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Design of an Efficient Traffic Control Signal Using Webster's Model: A Case Study of Akure, Nigeria

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Abstract

The fast growth of cities and the increase in automobile usage have made important junctions more congested. This study employs the Webster model to design an efficient traffic signal for Araromi intersection in Akure, Nigeria, using the site traffic data. Results of field observations showed that passenger cars formed the dominant vehicle category, while the observed peak hour (7:45-8:45 AM) recorded a passenger car unit (PCU) of 6505 and a peak flow rate of 6920 pcu/hr. The critical flow ratio of 0.736 confirms that the intersection remains within capacity and the geometry is sufficient. The peak hour factor of 0.94 also indicates that a reliable and efficient signal control system can be effectively implemented at the intersection. The west approach contributed 41% of the total traffic, justifying its longer green interval, followed by the east (35%) and north (24%) approaches. The optimized signal design resulted in a cycle length of 110 seconds. Within this cycle, phases 1, 2, and 3 received 35, 32, and 28 seconds of effective green time, respectively, along with 4 seconds yellow interval for each phase. The signal achieved Level of Service (LOS) C on all approaches, with control delays of 29.94, 31.95, and 32.55 secs/veh, and a volume-to-capacity ratio below 1, indicating stable operations within capacity. This validates the efficiency of the Webster-based signal design and possible improvements in the performance of the intersection when implemented in pretimed or vehicle actuated traffic control systems. These findings provide a practical template for similar urban intersections in the implementation of efficient traffic signals.

Keywords: Traffic Signal Control, Road Intersection, Congestion, Level of Service, Akure Nigeria, Urban Transport

Introduction

The rapid urbanization and increase in vehicle ownership have exacerbated congestion levels, leading to longer travel times, higher fuel consumption, and increased environmental pollution [1,2]. Urban intersections play a pivotal role in the efficiency of transportation networks, often becoming points of congestion that hinder traffic flow and increase the likelihood of accidents [3]. In Akure, most of the major intersections experience severe traffic challenges, leading to significant delays and decreased productivity for road users [2,4]. The Araromi intersection in Akure, Nigeria, exemplifies these challenges, as traffic flow is often disrupted by rising vehicular volume and overwhelmed traffic wardens, necessitating a focused approach to traffic control design [5]. Designing an efficient traffic control sign is essential for managing traffic effectively, minimizing delays, and enhancing safety at this critical junction. Moreover, recent studies have highlighted the challenges posed by traffic congestion in Nigeria, emphasizing the need for efficient traffic control systems [4-10]. While the design and implementation of traffic control systems at urban intersections have been a major concern, a significant gap still exists with respect to using local traffic parameters for signal timings. Most of the present traffic control system layouts are heavily inclined towards high-tech integration such as adaptive signal control and real-time traffic monitoring without giving much consideration to essential traffic parameters that are specific to each site [11-17]. This oversight can lead to inefficiencies and safety concerns, particularly in complex urban environments where traffic patterns can vary significantly [18].

Therefore, designing an efficient traffic signal using Webster's model will significantly enhance traffic flow and safety at the intersection, through reductions in vehicle delay, queue length, and an improved Level of Service (LOS). Specifically, the proposed design seeks to create a more efficient traffic management solution, by focusing on the unique traffic parameters of this study area. This approach not only addresses the shortcomings of existing systems but also ensures that local traffic dynamics are prioritized, ultimately enhancing traffic flow and safety at the intersection.

Materials and Methods

Description of Study Area

Araromi intersection is a T-intersection on 7°15'16.4"N and 5°11'34.9"E along Oba-Adesida-road connecting the Central Business District (CBD) of Akure, Nigeria. As shown in Figure 1, this three-leg arterial intersection links Cathedral junction and the main market called Oja Oba, which are notable for substantial commercial and economic activities. Additionally, it serves major areas and key destinations like commercial, industrial, and residential zones, within the city. Figure 1 describes the various traffic movements through the intersection. Movements are classified into three signal phases: west approach (A, B, C), east approach (D, E, F) and north approach (H). However, routes D and G operate with continuous right of way, as they are excluded from signal control.

Data Collection

Observers were positioned at safe and suitable points around the intersection to collect traffic data, using instruments and devices such as cameras, stopwatch, recording sheets, among others. Peak hour traffic volume data were collected between 7:00 - 10:00 in the morning and between 16:00 to 19:00 in the evening for two weeks during weekdays. This selected time frames agree with the results of earlier studies concerning the peak periods in Akure [4,8,10]. It also elevates confidence in the most significant duration and data that can be employed for the proposed traffic signal design. The number of vehicles per hour on all approaches

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was converted into passenger car units per hour (PCU/hr) by multiplying the counts of various vehicle classifications with their corresponding percentage car equivalent factors (PCE), as established in Transportation Research Board [19] to be 3.0, 2.2, 1.0, 1.2, 0.5, 0.5, respectively for tractors/ trailer, trucks/ buses, passenger cars, tricycle, motorcycles, and bicycles.

