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Design and Analysis of a Pre-Timed Traffic Signal for an Isolated Intersection in Benin City, Edo State, Nigeria

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Abstract

The use of traffic warders in controlling the flow of vehicles across Iyaro intersection, in recent times, has lead to an increase in road user's costs in terms of excessive delay, funds, vehicular fuel consumption and exposure to fumes from idling vehicles. This has therefore necessitated a more effective traffic control scheme. This study carries out the design and analysis of a pre-timed traffic signal system in order to improve the intersection capacity while reducing the vehicular delay in the study area. Traffic volume count was obtained using the manual method of data acquisition in which traffic observers counted the number, composition and directional movements of vehicles into the intersection for every 15 minutes interval from 6.00a.m. to 6.00p.m. Saturation flow headway was acquired using the video recording technique. This data acquired was used in designing the pre-timed signal timing parameters. The phase design for the intersection was carried out using protected or permitted left turn's model obtained from Highway Capacity Manual. The intergreen period, lost times, pedestrian clearance interval and signal timings design was carried out in accordance with the procedure set up in Canadian capacity guide. The result from the phase design showed that a three phase cycle was the most efficient cycle structure for controlling the volume of traffic moving through this intersection. Signal timing parameters such as an amber value of 2 seconds (s), all-red period of 2s and a cycle length of 120s which was split into 60s for phase 1; 17s for phase 2; and 26s for phase 3 were obtained. The result of the analysis of the signal timing design showed that a level of service (LOS) of C was obtained. This is in conformity with the level of service normally employed for urban intersection design.

Key word: Traffic control scheme, Pre-timed traffic signal, Level of service, Phase design.

1.0 Introduction

Traffic congestion is a severe problem at an urban intersection causing many critical problems and challenges in major and most populated cities around the world due to increasing population and economic activity [1]. The earlier practice has been to control traffic by means of stationing traffic police officers at intersections by showing stop signs alternately so that one of the traffic streams may be allowed to move while the cross traffic is stopped. Thus the crossing streams of traffic flow were separated by time [2]. Many times accidents happen due to the poor performance of the system of control. Moreover, inadequate control by traffic wardens contributed to the delay because when the traffic warden is exhausted before he is relieved by another traffic warden, it becomes difficult for him to concentrate and process traffic stream approach at the right time [3].

The aim of every driver is to reach the destination without wasting time and fuel during the course of his trip. So, traffic management at road intersection is to crucially reduce waiting and travelling times, save fuel and money [4]. There have been several methods of controlling conflicting streams of vehicles at an intersection [5]. The choice of methods depends on the type of intersection and the volume of traffic in each of the conflicting streams. Intersection controls are made up of yield signs, stop signs, multi-way signs, intersection channelization, rumble strips and traffic signals [6]. The origin of traffic control signal can be traced back to the manually operated semaphores first used in London as early as 1868. Its purpose was to give protection to members of parliament and lesser street crossers to a point where vehicle traffic was heavy [7, 8]. This

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control device has been used in different countries as a means of directing the flow of vehicles at an intersection [2].
Traffic signal control is a measure that is commonly used at road intersections to minimise vehicular delays [5]. Traffic signal control at road intersections allows vehicle movements to be controlled by allocating time intervals, during which separate traffic demands from each approach to the intersection can make use of the available road space [9]. The purposes of traffic signals at intersections are to draw attention, provide meaningful and timely response, and to provide minimum wastage of time by vehicles moving through these intersections [10].
Modern traffic signals allocate time in a variety of ways, from the simplest two-phase pretimed mode to the most complex multiphase actuated mode [11]. There are three types of traffic signal controllers: Pretimed, in which a sequence of phases is displayed in repetitive order. Each phase has a fixed green time and change and clearance interval that are repeated in each cycle to produce a constant cycle length; Fully actuated, in which the timing on all of the approaches to an intersection is influenced by vehicle detectors. Each phase is subject to a minimum and maximum green time, and some phases may be skipped if no demand is detected. The cycle length for fully actuated control varies from cycle to cycle; Semi actuated, in which some approaches (typically on the minor street) have detectors and some of the approaches (typically on the major street) have no detectors [11].
The objectives of the study include: carry out the traffic volume count and average flow headway survey; design the pretimed signal timing parameters and analyse the design.

