [0.5P_b + 0.3P_t]xN_b' \text{ .... Equation 4.1}$$

where N_b = adjusted number of vehicle stops in veh/h

N_b' = field (observed) number of vehicle stops in veh/h

P_b = proportion (%) of stoppages by local buses

P_t = proportion (%) of stoppages by taxis

For instance for a given one hour, 213 vehicle stopping with a distribution of 57% by local buses and 43% taxis means 121 local buses occupied half the lane space and 92 taxis took one-third the lane space which thus translates into an equivalent of 88 bus blockages. It must be noted that the adjustment is based on the lane space. The results are as presented in Table 4-14

Table 4-14 Vehicle Stoppages (Bus Blockage) at FSS Accesses Impeding Free Traffic Flow

Intersection	Movement	Actual No. of Stops (*veh/h*)	% Distribution By Vehicle Class		Adjusted No. of Stops (*veh/h*)
			Local Bus (P_b)	Taxi (P_t)	
Anloga	WBT	213	*57*	*43*	**88**
	NBL	80	*55*	*45*	**33**
	SBR	54	*43*	*57*	**21**
Aboabo	EBL	21	*39*	*61*	**8**
	WBR	62	*37*	*63*	**23**
	NBT	118	*33*	*67*	**43**
Krofrom	EBR	73	*35*	*65*	**27**
	WBL	109	*25*	*75*	**38**
	SBT	38	*33*	*67*	**14**
KMA	WBL	0	-	-	**0**
	SBT	1	*0*	*100*	**0**

4.2.5 Lost Times at the Intersections

The clearance distance of all the selected movements, being the stopline-to-stopline distance through the intersection's central point, were measured with a tape measure. Subsequently, the actual lost times for the major and other movements of interest were computed using the formula Lost Time = $(2 + D_i/S_i)$ seconds from Equation (3.1) and are presented in Table 4-15.

Table 4-15 Actual Lost Times at the Intersections

Approach	Movement	Average Speed *(km/h)*	Average Speed [S_i] *(m/s)*	Clearance Distance [D_i] *(m)*	Actual Lost Time *(sec)*
Anloga Signalized Intersection					
KNUST	WBT	22	6.11	30	6.9
Amakom	EBT	33	9.17	30	5.3
Anloga Jn	NBL	25	6.94	33	6.8
Airport R/A	SBR	18	5.00	18	5.6
Aboabo Signalized Intersection					
Asokore Mampong	WBR	18	5.00	20	6.0
Aboabo	EBL	25	6.94	35	7.0
Anloga Jn	NBT	28	7.78	30	5.9
Airport R/A	SBT	31	8.61	30	5.5
Krofrom Signalized Intersection					
Airport R/A	WBL	27	7.50	40	7.3
Suame R/A	EBR	22	6.11	25	6.1
Kejetia	NBT	27	7.50	32	6.3
New Town	SBT	28	7.78	38	6.9
KMA Signalized Intersection					
STC	WBL	27	7.50	25	5.3
Nhyieso	NBT	29	8.06	24	5.0
Adum	SBT	31	8.61	22	4.6

As described earlier, the default value for the lost time for a given movement is usually 4.0 seconds being 2.0 seconds apiece for start-up and clearance lost times. However in actual fact, the clearance time depends on the time required for vehicles to clear the intersection once the amber flashes for the green of the next movement. Because of the traffic impedances to certain flows, average clearance speeds are lower resulting in prolonged clearance times for those movements. Thus the actual lost times for certain movements, as shown in Table 4-15, are far greater than the allotted design periods of 4 seconds. This phenomenon tends to affect the successive set of movements as their effective green times are reduced.

4.2.6 *Delays at the Intersections*

The procedure for the delay studies were carried out as described in the methodology and the results are displayed in Table 4-16.

From Table 4-16, it is seen that the stopped delay per vehicle was quite high for certain movements in excess of 300 seconds. The percentage of stopped vehicles which gives the proportion of vehicles which were stopped at the intersections show that quite a high number of vehicles experience stoppage(s). The '*No. of Stops Per Vehicle*' of Table 4-16 shows that for most of the movements, vehicles experience at least one stop before clearing the intersection.

Table 4-16 **Delays at the Intersections**

Approach	Movement	Approach Volume (veh/h)	Percentage of Stopped Vehicles (%)	Delay per Stopped Vehicle (sec)	No. of Stops Per Vehicle (#/veh)
Anloga Signalized Intersection					
KNUST	WBT	1907	95.3	321	2.1
Amakom	EBT	1554	78.1	263	1.6
Anloga Jn	NBR	236	56.3	98	1.0
Airport R/A	SBR	587	63.4	288	0.9
Aboabo Signalized Intersection					
Asokore Mampong	WBT	315	71.6	121	1.2
Aboabo	EBT	205	65.0	59	1.0
Anloga Jn	NBT	1457	81.9	173	1.8
Airport R/A	SBT	1012	73.4	68	1.1
Krofrom Signalized Intersection					
Airport R/A	WBT	1553	89.1	338	1.9
Suame R/A	EBT	892	77.0	271	1.5
Kejetia	NBT	807	68.4	105	1.1
New Town	SBT	283	88.2	308	1.8
KMA Signalized Intersection					
STC	WBL	570	73.5	173	1.1
Nhyieso	NBT	585	82.3	154	1.3
Adum	SBT	725	69.7	88	0.9

4.3 Results of Simulation

4.3.1 Existing Traffic Situations

4.3.1.1 Anloga Signalized Intersection

The intersection is currently performing at the ICU LOS E and utilizes 86.3% of its capacity. The existing operational cycle length is 212 sec, and the average intersection delay is 283.7 seconds per vehicle. The KNUST [West bound] approach experiences the worst delays of an average of 410.6 seconds per vehicle. The intersection is right on the verge of congested conditions. Many vehicles are not served on the first cycle. Minor traffic fluctuations, accidents, and other traffic incidents cause significant congestion.

4.3.1.2 Aboabo Signalized Intersection

With a cycle length of 120 seconds, the Aboabo intersection is performing at an ICU LOS D. Its capacity utilization is 75.1% indicating a reserve capacity of almost 25%. Though the Anloga Junction [North bound] approach experiences the worst delays (142.1 sec/veh), the overall intersection delay is 95.7 seconds per vehicle. Occasionally the intersection experiences congestions which could indicate sub-optimal signal timings. Traffic fluctuations and other traffic incidents tend to cause significant congestion.

4.3.1.3 Krofrom Signalized Intersection

The intersection is currently just about capacity (ICU of 97.5%) giving rise to an ICU LOS F. The intersection's overall average delay is 166.9 seconds per vehicle, however vehicles on the single shared lane of the New Tafo [South bound] approach experiences the worst delays of more than 400 seconds per vehicle. Consequently, long queues are common. As a result some motorists try to use alternative routes around the Krofrom signalized intersection.

4.3.1.4 KMA Signalized Intersection

The current traffic situation is such that ICU LOS is F. The ICU is 95.6% indicating less than 5% of reserve capacity. The average overall intersection delay is 135.0 sec/veh. The intersection experiences quite severe congestion during the peak periods. Residual queues at the end of green are common. The slightest traffic fluctuations and incidents as well as sub-optimal signal timings cause severe congestions.

4.3.2 Hypothetical Traffic Scenarios

4.3.2.1 Generating the Hypothetical Traffic Scenarios

The hypothetical traffic scenarios represent the situations of non-existence of the FSS at the signalized intersections. Essentially, they represent the situation in which the accesses which allow the traffic activities at the FSS are absent. That is to say, where applicable there are no vehicular maneuvers and no bus blockages at those accesses to the FSS. Again, the traffic volumes of certain approaches were adjusted to cater for the vehicles which used the FSS area as by-passes to the signalized intersections.

For instance, in the case of Anloga signalized intersection, the existing right-turners from the Airport R/A approach of 144 veh/h was adjusted to 373 veh/h to absorb those 229 veh/h that by-passed the intersection through the TOTAL fuel service station. But there were no vehicle maneuvers and bus blockages at the FSS accesses to impede traffic flows.

The respective summary results of simulation (detail results have been presented as Appendix A-2) of these scenarios are presented below and were further compared with their corresponding existing situations.

4.3.2.2 Anloga Signalized Intersection (FSS is absent)

The simulation results of the hypothetical scenario of the Anloga signalized intersection show that the intersection's ICU LOS is E. Its capacity utilization increased marginally from 86.3% of the existing situation to 87.7%, which is a result of channeling the by-passers (through the FSS) through the signalized intersection. The overall intersection delay for the hypothetical case is 260.2 sec/veh, which is lower than the existing (283.5 sec/veh) by almost 9%.

It is significant to note that by the channeling of the right turn by-passers (through the FSS) on the Airport R/A approach results in a significant jump in the approach delay from 283.5 sec/veh in the existing traffic situation to 656.4 sec/veh in this hypothetical scenario. This is because longer green periods are required to serve the increased volume on this approach. But there were considerable reductions in delays on the other three approaches of the intersection due to the absence of traffic activities on the FSS accesses. Thus the net effect of the overall intersection delay is the reduction of almost 9% of the existing.