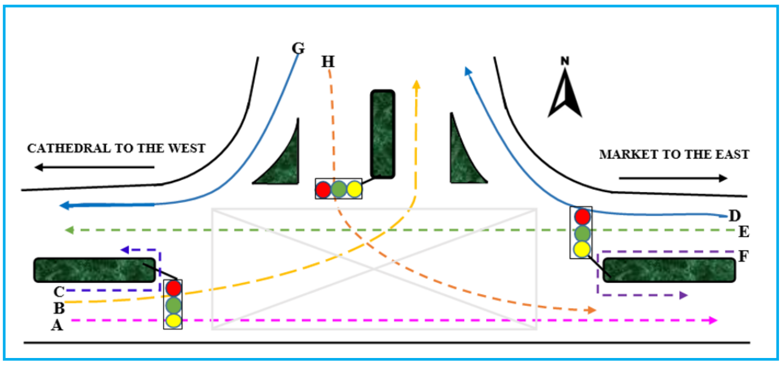

Figure 1. Traffic movements at Araromi intersection [17]

Traffic Analysis

Peak hour factor and peak rate of flow: The day with the highest observed traffic count was chosen for analysis. The peak hour, which was the hour with the highest four consecutive traffic count at 15 minutes' interval, was applied to determine the variation of traffic flow during this peak hour by calculating the peak hour factor (PHF). The peak flow rate also represents the number of vehicles expected to pass through the intersection during the peak hour. According to Transportation Research Board [19] and Kutz [20], the PHF and peak flow rate for the intersection is calculated using Equation 1 and 2, respectively.

$$PHF = \frac{Peak\ hour\ volume}{4 \times peak\ volume\ in\ 15\ minutes\ within\ peak\ hour} \quad (1)$$

$$Peak\ flow\ rate = \frac{Peak\ hour\ volume}{PHF} \quad (2)$$

Discharge headway and saturation headway: Discharge headway data were collected from each approach of the intersection utilizing a split timer. This process involved recording the successive durations taken for vehicles in a queue to pass the designated stop point when granted the right of way following the procedures outlined in Transportation Research Board [19]. Once the initial start-up lost time had passed and vehicles moved consistently, saturation headway was quantified as the average of constant discharge headway from the fourth vehicle onward in the line. Additionally, start-up lost time was subsequently calculated from the discharge headway of the first three vehicles on each approach and saturation headway using Equation 3. The clearance lost time was measured as the time gap between when the last vehicle passes as the warden stops the stream, and when the warden shifts control to the next stream. Further, the total lost time was taken as the sum of start-up and clearance lost time observed across different cycles and approaches.

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Where l_s is start up lost time, h_i is average discharge headway, and h_s is saturation headway.

$$l_s = \sum_{i=1}^{3}(h_i - h_s) \tag{3}$$

Saturation flow Rate: This is the maximum number of vehicles that can pass through a lane of the intersection under ideal conditions. Equation 4 [19] was applied as the most reliable way to compute the saturation flow rate from the observed field data to reflect the site-specific local condition of the intersection. Where $S_{per\ lane}$ is the Saturation flow rate (pcu/hr/lane), h_s is saturation headway.

$$S_{per\ lane} = \frac{3600}{h_s} \tag{4}$$

Flow ratio and critical flow ratio: Flow ratio is the ratio of peak flow rate to saturated flow rate for a specific lane group as in Equation 5 [19]. Further, the critical lane group is identified from each phase with the highest flow ratio, while the critical flow ratio is the sum of the flow ratios for all critical lane groups at the intersection. The critical flow ratio measures how much of the intersection's capacity is demanded and it is central to calculating cycle length. Where f_r is flow ratio, v is peak flow rate, S is saturation flow rate.

$$f_r = \frac{v}{S} \tag{5}$$

Signal Timing using Webster's Model

Traffic signal design is a systematic process of planning, implementing, and managing traffic control signals at the intersection using signals that indicate when to stop or proceed, and the signal timing parameters which are yellow interval, green interval and red interval.

Cycle length: Webster's formula in Equation 6 [19-21] provides a systematic approach to calculating the total time (in seconds) it takes for all approaches to go through one full set of traffic light indications. Where C is the optimal cycle length, l_t is the total lost time of all the phases, and y_n is the critical flow ratio.

$$C_0 = \frac{1.5\, l_t + 5}{1 - y_n} \tag{6}$$

$$Y = t_r + \frac{u}{2(a + Gg)} \tag{7}$$

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Yellow interval: The amber duration was designed using Equation 7 [21] to allow vehicles to stop if far or move if close by, as the green light of that phase is about to terminate. Where Y is the yellow interval, tr is perception-reaction time, u is the approach speed, a is the comfortable deceleration rate, G is % of grade divided by 100 (+ for upgrade, - for downgrade), and g is the gravitational constant.

$$G_{ei} = \frac{y_i}{y_n} \times (C_0 - l_t) \tag{8}$$

Green time: The effective green time of the different phases was determined in respect to the critical flow ratio using Equation 8 [20,21]. This allocation aimed to ensure that each movement received an appropriate duration of green time based on demand to optimize traffic flow and enhance efficiency, thereby improving overall traffic performance at the intersection. Where G_{ei} is the effective green for each approach, y_i is the critical lane flow ratio for each approach, y_n is the critical flow rate, C is the cycle length, l_t is the total lost time for all phases.

Red interval: This refers to the period during which conflicting approaches are held on a red signal as the traffic phase transitions to provide right of way to another approach, and is expressed in Equation 9 [19,21]. R is the red interval, C_0 is the total cycle length of the traffic light, Y is the yellow interval and G_{ei} is the effective green time for different phases.

$$R = C_0 - (G_{ei} + Y) \tag{9}$$

Performance and Efficiency

The efficiency of the traffic signal and the performance of this intersection is measured through volume to capacity ratio and level of service (LOS).