2.0 The Study Area

The study area is situated along Urubi-Lagos Road in Oredo Local Government area, Benin City, Edo state, Nigeria. It lies between latitude 06°21'6.38" to 06°21'7.80"N and longitude 05°37'43.22" to 05°37'44.80"E**.** It is a four-legged intersection that comprises of Federal Government roads, which is the Dawson-Urubi-Uselu road from the East-West axis and the Lawani-Evbiemwen from the North-South axis, as shown in Fig. 1. The major roads measure 11.50 meters in width from the median to the edge of curb while the minor roads measure 5.75 meters in width from the centreline to the edge of curb. Commercial buildings and filling stations and their accesses are found close to the intersections. This intersection works in isolation i.e. it controls traffic without considering adjacent signalized intersections.

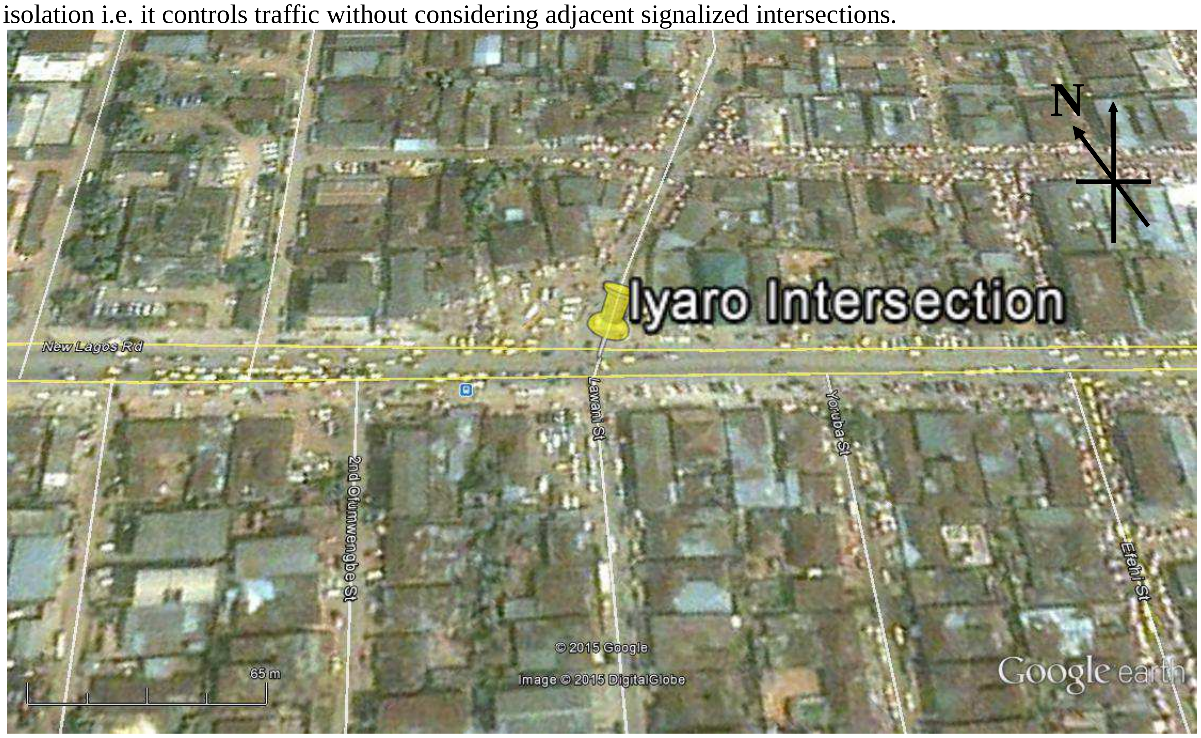

Fig. 1: Satellite imagery showing the study area, Iyaro Intersection [12]

3.0 Methodology

3.1 Data Collection

For this study, the manual method of conducting traffic volume count was carried out. This method was used to gather turning movement counts and composition of traffic moving into the study area. For each approach lane into the intersection, a traffic observer was stationed at a safe distance. The turning movements of vehicles and the composition of traffic were recorded into a data form for every 15 minutes interval which lasted from 6.00a.m. to 6.00p.m. for three days (Tuesday,

Wednesday and Thursday). This is in line with traffic volume studies done by other researchers [13].
Video recording technique was used in acquiring saturation flow headway [14]. For each approach lane, a traffic observer held a camcorder and stood perpendicular to the stop line while recording the flow of vehicle into the intersection. This technique was used for this study when compared to other forms of data acquisition in capturing saturation flow headway because this technique could be replayed several times to minimize any error when extracting data. This was carried out between 8.00am to 10.00am and 4.00pm to 6.00pm during the duration of traffic volume count.