4.3.2.3 Aboabo Signalized Intersection (FSS is absent)

The capacity utilization of this intersection is 80.7% with ICU LOS D. The overall intersection delay is 67.8 sec/veh which is a 29.2% reduction of the exiting situation (95.7 sec/veh). Just as in the case of Anloga intersection there is a significant increase of the delay on the Asokore Mampong approach from 115.2 sec/veh (existing situation) to 227.0 sec/veh (hypothetical scenario) due to the channeling of traffic through the intersection rather than by-passing through the FSS. However there is considerable reduction on the Anloga Junction approach from 115.2 sec/veh to 48.5 sec/veh with the absence of the FSS, which ultimately translates into an overall reduction by almost 30% for the intersection.

4.3.2.4 Krofrom Signalized Intersection (FSS is absent)

Channeling the relatively high by-passing traffic on the Suame R/A approach through the Krofrom signalized intersection results in the existing close-to-capacity (97.5%) to an ICU of 105.7% giving rise to an ICU LOS G. The intersection's average delay is 195.1 sec/veh, and this is a result of the considerably longer delays on the Suame

R/A approach; from 259.2 sec/veh (existing situation) to 321.4 sec/veh (hypothetical scenario).

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4.3.2.5 KMA Signalized Intersection (FSS is absent)

The hypothetical scenario gives ICU LOS F. The capacity utilization remains 95.6% just as in the existing situation, since there are no volume changes (there is no channeling of intersection by-passers). The average intersection delay is 118.9 sec/veh which is a reduction of almost 20 sec/veh of the existing traffic situation (136.5 sec/veh). The reduction in overall intersection delay is solely due to the absence of the vehicle maneuvers to the FSS (i.e FSS is absent).

4.4 Impact on Saturation Flow Rates

It has been acknowledged that the presence of accesses within the vicinity of the intersections reduces the saturation flow rates of certain movements as was highlighted (in red) on the intersection layout sketches of Figures 3-2 to 3-5. From Table 4-17, it is seen that the reductions in the saturation flow rates of these movements could be as high as in excess of 50% (example the through traffic on the KNUST approach [WBT] at the Anloga signalized intersection) and as marginal as 2% (example the right turning traffic on the Asokore Mampong approach [WBR] of the Aboabo signalized intersection).

It must also be noted that saturation flow rates of movements which do not experience any contact with traffic activities at the FSS accesses remain unchanged. Examples of such movements, as shown in Table 4-17, are through traffic on the Airport R/A approach [SBT] of the Aboabo signalized intersection; and through traffic on the Kejetia approach [NBT] of the Krofrom signalized intersection.

Table 4-17 **Comparison of Saturation Flow Rates**

Approach	Movement	Existing Saturation Flow Rate *(veh/h/l)*	Hypothetical Saturation Flow Rate *(veh/h/l)*	Percentage Reduction *(%)*
Anloga Signalized Intersection				
KNUST	WBT	1125	1756	56.1
Amakom	EBT	1609	1686	4.8
Anloga Jn	NBL	1202	1810	50.1
Airport R/A	SBR	1122	1580	40.8
Aboabo Signalized Intersection				
Asokore Mampong	WBR	1325	1351	2.0
Aboabo	EBL	1232	1642	33.3
Anloga Jn	NBT	1168	1534	31.3
Airport R/A	SBT	1485	1485	0.0
Krofrom Signalized Intersection				
Airport R/A	WBR	1483	1483	0.0
Suame R/A	EBL	1486	1516	2.0
Kejetia	NBT	1519	1519	0.0
New Town	SBT	1407	1818	29.2
KMA Signalized Intersection				
STC	WBL	1637	1637	0.0
Nhyieso	NBT	1709	1709	0.0
Adum	SBT	1273	1900	49.3

These reductions in saturation flow rates ultimately translate into lower traffic discharges as shown in Figure 4-5, thereby leading to queue formations and avoidable delays. Note that the area under the curve gives the vehicles discharged through the intersection per a given time.

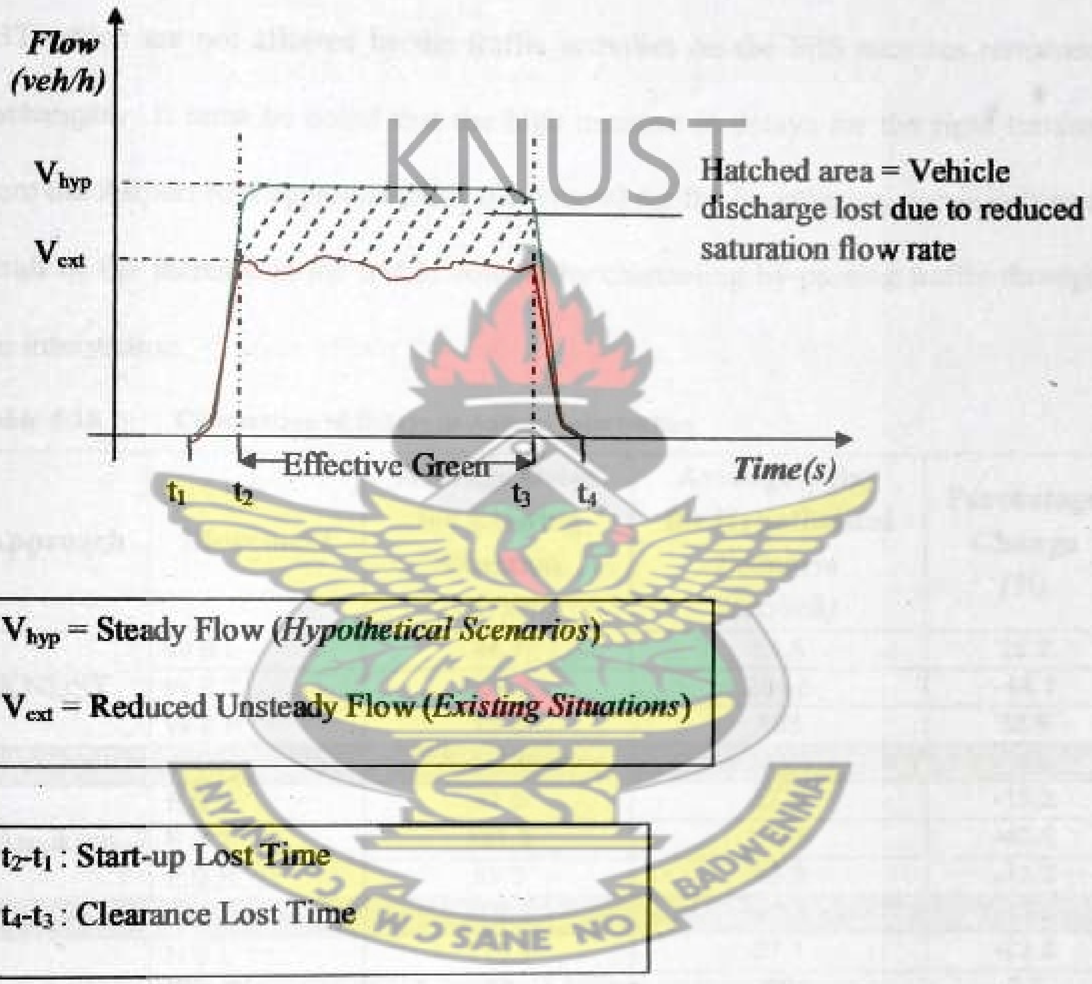

Figure 4-5 Flow-Time Sketch Showing the Effect of Reduced Saturation Flow Rates on Vehicle Discharges

4.5 Comparison of Associated Delays for the Scenarios

The delays associated with the movements for the hypothetical scenarios are generally lower compared with their corresponding delays for the existing situations. As shown in Table 4-18, delays on a few movements such as NBT, NBR, SBL and SBT which are not affected by the traffic activities on the FSS accesses remained unchanged. It must be noted that the high increase in delays for the right turning from the Airport R/A approach [SBR movement] for the hypothetical scenario is as a result of the increase in the traffic volume by channeling by-passing traffic through the intersection.

Table 4-18 Comparison of Delays at Anloga Intersection

Approach	Movement	Average Delay for Existing Situation *(sec/veh)*	Average Delay for Hypothetical Scenario *(sec/veh)*	Percentage Change *(%)*
KNUST	WBL	44.7	51.5	15.2
	WBT	509.3	284.6	-44.1
	WBR	75.8	103	35.9
Approach Delay		**407.5**	**241.4**	**-40.8**
Amakom	EBL	67.8	57.5	-15.2
	EBT	191.9	102.5	-46.6
	EBR	52.5	45.5	-13.3
Approach Delay		**162.6**	**91.7**	**-43.6**
Anloga	NBL	113.5	87.1	-23.3
	NBT	98	98	0.0
	NBR	105.2	105.2	0.0
Approach Delay		**109.5**	**92.6**	**-15.4**
Airport R/A	SBL	263.3	263.3	0.0
	SBT	68	68	0.0
	SBR	520.8	1200.5	130.5
Approach Delay		**283.5**	**656.4**	**131.5**
Intersection's Overall Delay		**285.4**	**260.2**	**-8.8**

At the Aboabo signalized intersection, the Asokore Mampong and Aboabo approaches would experience increases in delays with the absence of the FSS whereas the Anloga Junction approach experience a significant reduction of delays. It should be noted that traffic volume on the Anloga Junction approach is relatively high and thus has a weighting effect. The reduction in delays on this approach (65.9%) thus offsets the increments on the Asokore Mampong and Aboabo approaches and gives a net reduction of almost 30% in the overall intersection delay, as shown in Table 4-19.