Volume to capacity ratio: The volume-to-capacity ratio measures how much of a phase's capacity is used by a lane group. It is the degree of saturation used to assess road performance and efficiency, as expressed in Equation 10 [19]. The critical lane volume v represents the peak flow rate in the numerator, and the corresponding capacity c is used in the denominator. Where v is peak flow rate, c is the capacity, S is the saturation flow rate of the lane group, G_{ei} is the effective green time, and C_0 is the cycle length.

$$(v/c) = \frac{v}{S\left(\frac{G_{ei}}{C_o}\right)} \tag{10}$$

Level of service: LOS is a grading system (A-F) that describes how well the intersection functions from the perspective of the road user. LOS for an intersection is measured mathematically using control delay in Equation (11) [19]. Where d is control delay, X_A is the volume to capacity ratio of the lane-group, c_A is average capacity, k is incremental delay factor (taken as 0.5), T

is analysis period duration (0.25 hr), I is upstream filtering adjustment factor (assumed as 0.9 for isolated intersection), ϕ is initial queue delay (taken to be zero), β is progressive factor (taken as 1 for isolated intersection).

$$d = 0.5\beta C_0\left(\frac{\left(1-\frac{G_{ei}}{C_O}\right)^2}{1-x_A\left(\frac{G_{ei}}{C_O}\right)}\right) + 900T\left((X_A-1)+\sqrt{(X_A-1)^2+\frac{8kIX_A}{c_A T}}\right) \tag{11}$$

Results and Discussion

Peak Day Traffic Volume

The highest daily traffic count observed at selected hours on all routes at the intersection is presented in Figure 2. This traffic count was taken on a Monday market day, when the intersection experienced the busiest vehicular movements. This is typically accompanied by increased fuel consumption, economic losses, and driver discomfort due to longer delays and stop-and-go conditions. The traffic shows a total of 11778 vehicles on the west approach, 11681 vehicles on the east approach, and 7240 vehicles on the north approach, while through movements contributed the most to the traffic. Similar movements in east and west approaches are observed, followed by the north approach. It suggests the major approaches for prioritization in signal design while ensuring safety for the minor approach. Although the traffic counts in [5] were lower, the overall distribution remained similar, indicating increased traffic demand over the years. This implies that signal timing must be designed to accommodate future growth in traffic demand. The equivalent PCU of the traffic count at different time intervals is shown in Figure 3. Further, Figure 4 showed that passenger cars formed the largest vehicle type in the traffic composition, which are dominated by taxis and private vehicles, followed by motorcycles, while tricycles are on the rise, with a negligible number of bicycles. These findings agree with [4,5,8,10] as they also reported the dominance of passenger cars and motorcycles.