3.2 Data Analysis

3.2.1 Intersection Traffic Volume Count Analysis

Traffic volume count, which was recorded in vehicles per hour from all approach lanes, was converted to passenger car unit per hour (pcu/h). This was carried out by multiplying the various vehicle classifications by their respective conversion values. The values used in the study area, which are in conformity with Road Sector Development Team are 1 for cars, 1.3 for trucks (rigid 2 to 4 axles) and 1.8 for buses/trailers (rigid 5 and above axles) [15]. The arrival flows in pcu/hr, peak hour factors (PHF) and adjusted design flow in pcu/hr was calculated for each lane using equation (1), (2) and (3) respectively [14, 16]

$$q = \sum_K f_k q_k \tag{1}$$

$$PHF = \frac{Hourly\,volume}{Maximum\,15\,\min ute\,volume * 4} \tag{2}$$

$$q_{adj} = \frac{q}{PHF} \tag{3}$$

where:
q = Arrival flow in a given lane (pcu/h)
f_k= Passenger car unit equivalent of a vehicle category k (pcu/h)
q_k= Flow of vehicles of category k in a given lane (veh/h)
q_{adj}= Adjusted design flow
PHF = Peak hour factor

3.2.2 Saturation Flow Headway Survey Analysis

The analysis of the saturation flow headway survey was carried out in accordance with the Canadian capacity guide for signalised intersection [14]. The video clip for saturation flow headway survey previously acquired by traffic observers was taken back to the studio for data extraction. Data extraction was carried out by two observers; the first observer was responsible for dictating the classification of vehicles which passed through the stop line while the second observer was responsible for entering data called out by the first observer as well as timing the process with a stop watch. The first observer shouted 'G' at the instant when the green signal appeared while the timekeeper started timing the stopwatch. When the front bumper of the first vehicle crossed the stop line, the observer shouted 'C' (for cars), 'T' (for trucks) and 'B' (for buses/trailers). When the front bumper of the next vehicle reached the stop line, the vehicle was identified in the same manner as for the first vehicle. The timekeeper wrote down the letters dictated by the observer while moving to a new column after every five seconds. This was repeated for all approach lanes. The numbers in the portions of the green interval written down was converted to passenger car unit, using the passenger car equivalents for individual vehicle categories from Road Sector Development Team [15]. The saturation flow rate was calculated using equations (4) and (5)

$$h = \frac{t_g n_s}{v_s} \tag{4}$$

$$S_i = \frac{36000 * v_s}{t_g * n_s} \tag{5}$$

where:
h = Average saturation flow headway (s)
t_g = Duration of the green interval increments (5s, or exceptionally 10s)
n_s = Number of fully saturated increments of green intervals.
V_s = Total number of passenger car units in all saturated portions of green intervals
S_i = Saturation flow in a given increment of the green interval (pcu/h)

3.3 Design of Pre-Timed Signal Timing Parameters

3.3.1 Phase Design of the Intersection

The phase design for the intersection was carried out by using protected or permitted left turn's model obtained from

Highway Capacity Manual. The study took into consideration that a separate phase would be allocated if the product of the left turns and the opposing through movement for one hour exceeded: 50,000 vehicles per hour (veh/hr) for one opposing through lane, 90,000 veh/hr for two opposing through lanes or 110,000 veh/hr for three or more opposing through lanes [16].

3.3.2 Intergreen period and lost time analysis

The inter-green periods which consists of the amber interval and the all-red period was calculated using equations (6), (7) and (8) respectively [14]

$$A = t_{pr} + \frac{v}{2a} \quad (6)$$

$$r_{alli} = I - A \quad (7)$$

$$I = i + \frac{W_c + L_{veh}}{v_c} \quad (8)$$

where:
A = Amber interval (s)
t_{pr} = Time of perception and reaction (s) assumed to be 1.5s
v = Speed (m/s) assumed to be 15m/s
a = Average deceleration rate (m/s^2) usually taken to be 3.0 m/s^2
r_{alli} = All - red period (s)
I = Intergreen period (s)
A = Amber interval (s)
i = Amber overrun = A - 1 = 4 - 1 = 3s
W_c = distance to clear (m), measured from the stop line to the far end of the potential conflict zone for the most critical combination of lanes for which the green interval terminates, and lanes for which the green indication is about to start = 20m
L_{veh} = Length of the clearing vehicle (m) usually taken as the space for a passenger car = 6.0m
v_c = Clearing speed based on regional practice = 10.0 m/s
Lost time was calculated using equation (9):

$$L_j = I_j - 1 \quad (9)$$

where:
L_j = Lost time associated with phase j (s)
I_j = Inter-green period between phases j and (j+1) (s)