Expectedly, delays associated with both scenarios on the Airport R/A approach remain unchanged since the traffic activities on the FSS accesses do not have direct effect.

Table 4-19 **Comparison of Delays at Aboabo Intersection**

Approach	Movement	Average Delay for Existing Situation *(sec/veh)*	Average Delay for Hypothetical Scenario *(sec/veh)*	Percentage Change *(%)*
Asokore Mampong	WB L+R+T	115.2	227	97.0
Approach Delay		115.2	227	97.0
Aboabo	E B L+R+T	53.5	65.7	22.8
Approach Delay		53.5	65.7	22.8
Anloga Jn.	N B L	67	67	0
	N B T+R	153.2	45.7	-70.2
Approach Delay		142.1	48.5	-65.9
Airport R/A	S B L	51	51	0
	S B T+R	28	28	0
Approach Delay		30.4	30.4	0
Intersection's Overall Delay		95.7	67.8	-29.2

In the case of Krofrom Signalized intersection, channeling the high number of by-passers of the intersection on the Suame R/A approach results in 55.6% increase in approach delay over the existing. As shown in Table 4-20, delays on the Airport R/A and Kejetia approaches remained unchanged whereas the New Tafo approach experienced 37.5% reduction. The reduction in average delay on the New Tafo approach is as a result of the absence of traffic activities at the FSS. However, this reduction would not offset the increase in average delays on the Suame R/A approach due to weighting differences in approach volumes. Hence, the resulting net effect is an overall intersection delay increase of 7.9%. Note that the hypothetical scenario of the Krofrom signalized intersection is over the current capacity due to channelization of the by-passers on the Suame R/A approach through the intersection. This reveals the need of right turn filter lanes which the presence of the FSS seems to be offering.

Table 4-20 **Comparison of Delays at Krofrom Intersection**

Approach	Movement	Average Delay for Existing Situations *(sec/veh)*	Average Delay for Hypothetical Scenarios *(sec/veh)*	Percentage Change *(%)*
Airport R/A	W B L+R+T	343.9	343.9	0
Approach Delay		**343.9**	**343.9**	**0**
Suame R/A	E B L+R+T	257.2	400.3	55.6
Approach Delay		**257.2**	**400.3**	**55.6**
Kejetia	N B L+T	162	162	0
	N B R	49.7	49.7	0
Approach Delay		**105.8**	**105.8**	**0**
New Tafo	S B L+R+T	596.7	373.2	-37.5
Approach Delay		**596.7**	**373.2**	**-37.5**
Intersection's Overall Delay		**287.3**	**310**	**7.9**

Table 4-21 show that there is a reduction in overall intersection delay of almost 13% as a result of reductions in most of the movements at the KMA signalized intersection. These reductions are solely due to the absence of the FSS.

Table 4-21 Comparison of Delays at KMA Intersection

Approach	Movement	Average Delay for Existing Situations *(sec/veh)*	Average Delay for Hypothetical Scenarios *(sec/veh)*	Percentage Change *(%)*
S.T.C.	W B L+R	173.5	166.9	-3.8
Approach Delay		**173.5**	166.9	**-3.8**
Nhyieso	N B T+R	160.8	158.6	-1.4
Approach Delay		**160.8**	158.6	**-1.4**
Adum	S B L	48.1	48.1	0.0
	S B T	122.8	48.2	-60.7
Approach Delay		**88.3**	48.1	**-45.5**
Intersection's Overall Delay		**136.5**	118.9	**-12.9**

4.6 Impact of the Location of FSS on the Intersection Performance

The field studies have shown that the location of the FSS in the vicinity of the signalized intersections may facilitate by-passing of the intersections' central areas. This is especially so when the FSS is located at the space between adjacent legs of the intersection with accesses to the FSS on the entry and exist legs. This phenomenon tends to ease traffic through the intersections and consequently reduce congestion at the signalized intersections.

On the other hand, the comparison of the existing situations with their corresponding hypothetical scenarios has established that traffic activities at the FSS (i.e vehicle maneuvers through the FSS and use of accesses to FSS as bus bays causing blockage to traffic flow) potentially reduce the saturation flow rates of certain movements through the signalized intersections. The affected movements depend on the orientation of location of the FSS and their associated accesses on the intersection roads. The reductions in the saturation flow rates translate into lower vehicle discharges through the signalized intersections and consequently contribute to delays. Again, traffic activities at the FSS increases the lost times for some movements as a result prolonged clearance times. This phenomenon also reduces the effective green periods leading to relatively lower vehicle discharges and again contributes to delays at the signalized intersections.

4.7 Improvement of Existing Traffic Situations at the Intersections

4.7.1 Anloga Signalized Intersection

Since there is a reserve capacity of more than 10% (existing ICU = 86.3%) and the intersection lies in a space-constrained area, the proposed improvement is basically with the optimization of the existing signal timings and phase plan. A 4-phase pretimed system of both protected and permitted movements is recommended. The modified phase plan of cycle length 212 seconds is as shown in Figure 4-6. The yellow and all red times are also adjusted to reflect the actual lost times.

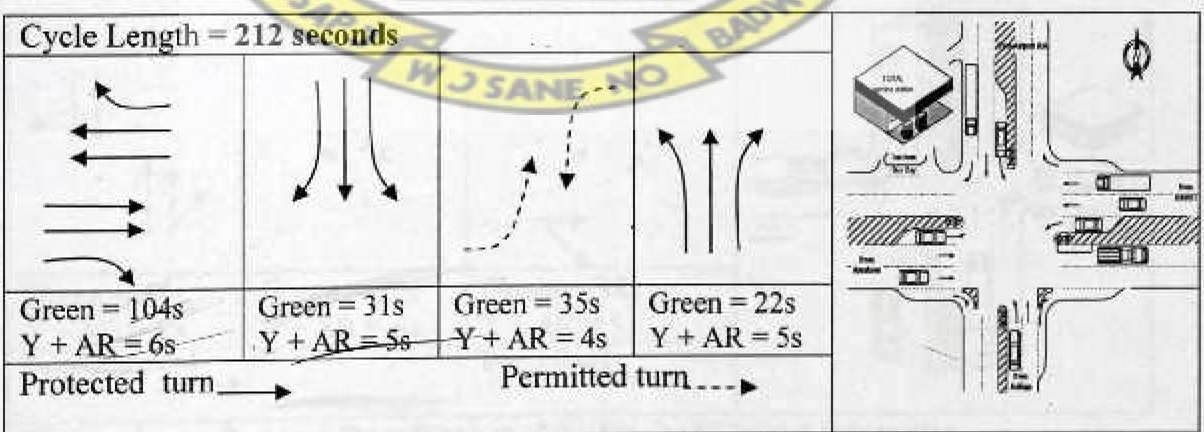

Figure 4-6 Proposed Phase Diagram of the Anloga Signalized Intersection

With this modified signal system, the overall intersection delay is reduced from 285.4 sec/veh to 108.0 sec/veh. It is worthy to note that the ICU LOS, which is based on intersection capacity, of both the existing and optimized signal timings remain in category E. A more drastic physical intervention to increase the capacity is required to yield a far better ICU LOS. This may require grade separation rather than introduction of additional lanes due to space constraints.

4.7.2 Aboabo Signalized Intersection

Due to the relatively high proportions of turning traffic on the Aboabo and Asokore Mampong approaches, additional lanes are proposed to expand the capacities of existing shared lanes. Left turning filter lanes of storage lengths 30m, in accordance with the minimum requirement as contained in the Ghana Highway Authority (GHA) Road Design Guide (1991), are to be introduced on both approaches. In addition to that is a modification of the current 3-phase plan with increased cycle length of 150 seconds with the respective green, and yellow and all red times as displayed in the phase diagram of Figure 4-7.

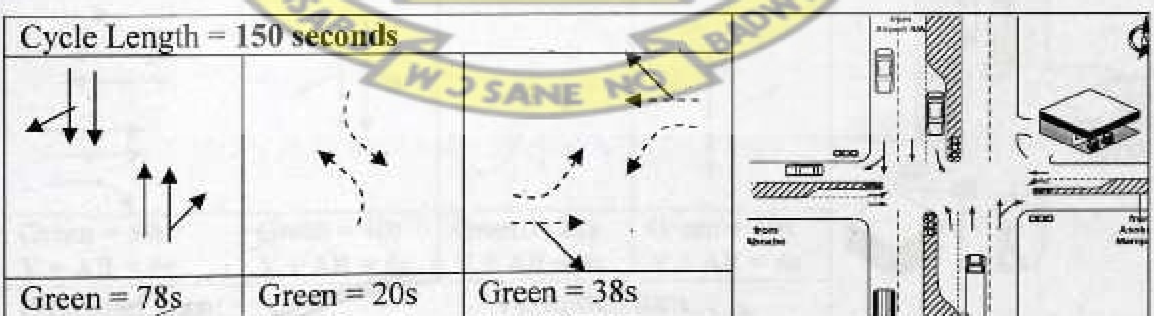

Figure 4-7 Proposed Phase Diagram of the Aboabo Signalized Intersection

The improvements results in a considerable reduction in the overall intersection delay from the existing 95.7 sec/veh of ICU LOS D to 56.8 sec/veh of ICU LOS B. The total lost times being the all red and yellow times is 14.0 seconds and are distributed for the phases as shown in Figure 4-7.