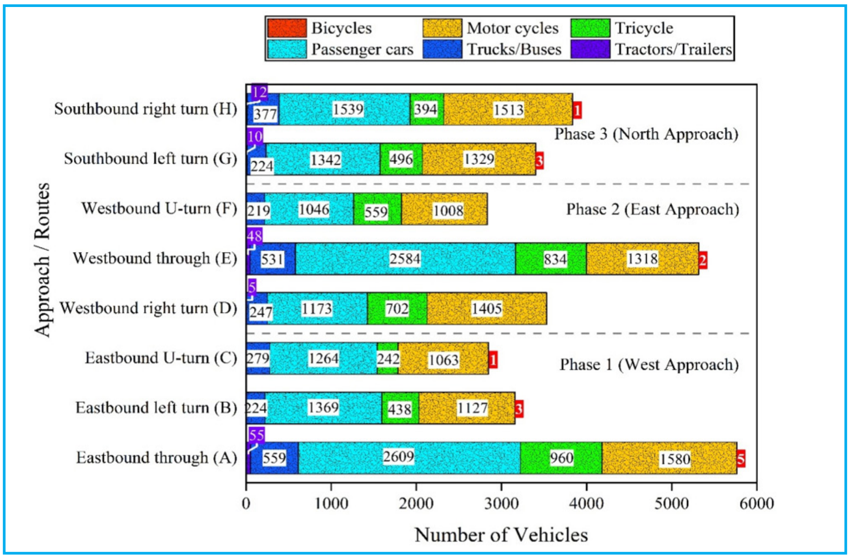

Figure 2. Traffic count for the peak day across all directions

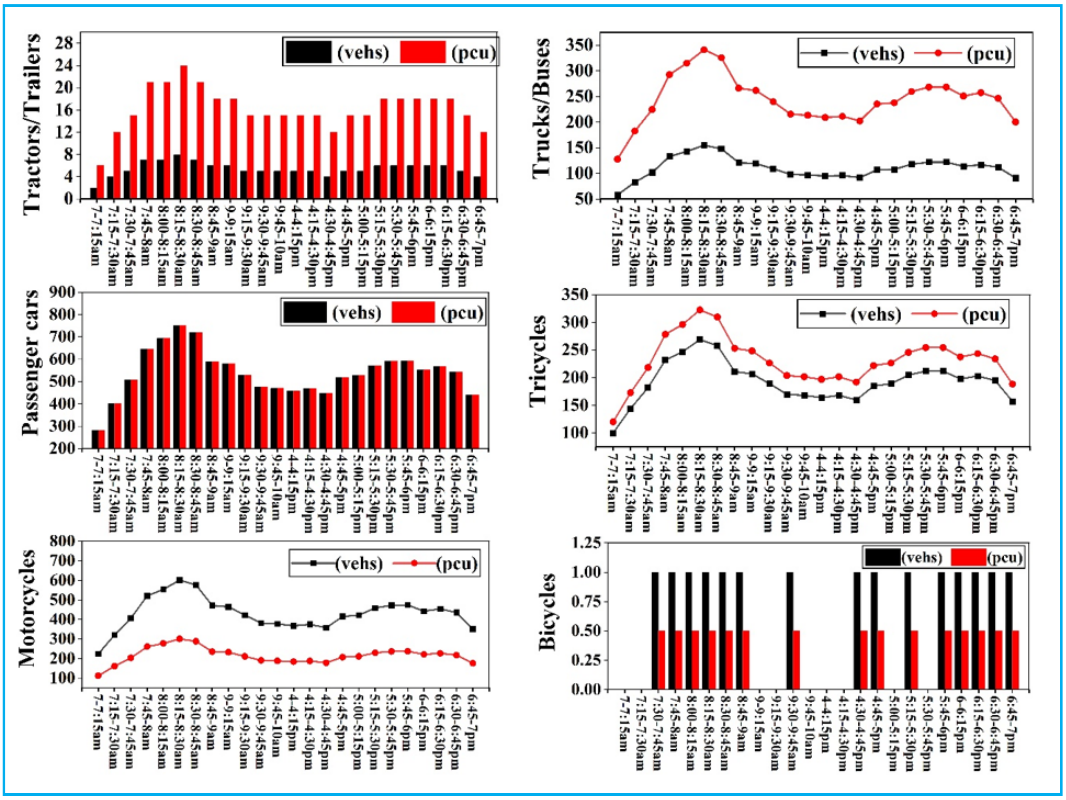

Figure 3. Peak day traffic composition and PCU equivalents

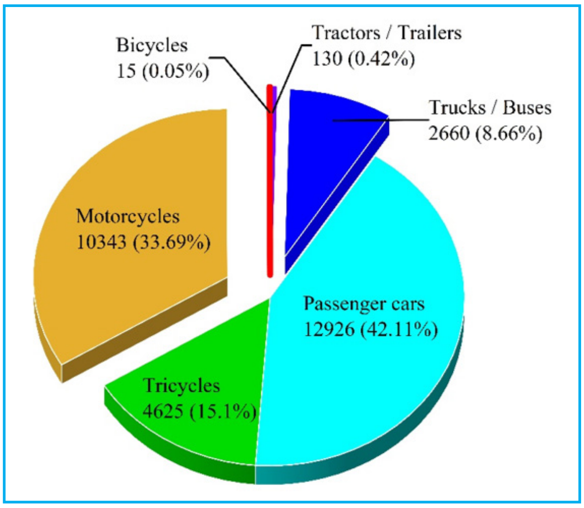

Figure 4. Proportion of vehicle types for the peak day

Peak Hour of the Traffic

Figure 5 shows gradual increase in traffic and the peak hour occurring early in the morning between 7:45 AM to 8:45 AM, with the west approach contributing the most to this peak hour. The travel behavior of people contributed to the peak hour, as more people travelled towards the market, schools and offices, indicating that commuters may be trying to arrive at work and destinations by the time. Following peak hour, tailback begins in the west approach, leading to slow movements of vehicles from 8:45 AM to 10:00 AM. However, traffic flow gradually increased between 4:00 PM and 6:00 PM, with relatively slow vehicular movements in both the east and west approaches. Moreover, high traffic demand from 6:00 PM to 7:00 PM intensified congestion levels. Meanwhile, the north approach experienced comparable traffic throughout the day, with higher volumes in the peak hours of 7:45 am to 8:45 am. Moreover, near peak hour traffic was observed in the evening, resulting majorly from the east approach due to commuters returning from the market direction to residential areas. The findings of [4,5,8] also suggest similar travel behavior to be the major contributor to the peak hours of morning and near peak hours of evening. Addressing peak-hour queues is essential to minimize loss of productive hours and economic losses for the road users, necessitating the implementation of efficient traffic control strategies to improve mobility. This includes optimizing cycle lengths to reduce queues and control delays during these peak periods.