3.3.3 Pedestrian Clearance Interval Design

There was no pedestrian refuge on any of the approach lanes. Therefore, the minimum pedestrian walk interval was selected in conformity with the standard set out in Manual of Uniform Traffic Control Devices. The minimum walk intervals and pedestrian clearance periods was calculated using equation (10) and (11) respectively [14]

$$w_{min} = 10.0\text{s for all crosswalks} \quad (10)$$

$$w_{cleari} = \frac{w_{ped}}{v_{ped}} \quad (11)$$

where:
w_{min} = Minimum walk interval (s)
w_{cleari} = Pedestrian clearance interval (s)
w_{ped} = Length of the crosswalk measured midway between lines
v_{ped}= Pedestrian walking speed = 1.0 m/s

3.3.4 Signal Timing Design

The signal timing design was carried out in accordance with the procedures set out by [14, 17]. Average peak arrival flows in pcu/hr was allocated to their respective phases. The flow ratio for a given lane was calculated using equation (12). Under the assumption that all lane arrival flows depart during the green interval at full saturation flow, the lane with the highest flow ratio determined the duration of the green interval for that phase [8]. The intersection flow ratio was calculated using equation (13). Optimum cycle length was calculated by applying equation (14). Minimum cycle length needed to accommodate pedestrians was calculated using equation (15). The total time available in the cycle for the allocation of green intervals was calculated using equation (16). The total available green time per phase was allocated in proportion to the flow ratio of the critical lanes for the corresponding phases and the intersection flow ratio and was calculated using equation (17).

$$y_i = \frac{q_{iadj}}{S_i} \tag{12}$$

$$Y = \sum_j y_{ij} = \sum_j \left(\frac{q_{ij}}{S_{ij}}\right) \tag{13}$$

$$C_{opt} = \frac{(1.5 * L + 5)}{(1 - Y)} \tag{14}$$

$$c_{ped\ \min} = \sum_j \max(w_{\min i} + w_{clear\ i})_j \tag{15}$$

$$\sum g_j = c - \sum I_j \tag{16}$$

$$g_j = \frac{\sum g_j y_j}{Y} \tag{17}$$

where:
y_i = Flow ratio for lane i
q_{iadj} = Adjusted arrival flow in lane i (pcu/hr)
S_i = Adjusted saturation flow of lane i (pcu/h)
Y = Intersection flow ratio
y_{ij} = Flow ratio for the critical lane i in phase j (pcu/hr)
q_{ij} = Arrival flow of critical lane i in phase j (pcu/hr)
S_{ij}= Saturation flow of critical lane i in phase j (pcu/hr)
$\sum_j$ = Summation over critical lanes in phases j (one critical lane or each phase)
C_{opt} = Optimum cycle time (s)
L = Intersection lost time (s) = 5+5+5= 15s
$c_{ped\ min}$ = Minimum cycle time required for pedestrian (s)
$w_{min\ i}$= Minimum pedestrian walk interval for crosswalk i (s)
$w_{clear\ i}$= Pedestrian clearance period for crosswalk i (s)
$max(w_{min\ i}+ w_{clear\ i})_j$ = Maximum of the sum of the minimum walk interval plus the corresponding clearance period in each phase j (s).
$\sum g_j$= Total green time available in the cycle (s)
c = Selected cycle time (s)
I_j= Inter-green period following phase j (s)
g_j = Green interval for phase j (s)
y_j = Flow ratio for the critical lane in phase j (s)
$\sum g_j$= Total green time available in the cycle (s)

3.4 Analysis of Signal Timing Design

The analysis of signal timing parameterswas carried out in accordance with the procedure detailed in Highway Capacity Manual for signalized intersection [17]. Capacity of lane i during phase j was calculated using equation (18). Degree of saturation model was calculated using equation (19). The basic equation for estimating average intersection control delay model was calculated using equation (20). The average overall intersection control delay model was calculated using equations (21). The Level of Service (LOS) was obtained by comparing the values obtained from the average overall intersection control delay model of the designed signal timing parameters to the LOS value for signalized intersections [16, 18].