4.7.3 Krofrom Signalized Intersection

The single shared lanes of the Kejetia and New Town approaches are proposed for capacity enhancement by the introduction of one additional lane each. The right turn filter lanes are to be introduced on the main approaches – Airport R/A and Suame R/A. These are to serve the relatively high number of turning vehicles. The current 3-phase plan is thus to be replaced with a 4-phase system of cycle length 150 seconds and the respective green, and yellow and all red times as displayed in the phase diagram of Figure 4-8. The yellow and all red times are apportioned to give a total of 18 seconds in reflection of the actual lost times.

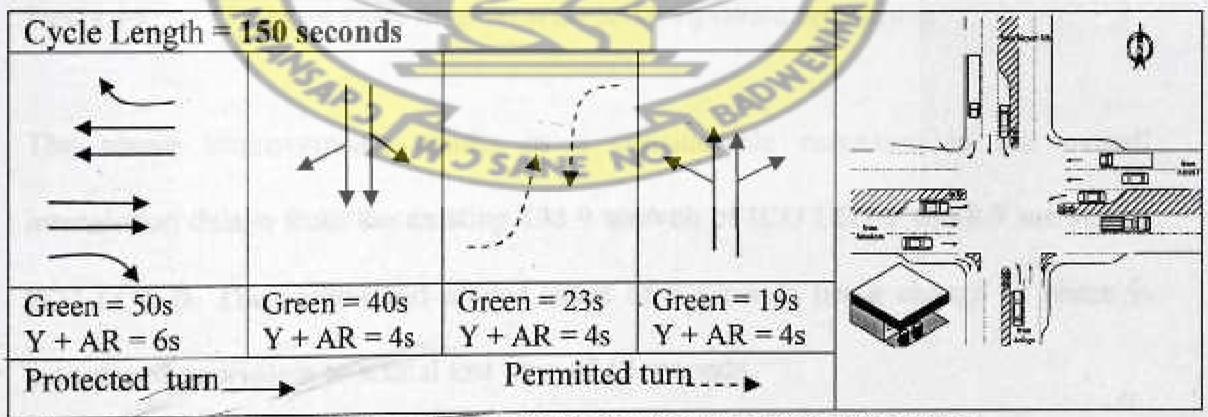

Figure 4-8 Proposed Phase Diagram of the Krofrom Signalized Intersection

This improvement would transform the existing performance status from ICU LOS H to ICU LOS C. Consequently, the existing overall intersection delay of 288.5 sec/veh is more than halved to 110.5 sec/veh. In particular, the very long delay on the New Tafo approach is considerably reduced from 610.8 sec/veh to 175.5 sec/veh.

4.7.4 *KMA Intersection*

At the KMA intersection, a lane is added to the existing shared lane on the STC approach to serve as separate lanes. As shown in Figure 4-9, the existing 3-phase system is slightly modified with a cycle length 145 seconds.

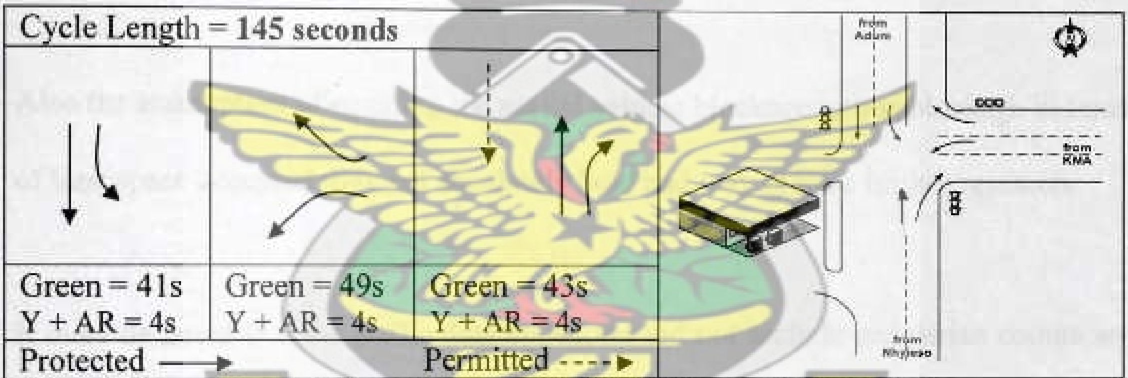

Figure 4-9 Proposed Phase Diagram of the KMA Signalized Intersection

The above improvements results in a considerable reduction in the overall intersection delays from the existing 133.9 sec/veh of ICU LOS F to 48.9 sec/veh of ICU LOS B. The yellow and all red times of 4 seconds per a change of phase is maintained equivalent to a total lost time of 12 seconds.

4.8 Limitation of Study

Ideally the Synchro 6 Trafficware plus SimTraffic package should have been totally calibrated to conform to the local traffic characteristics, and the calibration validated before using for the study. Most of the default calibration sets were used for the simulation. Thus since the assessments were basically carried out from the simulation results from the software, the outcome of the assessments are subject to the accuracy levels of the model results with respect to the local traffic situations. Nonetheless a rough comparison of the field results and the simulation results do not give striking differences.

Also the assumption of equating the partial vehicle blockage to bus blockage in terms of lane space occupied may not absolutely hold and may require further research.

It must be noted that the traffic data collection did not include pedestrian counts and other forms of transport like cycle for their effects in the simulation. Thus the simulation results did not feature the influence of pedestrians and cyclists on the performances of the intersections in this study.

5.0 CONCLUSION

5.1 General Concluding Remarks

The performance of four (4) signalized intersections in the Kumasi Metropolis, namely Anloga, Aboabo, Krofrom, and KMA have been studied. The objectives of the study were to: establish the impact of location of FSS on the observed saturated flow rates of identified lane groups at the signalized intersections; estimate the actual lost times at the intersections; and assess the changes, if any, in delays at the signalized intersections using the simulation results from the *Synchro 6.0 plus SimTraffic* software. The outcome of the study is a useful guide to transportation professionals as well as officials in charge of issuing permits for the establishment of FSS and other similar facilities. The key research findings are highlighted as follows:

5.2 By-passing of Intersections Through the FSS

The location of the FSS in the vicinity of the signalized intersections may facilitate by-passing of the intersections' central areas depending on the orientation of the FSS to the intersection. This phenomenon tends to ease traffic through the intersections and consequently reduce congestion at the signalized intersections.

5.3 Reduction of Saturation Rates of Certain Movements

Vehicle maneuvers through the FSS and use of accesses to FSS as bus bays reduce the saturation flow rates of certain movements at the signalized intersections. The affected movements depend on the orientation of location of the FSS and their associated accesses on the intersection roads. The range of percentage of reduction in the saturation flow rates is wide; as marginal as 2% through to the excess of 50% depending on the orientation of the FSS accesses to the main intersection roadways. These reductions translate into lower vehicle discharges through the signalized intersections and consequently contribute to delays.

5.4 Increased Lost Times (Clearance Times)

The traffic activities at the FSS cause increases in lost times for some movements as a result of prolonged clearance times. The actual lost times for certain movements could be as high as 7 seconds as against the default value of 4 seconds. This phenomenon also reduces the effective green periods of succeeding movements leading to relatively lower discharges and delays at the signalized intersections.

5.5 Impact on Delays

The presence of FSS in the vicinity of the signalized intersections generally causes increases in the overall intersection delays. This percentage of increase varies in the range of 8% to about 30% depending on the traffic levels among other factors.

5.6 Recommendations and Further Research

Generally, the current signal phasing and timings of the respective control systems at the intersections should be reviewed and optimized. This is especially so with the current use of the default all-red and yellow (AR+Y) times of 4 sec which are unable to accommodate the existing lost times for certain movements because of the traffic activities at the FSS. In addition, it is proposed that the geometric characteristics of the Aboabo, Krofrom, and KMA intersections be improved by the addition of filter lanes to accommodate the relatively large turning movements. Warning signs prohibiting the use of accesses to the FSS as bus bays should be installed and proper vehicle stopping areas- bus bays and lay-bys provided at the intersections.

Road users, especially drivers of commercial vehicles, should be educated on the impact of their actions of using FSS accesses areas as bus bays. It would be imperative for the random and periodic dispatch of police from the MTTU to ensure traffic order and adherence of these directives at the signalized intersections.