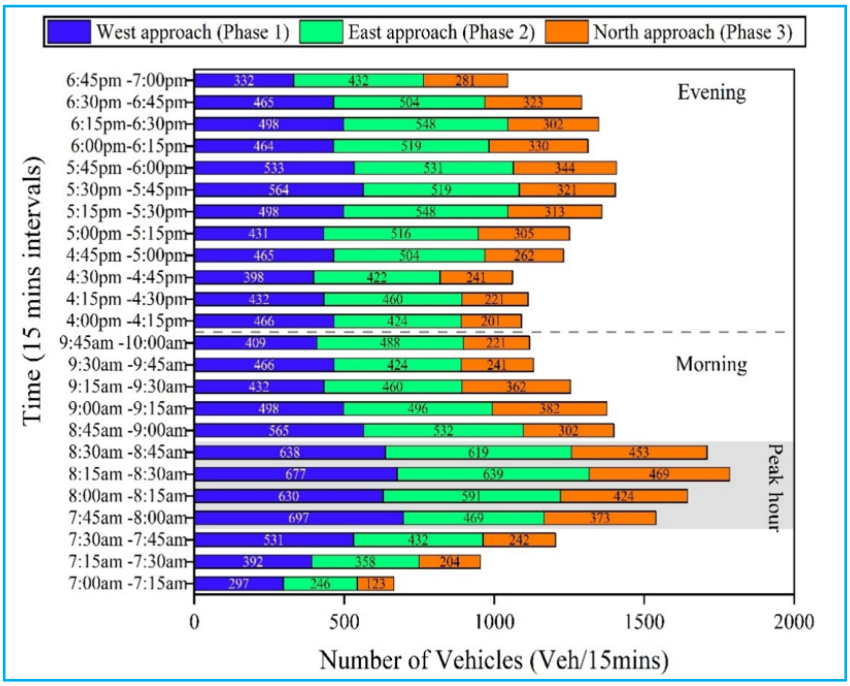

Figure 5. Peak day traffic at 15 mins intervals showing the peak hour

Peak Hour Volume

The distribution of traffic at the peak hour and the PCU equivalent is shown is presented in Table 1 and 2, respectively. A total of 6679 vehicles with an equivalent PCU of 6505 was recorded as the peak hour volume. Moreso, the peak fifteen minutes volume within this peak hour occurred between 8:15 and 8:30 with a PCU of 1737.9. Compared to this peak hour volume, [8] indicated a peak hour volume of 4111 PCU in 2023 along this corridor. This represents an estimated gain of 2394 PCU in traffic flow over a two-year span, potentially leading to heightened approach volumes that could intensify congestion, prolong delays, and diminish the overall level of service if not accompanied by appropriate signal timing. It also underscores the necessity of integrating current traffic counts into signal design, to guarantee that the intersection stays responsive to existing and future conditions. Additionally, the lane groups distribution of this peak hour volume is shown in Figure 6, with a total of 2665.8 pcu/hr, 2288.5 pcu/hr and 1550.7 pcu/hr respectively in the west, east and north approaches. This distribution is essential for further analysis of peak flow rate, flow ratios and critical flow ratio.

Table 1. Traffic composition at peak hour as measured in numbers of vehicles

Time interval	Approach	Tractors/ Trailers	Trucks/ Buses	Passenger cars	Tricycles	Motorcycles	Bicycles	Total	Total (vehs)
7:45am -8:00am	West	4	70	313	103	206	1	697	1539
	East	3	40	191	82	153	0	469	
	North	1	26	143	47	156	0	373	
8:00am -8:15am	West	3	64	268	101	193	1	630	1645
	East	3	61	253	80	194	0	591	
	North	1	18	169	67	169	0	424	
8:15am -8:30am	West	3	65	277	116	215	1	677	1785
	East	3	62	270	93	211	0	639	
	North	2	27	205	59	176	0	469	
8:30am -8:45am	West	2	64	274	111	186	1	638	1710
	East	3	55	258	92	211	0	619	
	North	1	27	188	55	182	0	453	
	Total	29	579	2809	1006	2252	4	6679	6679

Table 2. Traffic composition at peak hour as converted to PCU

Time interval	Approach	Tractors/ Trailers	Trucks/ Buses	Passenger cars	Tricycles	Motorcycles	Bicycles	Total	Total (vehs)
7:45am -8:00am	West	12	154	313	123.6	103	0.5	706.1	1506.6
	East	9	88	191	98.4	76.5	0	462.9	
	North	3	57.2	143	56.4	78	0	337.6	
8:00am -8:15am	West	9	140.8	268	121.2	96.5	0.5	636	1601.7
	East	9	134.2	253	96	97	0	589.2	
	North	3	39.6	169	80.4	84.5	0	376.5	
8:15am -8:30am	West	9	143	277	139.2	107.5	0.5	676.2	1737.9
	East	9	136.4	270	111.6	105.5	0	632.5	
	North	6	59.4	205	70.8	88	0	429.2	
8:30am -8:45am	West	6	140.8	274	133.2	93	0.5	647.5	1658.8
	East	9	121	258	110.4	105.5	0	603.9	
	North	3	59.4	188	66	91	0	407.4	
	Total	87	1273.8	2809	1207.2	1126	2	6505	6505

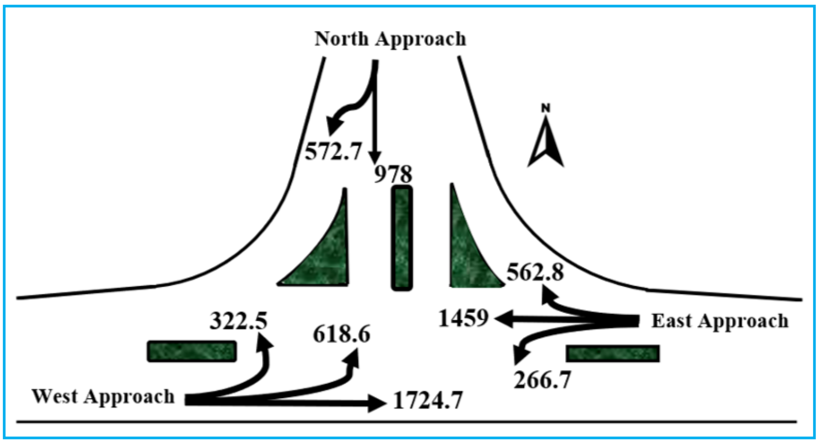

Figure 6. Peak hour volume in lane groups

Peak Hour Factor and Peak Flow Rate

Traffic analysis using 1737.9 pcu as the the peak 15 mins value within the peak hour, and the peak hour volume of 6505 pcu, produced a PHF of 0.94. This PHF value indicate that there are only minor variations in traffic demand during