$$C_{ij} = s_{ij} * \frac{g_{ej}}{c} \tag{18}$$

$$x_i = \frac{q_i}{C_i} \tag{19}$$

$$d = k_f d_1 + d_2 + d_3 \tag{20}$$

$$d_{\text{int}} = \frac{\sum_j \sum_i q_{ij} d_{ij}}{\sum_j \sum_i q_{ij}} \tag{21}$$

where:
C_{ij}= Capacity of lane i in phase j (pcu/h)
S_{ij} = Saturation flow of lane i in phase j (pcu/h)
$g_{e\,j}$= Effective green interval of phase j (s)
c = Cycle length (s)
x_i= Degree of saturation of lane i
q_i= Arrival flow of lane i (pcu/h)
C_i= Capacity of lane i (pcu/h)
d = Average overall delay (s/pcu)
d_{int} = Average overall intersection delay
k_f = Adjustment factor for the effect of the quality of progression, with

$$k_f = \frac{(1 - \frac{q_{gr}}{q})\, f_p}{\left(1 - \frac{g_e}{C}\right)} \tag{22}$$

q_{gr}/q = Proportion of vehicles arriving during the green time
q_{gr} = Average number of arrivals during green interval
f_p = Supplemental adjustment factor for platoon arrival time
d_1 = Average overall uniform delay (s/pcu)
d_2 = Average overflow delay (s/pcu)
d_3= Initial queue delay (s/pcu)

$$d_1 = \frac{c(1 - g_e / c)^2}{[2(1 - x_1 g_e / c)\}} \tag{23}$$

$$d_2 = [(x-1) + \sqrt{(x-1)^2} + \frac{240x}{Ct_e}]15_{te} \tag{24}$$

$$d_3 = \frac{(1800 Q_b (1+u)t)}{ct_e} \tag{25}$$

where:
c = Cycle time (s)
g_e = Effective green interval (s)
x_1 = Minimum of (1.0, x)
x = Degree of saturation
q = Arrival flow (pcu/h)
C = Capacity (pcu/h)
t_e= Analysis period (hour)
Q_b = Initial queue at the start of period t_e (pcu)
u = Delay parameter
q_{ij}= Arrival flow in lane i in phase j (pcu/h)
d_{ij} = Average overall delay for vehicles in lane i departing in phase j (s/pcu)
$\sum_j \sum_i$ = Summation over individual lanes i and over phase j

4.0 Results and Discussion

4.1 Results

Table 1 shows the hourly flow rate of vehicles moving into the intersection and the total hourly turning movement from each approach lane obtained from the intersection traffic count.

Table. 1: Traffic volume count at Iyaro intersection for all approaches in pcu/hr

Hour	From Dawson			From Urubi			From Lawani			From Evbiemwen			Total
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT	
6.00am - 7.00am	64	1036	38	51	840	13	68	38	50	12	34	32	2276
7.00am - 8.00am	81	1089	91	77	1178	21	115	60	65	26	42	47	2892
8.00am - 9.00am	47	1397	101	85	1293	30	144	40	123	25	85	43	3413
9.00am - 10.00am	64	1063	38	106	1174	47	85	34	102	38	83	55	2889
10.00am-11.00am	38	1122	30	115	1040	72	132	55	119	21	64	42	2850
11.00am-12.00pm	51	1238	43	85	1065	55	151	34	93	30	76	43	2964
12.00pm - 1.00pm	72	1074	64	132	1052	21	132	47	64	32	51	39	2780
1.00pm - 2.00pm	47	1337	43	127	1045	13	123	49	43	45	68	64	3004
2.00pm - 3.00pm	64	1329	85	136	1201	21	127	51	89	85	82	43	3313
3.00pm - 4.00m	81	1265	89	149	1123	21	149	57	109	51	98	64	3256
4.00pm - 5.00pm	106	1316	51	157	1229	26	149	32	81	34	100	45	3326
5.00pm - 6.00pm	85	1288	34	132	1235	34	183	34	101	47	110	50	3333

LT = Left turn movement
TH= Through movement
RT = Right turn movement

The total traffic volume analysis trend in passenger car unit per hour for the study area is presented in Fig. 2. This shows the hourly variation in the volume of passenger car unit making use of this intersection.