The permit issuing authority should ensure that the ensuing Traffic Impact Statements (TIS) contain measures put forward to minimize the negative impacts of accesses on the signalized intersection legs due to the presence of the existing FSS. Regarding the issuance of permits for the establishments of FSS at signalized intersections, as part of the Traffic Impact Assessments (TIA), considerations should be given to the vehicle traffic composition and distribution. This is especially important where the signalized intersection experiences relatively high commercial vehicle traffic.

It is recommended that further research should be carried out to assess the impacts of the FSS located at signalized intersections with regard to other performance indicators. For the purposes of appraisal of intervention schemes, economic impacts of the location of FSS at signalized intersections should also be considered.

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APPENDICES

APPENDIX A-1 Calibrated Simulation Results for Existing Traffic Situations at the Signalized Intersections

APPENDIX A-2 Simulation Results for Hypothetical Traffic Scenarios of Non-Existence of the Fuel Service Stations at the Signalized Intersections

APPENDIX B Simulation Results for Improvements of the Existing Traffic Situations at the Signalized Intersections

APPENDIX A-1 Calibrated Simulation Results for Existing Traffic Situations at the Signalized Intersections

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↑	↱	↰	↑↑	↱	↰	↑	↱	↰	↰↑	↱
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.4	3.3	4.7	3.0	3.4	3.0	4.8	4.8	4.7	3.0	3.8	3.8
Grade (%)		-3%			-5%			0%			-1%	
Total Lost time (s)	4.0	4.0	4.0	4.0	6.0	4.0	6.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	1.00	1.00	1.00	0.95	0.95	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.96	1.00
Satd. Flow (prot)	**1659**	**3218**	**1672**	**1528**	**2250**	**1486**	**1202**	**1696**	**1648**	**1462**	**1563**	**1122**
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.74	1.00
Satd. Flow (perm)	1659	3218	1672	1528	2250	1486	1202	1696	1648	1462	1200	1122
Volume (vph)	291	1211	52	27	1483	397	153	32	51	421	22	144
Peak-hour factor, PHF	0.91	0.97	0.92	0.89	0.96	0.90	0.92	0.88	0.91	0.90	0.94	0.92
Growth Factor (vph)	95%	100%	95%	95%	100%	95%	95%	100%	95%	95%	100%	95%
Adj. Flow (vph)	304	1248	54	29	1545	419	158	36	53	444	23	149
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	304	1248	54	29	1545	419	158	36	53	333	134	149
Heavy Vehicles (%)	8%	5%	10%	13%	3%	4%	13%	27%	10%	10%	33%	5%
Bus Blockages (#/hr)	0	23	0	0	88	0	33	0	0	0	0	21
Parking (#/hr)					69		27					25
Turn Type	Split		Prot	Split		Prot	Prot		Prot	Prot		Prot
Protected Phases	2	2	2	1	1	1	4	3	3	4	3	3
Permitted Phases												
Actuated Green, G (s)	66.0	66.0	66.0	76.0	76.0	76.0	38.0	16.0	16.0	38.0	54.0	16.0
Effective Green, g (s)	66.0	66.0	66.0	76.0	74.0	76.0	36.0	16.0	16.0	38.0	54.0	16.0
Actuated g/C Ratio	0.31	0.31	0.31	0.36	0.35	0.36	0.17	0.08	0.08	0.18	0.25	0.08
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Grp Cap (vph)	516	1002	521	548	785	533	204	128	124	262	371	85
v/s Ratio Prot	0.18	c0.39	0.03	0.02	c0.69	0.28	0.13	0.02	0.03	c0.23	0.06	c0.13
v/s Ratio Perm											0.03	
v/c Ratio	0.59	1.25	0.10	0.05	1.97	0.79	0.77	0.28	0.43	1.27	0.36	1.75
Uniform Delay, d1	61.6	73.0	51.9	44.5	69.0	60.7	84.1	92.6	93.6	87.0	64.8	98.0
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	4.9	118.9	0.4	0.2	440.3	11.1	24.4	5.4	10.4	148.4	2.7	382.5
Delay (s)	66.4	191.9	52.3	44.6	509.3	71.9	108.5	98.0	104.0	235.4	67.6	480.5
Level of Service	E	F	D	D	F	E	F	F	F	F	E	F
Approach Delay (s)		163.5			410.6			106.0			258.2	
Approach LOS		F			F			F			F	

Intersection Summary

HCM Average Control Delay	283.7	HCM Level of Service	F
HCM Volume to Capacity ratio	1.57		
Actuated Cycle Length (s)	212.0	Sum of lost time (s)	18.0
Intersection Capacity Utilization	86.3%	ICU Level of Service	E
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		✣			✣		↰	↑↱		↰	↑↱	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.7	3.8	3.7	3.7	3.7	3.7	2.5	3.0	3.7	2.5	2.9	3.7
Grade (%)		2%			1%			0%			5%	
Total Lost time (s)		4.0			6.0		4.0	4.0		4.0	4.0	
Lane Util. Factor		1.00			1.00		1.00	0.95		1.00	0.95	
Frt		0.96			0.94		1.00	0.99		1.00	0.99	
Flt Protected		0.99			0.99		0.95	1.00		0.95	1.00	
Satd. Flow (prot)		1642			1622		1495	2335		1367	2970	
Flt Permitted		0.74			0.81		0.95	1.00		0.95	1.00	
Satd. Flow (perm)		1232			1325		1495	2335		1367	2970	
Volume (vph)	51	99	55	62	104	149	187	1211	59	103	838	71
Peak-hour factor, PHF	0.90	0.90	0.90	0.92	0.92	0.92	0.92	0.93	0.93	0.92	0.95	0.95
Adj. Flow (vph)	57	110	61	67	113	162	203	1302	63	112	882	75
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	0	228	0	0	342	0	203	1365	0	112	957	0
Heavy Vehicles (%)	9%	13%	11%	8%	6%	12%	6%	9%	10%	13%	8%	8%
Bus Blockages (#/hr)	8	0	0	0	0	23	0	43	0	0	0	0
Parking (#/hr)	11					15		47				
Turn Type	Perm			Perm			Prot			Prot		
Protected Phases		3			3		2	1		2	1	
Permitted Phases	3			3								
Actuated Green, G (s)		31.0			31.0		21.0	56.0		21.0	56.0	
Effective Green, g (s)		31.0			29.0		21.0	56.0		21.0	56.0	
Actuated g/C Ratio		0.26			0.24		0.18	0.47		0.18	0.47	
Clearance Time (s)		4.0			4.0		4.0	4.0		4.0	4.0	
Lane Grp Cap (vph)		318			320		262	1090		239	1386	
v/s Ratio Prot							c0.14	c0.58		0.08	0.32	
v/s Ratio Perm		0.19			c0.26							
v/c Ratio		0.72			1.07		0.77	1.25		0.47	0.69	
Uniform Delay, d1		40.5			45.5		47.2	32.0		44.5	25.2	
Progression Factor		1.00			1.00		1.00	1.00		1.00	1.00	
Incremental Delay, d2		13.0			69.7		19.8	121.2		6.5	2.8	
Delay (s)		53.5			115.2		67.0	153.2		51.0	28.0	
Level of Service		D			F		E	F		D	C	
Approach Delay (s)		53.5			115.2			142.1			30.4	
Approach LOS		D			F			F			C	

Intersection Summary

HCM Average Control Delay	95.7	HCM Level of Service	F
HCM Volume to Capacity ratio	1.11		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	14.0
Intersection Capacity Utilization	75.1%	ICU Level of Service	D
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱		✣	↱		✣	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.5	3.7	3.5	3.5	3.8	3.5	3.7	3.5	2.8	3.7	4.8	3.7
Grade (%)		0%			0%			2%			1%	
Total Lost time (s)	4.0	5.0	4.0	4.0	4.0	5.0		4.0	4.0		5.0	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00		0.95	0.95		1.00	
Frt	1.00	1.00	0.85	1.00	1.00	0.85		1.00	0.85		0.98	
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.98	1.00		0.98	
Satd. Flow (prot)	1580	1731	887	982	1750	1331		1614	1331		1410	
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00		0.72	1.00		0.36	
Satd. Flow (perm)	1580	1731	887	982	1750	1331		1180	1331		513	
Volume (vph)	111	584	97	202	684	467	134	249	304	98	144	41
Peak-hour factor, PHF	0.90	0.93	0.88	0.91	0.94	0.92	0.91	0.91	0.91	0.92	0.92	0.92
Growth Factor (vph)	95%	100%	95%	95%	100%	95%	95%	100%	95%	95%	100%	95%
Adj. Flow (vph)	117	628	105	211	728	482	140	274	317	101	157	42
RTOR Reduction (vph)	0	0	49	0	0	194	0	0	82	0	0	0
Lane Group Flow (vph)	117	628	56	211	728	288	0	414	235	0	300	0
Heavy Vehicles (%)	13%	11%	10%	1%	11%	20%	9%	7%	4%	22%	10%	6%
Bus Blockages (#/hr)	0	0	27	38	0	0	0	0	0	0	14	0
Parking (#/hr)			43	49							16	
Turn Type	Split		Perm	Split		Perm	Perm		Perm	Perm		
Protected Phases	1	1		2	2			3			3	
Permitted Phases			1			2	3		3	3		
Actuated Green, G (s)	31.0	31.0	31.0	46.0	46.0	46.0		46.0	46.0		46.0	
Effective Green, g (s)	31.0	30.0	31.0	46.0	46.0	45.0		46.0	46.0		45.0	
Actuated g/C Ratio	0.23	0.22	0.23	0.34	0.34	0.33		0.34	0.34		0.33	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0	4.0		4.0	
Lane Grp Cap (vph)	363	385	204	335	596	444		402	454		171	
v/s Ratio Prot	0.07	c0.36		0.21	c0.42							
v/s Ratio Perm			0.06			0.22		0.36	0.18		c0.58	
v/c Ratio	0.32	1.63	0.27	0.63	1.22	0.65		1.03	0.52		1.75	
Uniform Delay, d1	43.3	52.5	42.7	37.4	44.5	38.3		44.5	35.6		45.0	
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00		1.00	
Incremental Delay, d2	2.3	295.6	3.3	8.7	114.2	7.2		52.8	4.2		362.4	
Delay (s)	45.6	348.1	46.0	46.0	158.7	45.4		97.3	39.8		407.4	
Level of Service	D	F	D	D	F	D		F	D		F	
Approach Delay (s)		269.2			103.6			72.3			407.4	
Approach LOS		F			F			E			F	