15-minute periods of the peak hour. It also suggests that the observed traffic stream near to unity is consistent and that an efficient signal control system with a high level of dependability can be designed for the intersection [20]. By using the PHF value, peak flow rate was calculated as 6920 pcu/hr, which represents the overall demand for traffic on all routes during the peak hour. According to the lane groups distribution in Figure 7, the west approach has the highest traffic load, making up about 41% of the total peak flow rate, followed by the east and north approaches, which carry 35% and 24% of the demand, respectively. Since the allocation of effective green time in signal timing is based on critical lane volumes and demand distribution in achieving operational efficiency [19-21], the comparatively greater demand in the west approach emphasizes the necessity for priority allocation to reduce latency and avoid queue spillback. Moreover, this distribution provides for a proportional allocation of green times among the other approaches, thereby lowering the possibility of oversaturation.

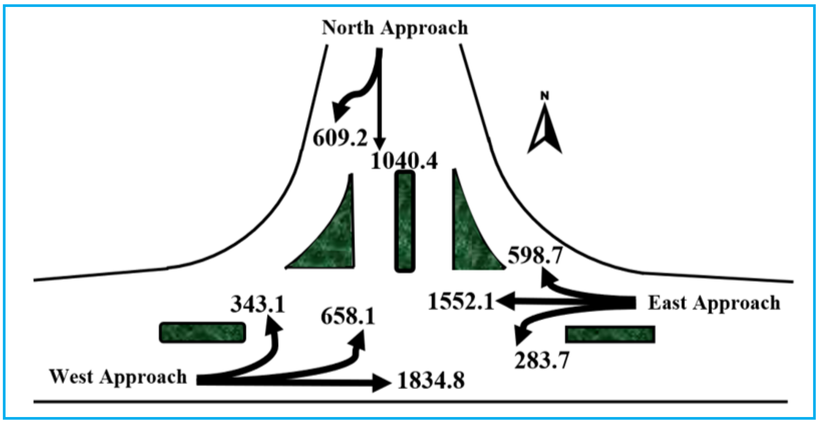

Figure 7. Peak flow rate for each approach

Discharge Headway and Saturation Headway

Figure 8 illustrates the variation of average discharge headway (hi) with vehicle position in the queue for the West, East, and North approaches of the intersection. A consistent pattern is observed across all approaches. The first

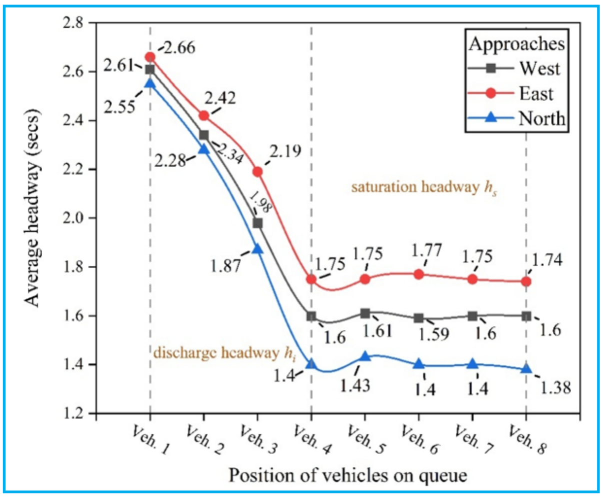

Figure 8. Average discharge headway and saturation headway

few vehicles in the queue exhibit relatively high headways due to start-up delay, after which the values decline and stabilize at the saturation headway (hs). Flow stabilizes to a saturation headway beyond the third vehicle, following the contribution of the first three vehicles to start-up lost time. Table 3 summarizes the start-up lost time, clearance lost time, and total lost time measured from the three approaches. The aggregate lost time for the intersection was observed as 16.03 s per cycle, distributed across the West, East, and North approaches. The comparatively higher value on the North approach suggests a slower discharge response, which may be attributed to driver behavior and geometric conditions, specifically the approach grade or slope that influenced vehicle acceleration. Consequently, the lost time constitutes a critical parameter in traffic signal analysis as it directly affects the effective green time and subsequently the capacity of an intersection.

Table 3. Lost time observed from each approach

Approach	Start-up lost time (secs)	Clearance lost time (secs)	Total lost time (secs)
West approach	2.13	3.18	5.31
East approach	2.02	3.07	5.09
North approach	2.5	3.13	5.63
Total lost time (l_t)			16.03

Saturation Headway and Saturation Flow Rate

The base saturation flow rates presented in Table 4 are substantially higher than the 1900 pcu/hr/lane proposed but align with findings from other countries with comparable traffic conditions and driving behavior [19]. And by considering the number of lanes on each approach, the resulting flow rate values represent the theoretical maximum service rates of each approach under ideal conditions. Notably, the west approach demonstrates the highest service potential, followed closely by the east, while the north yields the lowest overall service rate due to its fewer lanes. This suggests that the west approach may require proportionally longer green times to optimize throughput, and that the north approach may still act as a bottleneck under heavy demand due to the restricted number of lanes, thereby requiring balance in signal timing to minimize delay and maximize overall intersection performance.