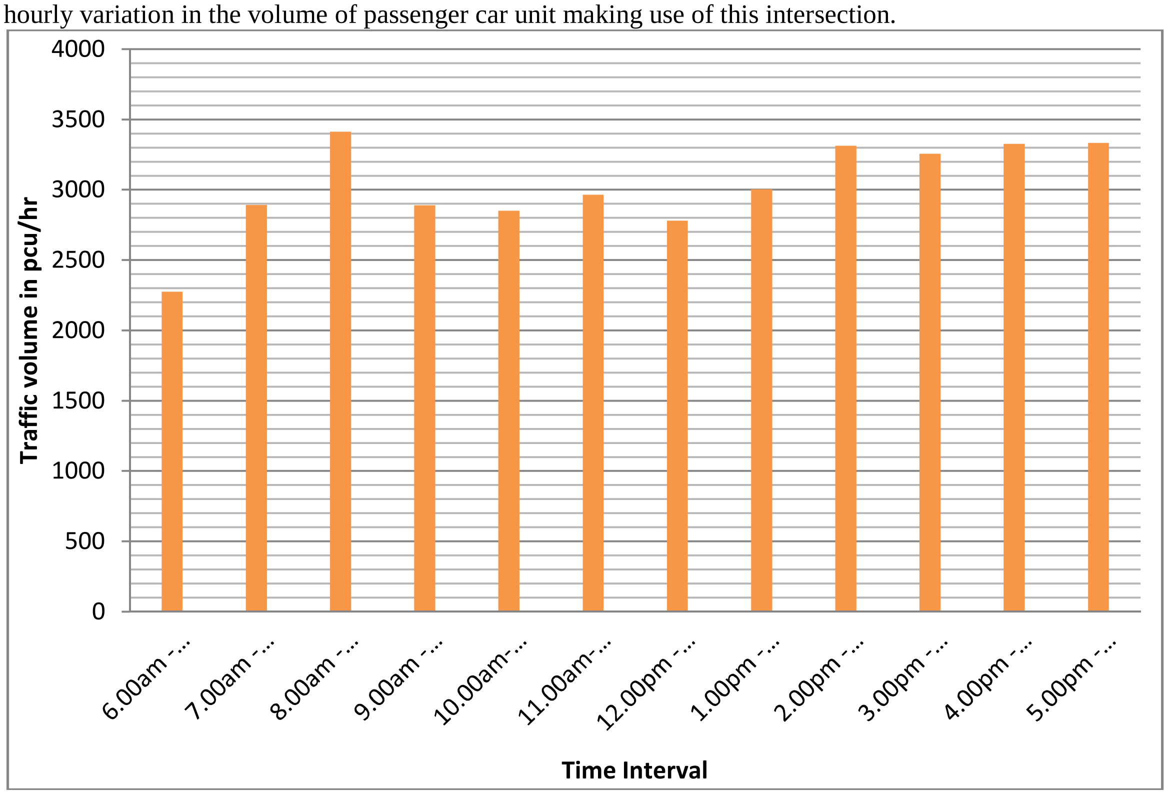

Fig. 2: Total traffic volumes at Iyaro intersection for each hour

Table 2 shows the saturation flow rate per lane expressed in pcu/hr/lane obtained from the average saturation flow headway survey.

Table. 2: Average Saturation Flow Rate Per Lane

Movements	Saturation Flow pcu/hr/lane
D-E (LT)	1400
D- U(TH)	1850
U-L(LT)	1360
U-D (TH)	1820
E-L (TH)	1410
E-U (LT)	1050
L-E (TH)	1026
L-D (LT)	1050

Fig. 3 shows the allowed left turn movements during the designed three phase cycle based on examination of the arrival flows and allowable movements, presented in Table 3.

Table. 3: Left Turn Protection Calculations

Movement	Flow rate (Veh/hr)	Cross Product	HCM guideline	Recommended LT phasing
NB-LT	183	10431	50000	**PERMITTED**
SB-TH	57			
SB-LT	183	20130	50000	**PERMITTED**
NB-TH	110			
EB-LT	157	219329	110000	**PROTECTED**
WB-TH	1397			
WB-LT	106	137058	110000	**PROTECTED**
EB-TH	1293			
where:				
NB= Northbound		LT = Left turn movement		
SB = Southbound		TH = Through movement		
EB = Eastbound				
WB = Westbound				

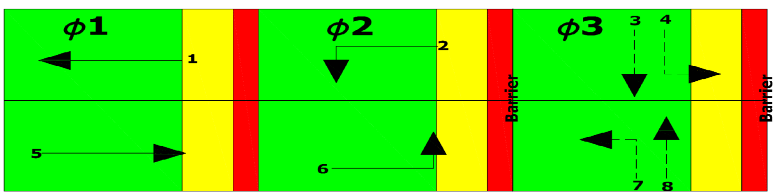

Fig. 3: Three Phase Cycle Configuration Showing the Allowed Movements

Table 4 and Table 5 show the basic vehicular and pedestrian timing requirement for the intersection based on the intersection layout.