Intersection Summary

HCM Average Control Delay	166.9	HCM Level of Service	F
HCM Volume to Capacity ratio	1.52		
Actuated Cycle Length (s)	135.0	Sum of lost time (s)	14.0
Intersection Capacity Utilization	97.5%	ICU Level of Service	F
Analysis Period (min)	15		

c Critical Lane Group

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Lane Width	3.4	3.7	3.6	3.7	3.5	3.6
Grade (%)	0%		3%			-2%
Total Lost time (s)	4.0		4.0		4.0	4.0
Lane Util. Factor	1.00		1.00		1.00	1.00
Frt	0.92		0.94		1.00	1.00
Flt Protected	0.98		1.00		0.95	1.00
Satd. Flow (prot)	1637		1709		1767	1273
Flt Permitted	0.98		1.00		0.95	1.00
Satd. Flow (perm)	1637		1709		1767	1273
Volume (vph)	223	347	308	279	337	388
Peak-hour factor, PHF	0.92	0.92	0.93	0.93	0.92	0.91
Adj. Flow (vph)	242	377	331	300	366	426
RTOR Reduction (vph)	0	0	0	0	0	0
Lane Group Flow (vph)	619	0	631	0	366	426
Heavy Vehicles (%)	4%	1%	2%	3%	2%	1%
Parking (#/hr)						46
Turn Type					Split	
Protected Phases	1		2		3	3
Permitted Phases						
Actuated Green, G (s)	41.0		41.0		41.0	41.0
Effective Green, g (s)	41.0		41.0		41.0	41.0
Actuated g/C Ratio	0.30		0.30		0.30	0.30
Clearance Time (s)	4.0		4.0		4.0	4.0
Lane Grp Cap (vph)	497		519		537	387
v/s Ratio Prot	c0.38		c0.37		0.21	c0.33
v/s Ratio Perm						
v/c Ratio	1.25		1.22		0.68	1.10
Uniform Delay, d1	47.0		47.0		41.3	47.0
Progression Factor	1.00		1.00		1.00	1.00
Incremental Delay, d2	126.5		113.8		6.8	75.8
Delay (s)	173.5		160.8		48.1	122.8
Level of Service	F		F		D	F
Approach Delay (s)	173.5		160.8			88.3
Approach LOS	F		F			F

Intersection Summary			
HCM Average Control Delay	136.5	HCM Level of Service	F
HCM Volume to Capacity ratio	1.19		
Actuated Cycle Length (s)	135.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	95.6%	ICU Level of Service	F
Analysis Period (min)	15		

c Critical Lane Group

APPENDIX A-2 Simulation Results for Hypothetical Traffic Scenarios of Non-Existence of the Fuel Service Stations at the Signalized Intersections

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↑	↱	↰	↑↑	↱	↰	↑	↱	↰	↰↑	↱
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.4	3.3	4.7	3.0	3.4	3.0	4.8	4.8	4.7	3.0	3.8	3.8
Grade (%)		-3%			-5%			0%			-1%	
Total Lost time (s)	4.0	4.0	4.0	4.0	6.0	4.0	6.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	1.00	1.00	1.00	0.95	0.95	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.96	1.00
Satd. Flow (prot)	1659	3373	1672	1528	3513	1486	1810	1696	1648	1462	1564	1580
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.74	1.00
Satd. Flow (perm)	1659	3373	1672	1528	3513	1486	1810	1696	1648	1462	1199	1580
Volume (vph)	291	1211	52	27	1483	397	153	32	51	421	22	373
Peak-hour factor, PHF	0.91	0.97	0.92	0.89	0.96	0.90	0.92	0.88	0.91	0.90	0.94	0.92
Adj. Flow (vph)	320	1248	57	30	1545	441	166	36	56	468	23	405
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	320	1248	57	30	1545	441	166	36	56	351	140	405
Heavy Vehicles (%)	8%	5%	10%	13%	3%	4%	13%	27%	10%	10%	33%	5%
Turn Type	Split		Prot	Split		Prot	Prot		Prot	Prot		Prot
Protected Phases	2	2	2	1	1	1	4	3	3	4	3	3
Permitted Phases												
Actuated Green, G (s)	76.0	76.0	76.0	66.0	66.0	66.0	38.0	16.0	16.0	38.0	54.0	16.0
Effective Green, g (s)	76.0	76.0	76.0	66.0	64.0	66.0	36.0	16.0	16.0	38.0	54.0	16.0
Actuated g/C Ratio	0.36	0.36	0.36	0.31	0.30	0.31	0.17	0.08	0.08	0.18	0.25	0.08
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Grp Cap (vph)	595	1209	599	476	1061	463	307	128	124	262	371	119
v/s Ratio Prot	0.19	c0.37	0.03	0.02	c0.44	0.30	0.09	0.02	0.03	c0.24	0.07	c0.26
v/s Ratio Perm											0.03	
v/c Ratio	0.54	1.03	0.10	0.06	1.46	0.95	0.54	0.28	0.45	1.34	0.38	3.40
Uniform Delay, d1	54.0	68.0	45.2	51.3	74.0	71.5	80.4	92.6	93.8	87.0	65.1	98.0
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	3.5	34.5	0.3	0.3	210.6	31.5	6.7	5.4	11.4	176.3	2.9	1102.5
Delay (s)	57.5	102.5	45.5	51.5	284.6	103.0	87.1	98.0	105.2	263.3	68.0	1200.5
Level of Service	E	F	D	D	F	F	F	F	F	F	E	F
Approach Delay (s)		91.7			241.4			92.6			656.4	
Approach LOS		F			F			F			F	

Intersection Summary

HCM Average Control Delay	260.2	HCM Level of Service	F
HCM Volume to Capacity ratio	1.43		
Actuated Cycle Length (s)	212.0	Sum of lost time (s)	18.0
Intersection Capacity Utilization	87.7%	ICU Level of Service	E
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		✥			✥		↰	↑↑↱		↰	↑↑↱	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.7	3.8	3.7	3.7	3.7	3.7	2.5	3.0	3.7	2.5	2.9	3.7
Grade (%)		2%			1%			0%			5%	
Total Lost time (s)		4.0			6.0		4.0	4.0		4.0	4.0	
Lane Util. Factor		1.00			1.00		1.00	0.95		1.00	0.95	
Frt		0.96			0.92		1.00	0.95		1.00	0.95	
Flt Protected		0.99			0.99		0.95	1.00		0.95	1.00	
Satd. Flow (prot)		1642			1588		1495	3068		1367	2970	
Flt Permitted		0.64			0.84		0.95	1.00		0.95	1.00	
Satd. Flow (perm)		1067			1351		1495	3068		1367	2970	
Volume (vph)	51	99	55	62	104	243	187	1211	59	103	838	71
Peak-hour factor, PHF	0.90	0.90	0.90	0.92	0.92	0.92	0.92	0.93	0.93	0.92	0.95	0.95
Adj. Flow (vph)	57	110	61	67	113	264	203	1302	63	112	882	75
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	0	228	0	0	444	0	203	1365	0	112	957	0
Heavy Vehicles (%)	9%	13%	11%	8%	6%	12%	6%	9%	10%	13%	8%	8%
Turn Type	Perm			Perm			Prot			Prot		
Protected Phases		3			3		2	1		2	1	
Permitted Phases	3			3								
Actuated Green, G (s)		31.0			31.0		21.0	56.0		21.0	56.0	
Effective Green, g (s)		31.0			29.0		21.0	56.0		21.0	56.0	
Actuated g/C Ratio		0.26			0.24		0.18	0.47		0.18	0.47	
Clearance Time (s)		4.0			4.0		4.0	4.0		4.0	4.0	
Lane Grp Cap (vph)		276			326		262	1432		239	1386	
v/s Ratio Prot							c0.14	c0.44		0.08	0.32	
v/s Ratio Perm		0.21			c0.33							
v/c Ratio		0.83			1.36		0.77	0.95		0.47	0.69	
Uniform Delay, d1		42.0			45.5		47.2	30.7		44.5	25.2	
Progression Factor		1.00			1.00		1.00	1.00		1.00	1.00	
Incremental Delay, d2		23.8			181.5		19.8	15.0		6.5	2.8	
Delay (s)		65.7			227.0		67.0	45.7		51.0	28.0	
Level of Service		E			F		E	D		D	C	
Approach Delay (s)		65.7			227.0			48.5			30.4	
Approach LOS		E			F			D			C	