Table 4. Flow rate and saturation flow rate

Approach	Saturation headway (secs/veh)	Base saturation flow rate (pcu/hr/lane)	Number of lanes	Saturation flow rate S (pcu/hr)
West	1.6	2250	3	6750
East	1.75	2057	3	6279
North	1.5	2400	2	4800

Flow Ratio and Critical Flow Ratio

The computed flow ratios and critical flow ratio are presented in Table 5. The critical flow ratio of 0.736 is less than one, implying that the overall demand is within the available capacity and that a feasible signal design can be implemented with queues expected to clear in each cycle. The value being less than one also suggests that the intersection is operating below saturation, and 74% of the usable green time will be consumed by demand. The remaining 26% green time can be reserved for near full capacity conditions during increases in demand that could lead to spillback and persistent queuing. Overall, the critical flow ratio shows that the webster's formulae can be reliably applied in the signal timing.

Table 5. Flow ratio and critical flow ratio

Phases	Lane group	Peak flow rate v (pcu/hr)	Saturation flow rate S (pcu/hr)	Flow ratio f_r	Critical lane flow ratio y_i	Critical flow ratio y_n
Phase 1 (West approach)	A	1834.8	6750	0.272	0.272	
	B	658.1	6750	0.097		
	C	343.1	6750	0.051		
Phase 2 (East approach)	D	598.7	6279	0.095		
	E	1552.1	6279	0.247	0.247	
	F	283.7	6279	0.045		0.736
Phase 3 (North approach)	G	609.2	4800	0.127		
	H	1040.4	4800	0.217	0.217	

Signal Timing

Using Webster's formula, the computed optimal cycle length for the intersection was 110 seconds, which aligns with prevailing urban traffic conditions and ensures efficient phase utilization. This cycle length provides a balance between minimizing delays and preventing excessive queue formation on any approach. The green times each phase in Table 6 reflect a proportional response to the traffic demands indicated by the critical flow ratios. This distribution ensures that the major traffic movements are prioritized, while still allowing adequate service to the minor approach. Regarding yellow interval timing, the north approach downgrade assumed to be 1% due to its gentle slope, was used to calculate the yellow interval for all phases as presented in Table 7. For safety reasons, the yellow interval was between 3 and 5 seconds [19]. The 4 seconds duration provides drivers with adequate time to perceive the signal change, decide, and safely stop or clear the intersection. This interval serves as a critical phase in traffic signal control, as it minimizes red-light violations and reduces the likelihood of rear-end collisions. Moreover, the red interval for phases 1, 2 and 3 were calculated as 71 secs, 74 secs, and 78 secs respectively, corresponding to the time each approach remains stopped while other approaches are served. Overall, the phase-by-phase distribution of one complete cycle of this efficient traffic control system is shown in Figure 9. The adopted signal plan demonstrates a well-balanced control system, achieving an equitable distribution of green times and minimizing both lost time and delay. The combined use of critical flow ratio analysis, yellow timing computation, and phase balancing guarantees efficient throughput and safety at the three-leg intersection.

Table 6. Effective green timing for each phase

Phase	Critical lane flow ratio y_i	Critical flow ratio y_n	Cycle length C_0 (secs)	Total lost time l_t (secs)	Effective green time G_{ei} (secs)
Phase 1	0.272	0.736	110	16.03	35
Phase 2	0.247	0.736	110	16.03	32
Phase 3	0.217	0.736	110	16.03	28

Table 7. Yellow interval for each phase

Perception-reaction time t_r (sec)	Approach speed U = 50 (km/hr)	Deceleration rate a (m/s^2)	% of grade G	Gravitational constant g (m/s^2)	Yellow interval Y (secs)
1	13.89 m/s	3	(-1/100)	9.81	4

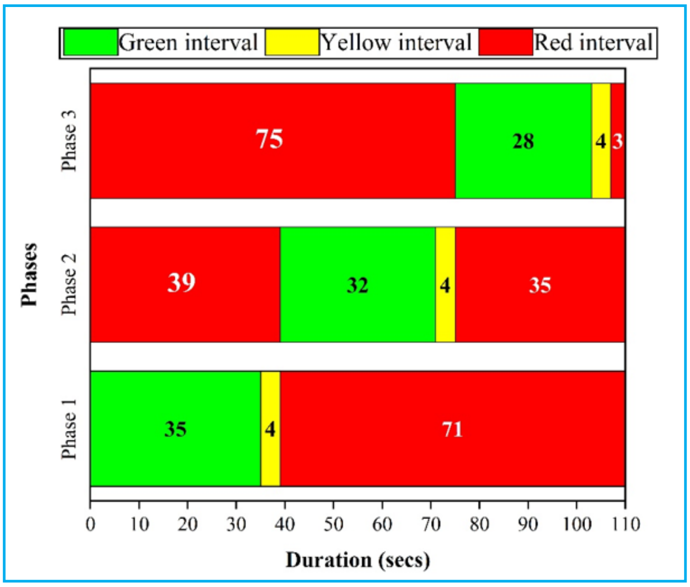

Figure 9. Sequential signal timing for a complete cycle

Performance and Efficiency

Volume to capacity ratio

Comparison of the peak flow rates and capacities in Table 8 indicates that all lanes operate in an undersaturated state, with demand below the available capacity. The provided capacity is sufficient to accommodate the peak flows at the intersection, thereby preventing saturation under the current signal timing. Moreover, the volume-to-capacity ratio for the critical lanes (A, E, H) was 0.85 each, which is less than one, implying that the flows remain within the available capacity of the signal timing. This also signifies stable traffic conditions where demand does not exceed capacity.