Table 4: Basic Vehicular Timing Requirements

Approach lane	Phase	Amber interval (s)	All-red period (s)	Intergreen period (s)	Lost time (s)
U-R(TH)	1	4	2	6	5
D-U (TH)	1	4	2	6	5
U-L(LT)	2	4	2	6	5
D-E(LT)	2	4	2	6	5
E-L (TH&LT)	3	4	2	6	5
L-E (TH&LT)	3	4	2	6	5

Table. 5: Basic Pedestrian Timings Requirements

Crosswalk	Phase	Walk Interval (s)	Clearance Period (s)
E	1	10	11
L	2	10	11
U	3	10	21
D	3	10	21

Table 6 shows the critical flow ratios in each phase for the intersection

Table. 6: Determination of Flow Ratios for Peak Period

Direction	Phase	Adjusted lane arrival flow q (puc/h)	Saturation flow S (pcu/h)	Lane flow ratio y=q/S	Flow ratios for critical lanes y_{crit}
U-D(TH)	1	1293	3640	0.3552	0.3776
D-U (TH)	1	1397	3700	0.3776	
U-L(LT)	2	157	1360	0.1154	0.1154
D-E(LT)	2	106	1400	0.0757	
E-L (TH)	3	110	1410	0.0780	0.1743
L-E (TH)	3	60	1026	0.0585	
E-U (LT)	3	85	1050	0.0810	
L-D (LT)	3	183	1050	0.1743	
				Intersection flow ratio Y=£y_{crit}=	0.6673

Table 7 shows the available green time allocated in proportions to the flow ratio of the critical lane for the corresponding phase and the intersection flow ratio.

Table. 7: Summary of Vehicular Timing for Peak Period in the Study Area

Phase No	Flow ratios for critical lanes y_{crit}	Intersection flow ratio (Y)	Total green time (s)	Green interval for phase I ($\sum g_j * y_1/Y$)	Intergreen
1	0.3776	0.6673	102	58	6
2	0.1154	0.6673	102	18	6
3	0.1743	0.6673	102	27	6
Total				102	18
Cycle length				120	

Fig. 4 shows the final retimed signal timing design for the intersection on a timing diagram. This also shows the required effective green intervals, amber intervals and red intervals timings in seconds of all phases.

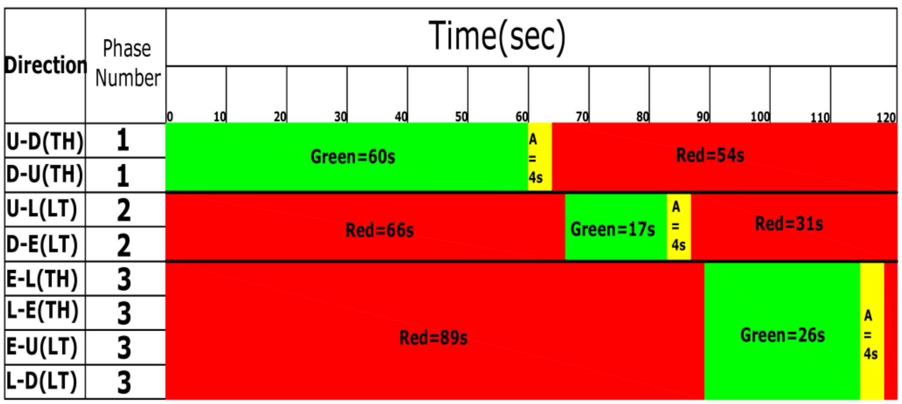

Fig. 4: Final retimed signal timing design shown in a timing diagram for the intersection.

Table 8 shows the maximum degree of saturation per phase of the retimed signal timing parameters, overall intersection control delay and the level of service that would be experienced by motorists.