Intersection Summary			
HCM Average Control Delay	67.8	HCM Level of Service	E
HCM Volume to Capacity ratio	1.03		
Actuated Cycle Length (s)	120.0	Sum of lost time (s)	14.0
Intersection Capacity Utilization	80.7%	ICU Level of Service	D
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱		✣	↱		✣	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.5	3.7	3.5	3.5	3.8	3.5	3.7	3.5	2.8	3.7	4.8	3.7
Grade (%)		0%			0%			2%			1%	
Total Lost time (s)	4.0	5.0	4.0	4.0	4.0	5.0		4.0	4.0		5.0	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00		0.95	0.95		1.00	
Frt	1.00	1.00	0.85	1.00	1.00	0.85		1.00	0.85		0.98	
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.98	1.00		0.98	
Satd. Flow (prot)	1580	1731	1452	1767	1750	1331		1610	1331		1822	
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00		0.70	1.00		0.32	
Satd. Flow (perm)	1580	1731	1452	1767	1750	1331		1149	1331		599	
Volume (vph)	111	684	267	202	784	567	154	249	404	98	144	41
Peak-hour factor, PHF	0.90	0.93	0.88	0.91	0.94	0.92	0.91	0.91	0.91	0.92	0.92	0.92
Growth Factor (vph)	95%	100%	95%	95%	100%	95%	95%	100%	95%	95%	100%	95%
Adj. Flow (vph)	117	735	288	211	834	585	161	274	422	101	157	42
RTOR Reduction (vph)	0	0	115	0	0	206	0	0	104	0	0	0
Lane Group Flow (vph)	117	735	173	211	834	379	0	435	318	0	300	0
Heavy Vehicles (%)	13%	11%	10%	1%	11%	20%	9%	7%	4%	22%	10%	6%
Turn Type	Split		Perm	Split		Perm	Perm		Perm	Perm		
Protected Phases	1	1		2	2			3			3	
Permitted Phases			1			2	3		3	3		
Actuated Green, G (s)	31.0	31.0	31.0	46.0	46.0	46.0		46.0	46.0		46.0	
Effective Green, g (s)	31.0	30.0	31.0	46.0	46.0	45.0		46.0	46.0		45.0	
Actuated g/C Ratio	0.23	0.22	0.23	0.34	0.34	0.33		0.34	0.34		0.33	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0	4.0		4.0	
Lane Grp Cap (vph)	363	385	333	602	596	444		392	454		200	
v/s Ratio Prot	0.07	c0.42		0.12	c0.48							
v/s Ratio Perm			0.12			0.28		0.38	0.24		c0.50	
v/c Ratio	0.32	1.91	0.52	0.35	1.40	0.85		1.11	0.70		1.50	
Uniform Delay, d1	43.3	52.5	45.5	33.3	44.5	41.9		44.5	38.5		45.0	
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00		1.00	
Incremental Delay, d2	2.3	418.7	5.7	1.6	189.7	18.5		78.6	8.7		249.4	
Delay (s)	45.6	471.2	51.2	34.9	234.2	60.4		123.1	47.2		294.4	
Level of Service	D	F	D	C	F	E		F	D		F	
Approach Delay (s)		321.4			146.0			85.7			294.4	
Approach LOS		F			F			F			F	

Intersection Summary

HCM Average Control Delay	195.1	HCM Level of Service	F
HCM Volume to Capacity ratio	1.56		
Actuated Cycle Length (s)	135.0	Sum of lost time (s)	14.0
Intersection Capacity Utilization	105.7%	ICU Level of Service	G
Analysis Period (min)	15		

c Critical Lane Group

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Lane Width	3.4	3.7	3.6	3.7	3.5	3.6
Grade (%)	0%		3%			-2%
Total Lost time (s)	4.0		4.0		4.0	4.0
Lane Util. Factor	1.00		1.00		1.00	1.00
Frt	0.92		0.93		1.00	1.00
Flt Protected	0.98		1.00		0.95	1.00
Satd. Flow (prot)	1637		1707		1767	1900
Flt Permitted	0.98		1.00		0.95	1.00
Satd. Flow (perm)	1637		1707		1767	1900
Volume (vph)	223	347	308	279	337	388
Peak-hour factor, PHF	0.92	0.94	0.95	0.92	0.92	0.97
Adj. Flow (vph)	242	369	324	303	366	400
RTOR Reduction (vph)	0	0	0	0	0	0
Lane Group Flow (vph)	611	0	627	0	366	400
Heavy Vehicles (%)	4%	1%	2%	3%	2%	1%
Turn Type					Split	
Protected Phases	1		2		3	3
Permitted Phases						
Actuated Green, G (s)	41.0		41.0		41.0	41.0
Effective Green, g (s)	41.0		41.0		41.0	41.0
Actuated g/C Ratio	0.30		0.30		0.30	0.30
Clearance Time (s)	4.0		4.0		4.0	4.0
Lane Grp Cap (vph)	497		518		537	577
v/s Ratio Prot	c0.37		c0.37		0.21	c0.21
v/s Ratio Perm						
v/c Ratio	1.23		1.21		0.68	0.69
Uniform Delay, d1	47.0		47.0		41.3	41.5
Progression Factor	1.00		1.00		1.00	1.00
Incremental Delay, d2	119.9		111.6		6.8	6.7
Delay (s)	166.9		158.6		48.1	48.2
Level of Service	F		F		D	D
Approach Delay (s)	166.9		158.6			48.1
Approach LOS	F		F			D

Intersection Summary			
HCM Average Control Delay	118.9	HCM Level of Service	F
HCM Volume to Capacity ratio	1.04		
Actuated Cycle Length (s)	135.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	95.6%	ICU Level of Service	F
Analysis Period (min)	15		

c Critical Lane Group

APPENDIX B Simulation Results for Improvements of the Existing Traffic Situations at the Signalized Intersections

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↑	↱	↰	↑↑	↱	↰	↑	↱	↰	↰↑	↱
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.4	3.3	4.7	3.0	3.4	3.0	4.8	4.8	4.7	3.0	3.8	3.8
Grade (%)		-3%			-5%			0%			-1%	
Total Lost time (s)	4.0	4.0	4.0	4.0	6.0	4.0	6.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	1.00	1.00	1.00	0.95	0.95	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.96	1.00
Satd. Flow (prot)	1659	3373	1672	1528	2731	1486	1385	1696	1648	1462	1581	1225
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.96	1.00
Satd. Flow (perm)	1659	3373	1672	1528	2731	1486	1385	1696	1648	1462	1581	1225
Volume (vph)	291	1211	152	27	1483	397	153	82	51	421	22	144
Peak-hour factor, PHF	0.91	0.97	0.92	0.89	0.96	0.90	0.92	0.88	0.91	0.90	0.94	0.92
Adj. Flow (vph)	320	1248	165	30	1545	441	166	93	56	468	23	157
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	320	1248	165	30	1545	441	166	93	56	251	240	157
Heavy Vehicles (%)	8%	5%	10%	13%	3%	4%	13%	27%	10%	10%	33%	5%
Parking (#/hr)					69		27					25
Turn Type	Prot		Prot	Prot		Prot	Split		Prot	Split		Prot
Protected Phases	3	1	1	3	1	1	4	4	4	2	2	2
Permitted Phases												
Actuated Green, G (s)	35.0	106.0	106.0	35.0	106.0	106.0	24.0	24.0	24.0	31.0	31.0	31.0
Effective Green, g (s)	35.0	106.0	106.0	35.0	104.0	106.0	22.0	24.0	24.0	31.0	31.0	31.0
Actuated g/C Ratio	0.17	0.50	0.50	0.17	0.49	0.50	0.10	0.11	0.11	0.15	0.15	0.15
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Grp Cap (vph)	274	1687	836	252	1340	743	144	192	187	214	231	179
v/s Ratio Prot	c0.19	0.37	0.10	0.02	c0.57	0.30	c0.12	0.05	0.03	c0.17	0.15	0.13
v/s Ratio Perm												
v/c Ratio	1.17	0.74	0.20	0.12	1.15	0.59	1.15	0.48	0.30	1.17	1.04	0.88
Uniform Delay, d1	88.5	42.1	29.4	75.4	54.0	37.7	95.0	88.2	86.3	90.5	90.5	88.6
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	107.6	3.0	0.5	1.0	77.8	3.5	121.9	8.5	4.1	116.1	69.8	41.1
Delay (s)	196.1	45.0	29.9	76.3	131.8	41.2	216.9	96.7	90.3	206.6	160.3	129.8
Level of Service	F	D	C	E	F	D	F	F	F	F	F	F
Approach Delay (s)		71.5			111.1			158.9			170.8	
Approach LOS		E			F			F			F	