Level of service

According to Transportation Research Board [19], the control delay in Table 9 classifies the LOS of each approach to be category C. This LOS shows that traffic progression is favorable and the cycle length is moderate. At this LOS, a

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Table 8. Volume to capacity ratio analysis result

Phases	Lane group (critical lane*)	Peak flow rate v (Pcu/hr)	Saturation flow Rate S (Pcu/hr)	Effective green time G_{ei} (secs)	Cycle Length C_0 (secs)	Capacity c (Pcu/hr)	Volume to Capacity ratio v/c	Remarks
Phase 1 (West approach)	A*	1834.8	6750	35	110	2147.7	0.85	Ok (under capacity)
	B	658.1	6750	35	110	2147.7	0.31	
	C	343.1	6750	35	110	2147.7	0.16	
Phase 2 (East approach)	D	598.7	6279	32	110	1826.6	0.33	Ok (under capacity)
	E*	1552.1	6279	32	110	1826.6	0.85	
	F	283.7	6279	32	110	1826.6	0.16	
Phase 3 (North approach)	G	609.2	4800	28	110	1221.8	0.49	Ok (under capacity)
	H*	1040.4	4800	28	110	1221.8	0.85	

lot of vehicles will still drive past the intersection without halting, but vehicles may also need to stop. However, queues usually dissipate within a single cycle. The signal causes a tolerable increase in travel time for vehicles, reflecting moderate driver discomfort, controllable queue, and slight increases in fuel consumption brought on by stopping and accelerating. Moreover, approaches at this intersection operate at a better LOS C compared to LOS D, E, F reported by previous studies along this intersection and similar routes [5,7,9]. This validates the efficiency of the Webster-based signal design and possible improvements in the performance of the intersection when implemented in pretimed or vehicle actuated traffic control systems.

Table 9. Uniform delay and level of service for each lane group

Phases	Sum of peak flow rate Σv (Pcu/hr)	Sum of capacity C_A (Pcu/hr)	Group volume to capacity ratio X_A	G_{ei} (secs)	C_0 (secs)	Control delay d (secs/veh)	LOS
Phase 1 (A, B, C)	2836	6443.1	0.44	35	110	29.94	C
Phase 2 (D, E, F)	2434.5	5479.6	0.44	32	110	31.95	C
Phase 3 (G, H)	1040.4	2443.6	0.43	28	110	32.55	C

CONCLUSION

The design of an efficient traffic control signal for Araromi intersection provides a practical template for similar urban intersections in Nigeria. Implementing this signal timing through adaptive or pre-timed traffic control systems, could address persistent delays and enhance safety. key findings include:

1. Passenger cars constituted the highest share of the total traffic composition, which are dominated with taxis and private vehicles, followed by motorcycles, while tricycles are on the rise, with a negligible number of bicycles.

2. The peak period, occurring between 7:45 AM and 8:45 AM on a Monday market day, recorded a PCU of 6505 and a peak flow rate of 6920 pcu/hr, indicating a high concentration of vehicular movement associated with morning activities involving travelling towards the market, schools and offices.

3. The west approach contributed 41% of the total traffic, justifying its longer green interval, followed by the east (35%) and north (24%) approaches. The west approach also showed the highest saturation flow rate, followed closely by the east, while the north yields the lowest overall service rate due to its fewer lanes.

4. The average delay per cycle was 16.03 seconds, with the north approach experiencing slightly longer delays, suggesting a slower discharge response due to gradient, specifically the downgrade that influenced vehicle acceleration.

5. A critical flow ratio of 0.736 confirmed that the intersection operates within its capacity, validating the efficiency of the Webster-based signal design with manageable queues per cycle. Also, peak hour factor of 0.94 indicates a stable and consistent traffic stream, suggesting that a reliable and efficient signal control system can be effectively implemented at the intersection

6. The 110-second cycle length, distributed with 35s, 32s, and 28s of green times across phase 1, 2, and 3, respectively, along with a standardized 4-second yellow interval and proportional red intervals, ensured both operational efficiency and safety.

7. The efficiency of the traffic signal improved to Level of Service (LOS) C on each approach, reflected by control delays of 29.94, 31.95, and 32.55 seconds per vehicle, along with a volume-to-capacity ratio below 1. This validates the efficiency of the Webster-based signal design and possible improvements in the performance of the intersection when implemented in pretimed or vehicle actuated traffic control systems.

8. This study was limited to two weeks peak-hour traffic data collected under fair weather conditions. Seasonal variations, off-peak conditions, and the effects of pedestrian movement were not incorporated, which may influence signal performance under different traffic scenarios. Future studies may consider incorporating pedestrian phase to the signal design.

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Conflicts of Interest

All authors declare that they have no conflict of interests.

Author Contributions

Reuben Ayodeji Bolaji: writing-original draft, writing- reviewing and editing, methodology, investigation, formal analysis, visualization. **Olumuyiwa Samson Aderinola:** conceptualization, project administration, supervision, writing- reviewing and editing. **Moses Ariyo Oladipo:** investigation, formal analysis, writing- original draft preparation. **Temitope Elizabeth Oguntelure:** data curation, writing- original draft preparation. **Michael Oluwagbenga Abanire:** Investigation, writing- original draft preparation.

Data Availability Statement

The data used to support the findings of this study are included within the article.

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