Table. 8:Evaluation of retimed signal timing parameters

Movement direction	Phase No	Effective green interval (s)	Red interval (s)	Max Degree of Saturation per phase	Arrival flow (pcu/h)	Average overall control delay (s/pcu)	Weighted Control delay $\sum_j\sum_i q_{ij}d_{ij}$ (s/h)
U-R(TH)	1	61	54	0.68	1293	26.94	34833.4
D-U (TH)					1397	28.48	39786.6
U-L(LT)	2	18	97	0.65	157	71.82	11275.7
D-E(LT)					106	54.11	5735.7
E-L (TH)	3	27	89	0.67	110	41.38	4551.8
L-E (TH)					60	40.34	2420.4
E-U (LT)					85	42.69	3628.7
L-D (LT)					183	65.08	11909.6
Summary =					3391		114141.9
Overall intersection control delay =						33.66 s/pcu	
Level of Service =						C	

4.2 Discussion

The morning and evening peak period observed from Fig. 2 occurred between 8.00 a.m. to 9.00 a.m and 5.00 p.m to 6.00 p.m respectively. This was as a result of the fact that the intersection linked federal institutions, state secretaries, banks, hospitals to residential areas. From the saturation flow headway survey carried out, it was observed from Table 2, that there was a drop in saturation flow rate per hour for the various approaches. The condition of the pavement surface was the primary cause of this drop. The pavement surfaces of both minor lanes (Lawani and Evibiemwen) were in deplorable condition which caused an increase in the time headway between vehicles passing through the stop line into the intersection.

It was observed from Table 3, that the logical number of phases for this intersection was a three phase cycle. Shown in Table 3, the East bound left turn and West bound left turns needs to be protected. North bound left turn and South bound left turn can be paired together, due to the low cross product values compared with Highway Capacity Manual guideline for provision of protected left turn [16]. The designed three phase cycle configuration for the intersection in Fig. 3 indicates the significance of a three phase cycle structure as this would reduce the amount of time spent waiting for an allowed green interval movement while maintaining vehicular safety. The movements paired together reduce the chances of vehicular conflicts while steering through the intersection.

Fig. 4 shows the respective green intervals, amber intervals and red intervals in a timing diagram for each cycle phase. A cycle length of 120 seconds which was adopted as the optimum cycle length was capable of handling the minimum pedestrian clearance interval based on the current volume of traffic [19]. The first green interval of 60 seconds is allocated to through movements from Dawson and Urubi. While this movement is going on, all other signal heads are displayed red. After this interval, an amber interval of 4 seconds displays which is a warning to motorists that the green interval is about coming to an end. An all red interval of 2 seconds comes on before the next movements which is left turns either from Urubi to Lawani or Dawson to Evbiemwen displaying a green interval of 17 seconds, after which an amber interval of 4 seconds, then an all red interval of 2 seconds. The final phase is allocated to through movements and left turns from Evbiemwen and Lawani, both displaying a green interval of 26 seconds, after which an amber interval of 4 seconds then an all red interval of 2 seconds. Pedestrians are allowed to cross during phase 1 and phase 3, with a timing interval of 10 seconds which is in conformity with Canadian capacity guide for signalised intersection [14].

The maximum degree of saturation ratio per phase, obtained from Table 8, has a value of 0.68, which indicates that the intersection would be under saturated and would typically have sufficient capacity and stable operations for the designed signal timing parameters. The overall intersection control delay, also shown in Table 8, was consistent with the previously determined maximum degree of saturation ratio [16]. The average overall intersection control delay of 33.66seconds per passenger car unit (s/pcu) obtained was less than 35s/pcu which indicates that the design has a level of service (LOS) of grade C [18]. At LOS of grade C, the operation is stable, with more frequently and fully utilized signal phases. Moreover, many vehicles are expected to go through the intersection without stopping. This level is normally employed in urban intersection design [14].

5.0 Conclusion

In this study, a three phase signal cycle structure was used for designing the pre-timed signal timing parameters for the intersection at the study area. This design proffered a more efficient and economical movements of vehicles through this intersection. It was also observed that an improved level of service compared to the use of traffic warders was obtained. It is believed that the use of a traffic signal would be beneficial to the study area through the proper allocation of right of way to different streams of vehicle, thereby reducing travel time which invariably affects: fuel consumption and exposure to fumes from idling vehicles which would indirectly offer better living conditions to commuters making use of this intersection [19].

6.0 References

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