Intersection Summary

HCM Average Control Delay	108.0	HCM Level of Service	F
HCM Volume to Capacity ratio	1.16		
Actuated Cycle Length (s)	212.0	Sum of lost time (s)	20.0
Intersection Capacity Utilization	87.7%	ICU Level of Service	E
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↑↱		↰	↑↱	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.7	3.8	3.7	3.7	3.7	3.7	2.5	3.0	3.7	2.5	2.9	3.7
Grade (%)		2%			1%			0%			5%	
Total Lost time (s)	4.0	4.0	4.0	6.0	6.0	6.0	4.0	4.0		4.0	4.0	
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.95		1.00	0.95	
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	0.99		1.00	0.99	
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00		0.95	1.00	
Satd. Flow (prot)	**1401**	**1702**	**1456**	**1681**	**1803**	**1197**	**1495**	**2554**		**1367**	**2970**	
Flt Permitted	0.66	1.00	1.00	0.63	1.00	1.00	0.95	1.00		0.95	1.00	
Satd. Flow (perm)	971	1702	1456	1113	1803	1197	1495	2554		1367	2970	
Volume (vph)	51	99	55	62	104	149	187	1211	59	103	838	71
Peak-hour factor, PHF	0.90	0.90	0.90	0.92	0.92	0.92	0.92	0.93	0.93	0.92	0.95	0.95
Adj. Flow (vph)	57	110	61	67	113	162	203	1302	63	112	882	75
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	57	110	61	67	113	162	203	1365	0	112	957	0
Heavy Vehicles (%)	9%	13%	11%	8%	6%	12%	6%	9%	10%	13%	8%	8%
Parking (#/hr)	11					15		47				
Turn Type	Perm		Perm	Perm		Perm	Prot			Prot		
Protected Phases		3			3		2	1		2	1	
Permitted Phases	3		3	3		3						
Actuated Green, G (s)	40.0	40.0	40.0	40.0	40.0	40.0	20.0	78.0		20.0	78.0	
Effective Green, g (s)	40.0	40.0	40.0	38.0	38.0	38.0	20.0	78.0		20.0	78.0	
Actuated g/C Ratio	0.27	0.27	0.27	0.25	0.25	0.25	0.13	0.52		0.13	0.52	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0		4.0	4.0	
Lane Grp Cap (vph)	259	454	388	282	457	303	199	1328		182	1544	
v/s Ratio Prot		0.06			0.06		c0.14	c0.53		0.08	0.32	
v/s Ratio Perm	0.06		0.04	0.06		c0.14						
v/c Ratio	0.22	0.24	0.16	0.24	0.25	0.53	1.02	1.03		0.62	0.62	
Uniform Delay, d1	42.8	43.1	42.1	44.5	44.6	48.4	65.0	36.0		61.4	25.5	
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00	
Incremental Delay, d2	2.0	1.3	0.9	2.0	1.3	6.6	69.1	32.1		14.6	1.9	
Delay (s)	44.8	44.4	43.0	46.5	45.9	55.0	134.1	68.1		76.0	27.4	
Level of Service	D	D	D	D	D	D	F	E		E	C	
Approach Delay (s)		44.1			50.3			76.6			32.5	
Approach LOS		D			D			E			C	

Intersection Summary

HCM Average Control Delay	56.8	HCM Level of Service	E
HCM Volume to Capacity ratio	0.89		
Actuated Cycle Length (s)	150.0	Sum of lost time (s)	14.0
Intersection Capacity Utilization	62.8%	ICU Level of Service	B
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↑	↱	↰	↑↑	↱		↰↑	↱		↰↑↱	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Lane Width	3.7	3.2	3.7	3.7	3.5	3.7	3.7	3.0	2.8	3.7	4.8	3.7
Grade (%)		0%			0%			2%			4%	
Total Lost time (s)	4.0	5.0	4.0	4.0	4.0	5.0		4.0	4.0		5.0	
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00		0.95	1.00		0.95	
Frt	1.00	1.00	0.85	1.00	1.00	0.85		1.00	0.85		0.98	
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00		0.98	1.00		0.98	
Satd. Flow (prot)	1615	3108	1017	1184	3216	1361		3037	1401		3089	
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00		0.58	1.00		0.70	
Satd. Flow (perm)	1615	3108	1017	1184	3216	1361		1794	1401		2199	
Volume (vph)	111	684	97	202	784	567	154	249	404	98	144	41
Peak-hour factor, PHF	0.90	0.93	0.88	0.91	0.94	0.92	0.91	0.91	0.91	0.92	0.92	0.92
Adj. Flow (vph)	123	735	110	222	834	616	169	274	444	107	157	45
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	123	735	110	222	834	616	0	443	444	0	309	0
Heavy Vehicles (%)	13%	11%	10%	1%	11%	20%	9%	7%	4%	22%	10%	6%
Parking (#/hr)			43	49							16	
Turn Type	Prot		Perm	Prot		Perm	Perm		Perm	Perm		
Protected Phases	3	1		3	1			2			4	
Permitted Phases			1			1	2		2	4		
Actuated Green, G (s)	23.0	52.0	52.0	23.0	52.0	52.0		40.0	40.0		19.0	
Effective Green, g (s)	23.0	51.0	52.0	23.0	52.0	51.0		40.0	40.0		18.0	
Actuated g/C Ratio	0.15	0.34	0.35	0.15	0.35	0.34		0.27	0.27		0.12	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0		4.0	4.0		4.0	
Lane Grp Cap (vph)	248	1057	353	182	1115	463		478	374		264	
v/s Ratio Prot	0.08	0.24		c0.19	0.26							
v/s Ratio Perm			0.11			c0.45		0.25	c0.32		c0.14	
v/c Ratio	0.50	0.70	0.31	1.22	0.75	1.33		3.93dl	1.19		1.27dl	
Uniform Delay, d1	58.2	42.8	35.9	63.5	43.2	49.5		53.6	55.0		66.0	
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00		1.00	1.00		1.00	
Incremental Delay, d2	6.9	3.8	2.3	138.2	4.6	163.0		26.4	108.0		109.5	
Delay (s)	65.1	46.6	38.2	201.7	47.8	212.5		80.0	163.0		175.5	
Level of Service	E	D	D	F	D	F		F	F		F	
Approach Delay (s)		48.0			128.9			121.6			175.5	
Approach LOS		D			F			F			F	

Intersection Summary

HCM Average Control Delay	110.5	HCM Level of Service	F
HCM Volume to Capacity ratio	1.25		
Actuated Cycle Length (s)	150.0	Sum of lost time (s)	18.0
Intersection Capacity Utilization	64.6%	ICU Level of Service	C
Analysis Period (min)	15		

dl Defacto Left Lane. Recode with 1 though lane as a left lane.
c Critical Lane Group

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations	↰	↱	↑	↱	↰	↑
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Lane Width	3.4	3.7	3.6	3.7	3.5	3.6
Grade (%)	0%		3%			-2%
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt	1.00	0.85	1.00	0.85	1.00	1.00
Flt Protected	0.95	1.00	1.00	1.00	0.95	1.00
Satd. Flow (prot)	**1697**	**1617**	**1835**	**1562**	1767	**1273**
Flt Permitted	0.95	1.00	1.00	1.00	0.95	1.00
Satd. Flow (perm)	1697	1617	1835	1562	1767	1273
Volume (vph)	223	347	308	279	337	388
Peak-hour factor, PHF	0.92	0.92	0.93	0.93	0.92	0.91
Adj. Flow (vph)	242	377	331	300	366	426
RTOR Reduction (vph)	0	0	0	0	0	0
Lane Group Flow (vph)	242	377	331	300	366	426
Heavy Vehicles (%)	4%	1%	2%	3%	2%	1%
Parking (#/hr)						46
Turn Type		Perm		Perm	Split	
Protected Phases	1		2		3	3
Permitted Phases		1		2		2
Actuated Green, G (s)	49.0	49.0	41.0	41.0	43.0	84.0
Effective Green, g (s)	49.0	49.0	41.0	41.0	43.0	84.0
Actuated g/C Ratio	0.34	0.34	0.28	0.28	0.30	0.58
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0
Lane Grp Cap (vph)	573	546	519	442	524	773
v/s Ratio Prot	0.14		0.18		c0.21	0.16
v/s Ratio Perm		c0.23		c0.19		0.17
v/c Ratio	0.42	0.69	0.64	0.68	0.70	0.55
Uniform Delay, d1	37.1	41.5	45.5	46.2	45.2	18.8
Progression Factor	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	2.3	7.0	5.9	8.1	7.5	2.8
Delay (s)	39.3	48.5	51.4	54.3	52.8	21.7
Level of Service	D	D	D	D	D	C
Approach Delay (s)	44.9		52.8			36.0
Approach LOS	D		D			D

Intersection Summary

HCM Average Control Delay	43.9	HCM Level of Service	D
HCM Volume to Capacity ratio	0.69		
Actuated Cycle Length (s)	145.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	57.2%	ICU Level of Service	B
Analysis Period (min)	15		

c Critical Lane Group