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PREDICTIVE MODELLING OF TRAFFIC FLOW IN AKURE, NIGERIA: UNSIGNALIZED INTERSECTIONS IN FOCUS

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Abstract: Unsignalized intersections namely two-way stop-controlled intersection (TWSC) and all-way stop-controlled intersection (AWSC) are widely used in Akure. Five intersections consisting of three Tee and two Cross that were critical to traffic flow in the study area were selected for study. Data on geometric features were collected using odometer, while traffic parameters were captured and metered using cine camera placed at vantage positions from the intersections during peak and off-peak periods on week days. Traffic flows at the intersections were expressed as functions of traffic characteristics and geometric features of the approaches; while the effect of distances of intersections before and after the intersections studied were also incorporated as correction factors in the models. The models were developed using multiple linear regression technique with the aid of SPSS software and validated with empirical data other than those used for model calibration. Adjusted R2 values of 0.881 and 0.882 were obtained for Tee and Cross intersections respectively for peak period, while 0.938 and 0.940 respectively were obtained for off-peak period. These indicate that the flow models are very robust in replicating the observed data. The predictive models have the potential to accurately estimate traffic flow at intersections in the study area and other cities of the world with similar traffic conditions.

Keywords: Intersections; geometric features; traffic; models

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INTRODUCTION

In urban road networks, intersections usually constitute major bottlenecks, due to conflicting interactions between traffic streams in different directions as illustrated in **Fig. 1**. Intersections are the most critical points from capacity, congestion and safety viewpoints for the operation of an urban road network. Studies on traffic characteristics at intersections have been focused more on signalized than unsignalized ones globally; the perception has been that research on unsignalized intersections is unnecessary, since most intersections are signalized. This is not so especially in developing countries of the world like Nigeria where unsignalized intersection are largely used; thus, unsignalized intersections play important roles in the control of traffic in road networks.

Un-signalized intersections are classified into three types namely two-way stop-controlled (TWSC), all-way stop-controlled (AWSC) and Rotary Intersection. Each of these intersections has rules guiding right-of-way.

In order to model traffic flow at intersections, an understanding of vehicular delay, headway and gap acceptance is invaluable. Delay is one of the principal parameters used as a measure of effectiveness to determine the level of service (LOS) at intersections. It is defined as the difference in travel time between when a vehicle is affected by the controlled intersection and when a vehicle is unaffected. Headways are the time differences between successive arrival instants of vehicles passing a reference point. At unsignalized intersections, time headway is one of the most important factors influencing merging and turning; while, gap acceptance is the minimum space required by which a vehicle in the secondary (minor) stream or road accept gap in the primary traffic stream or road for manouvering (Rodríguez, 2006).

Critical gap and follow-up time are the two main gap acceptance parameters. Critical gap is defined as the minimum time interval required for one minor-stream vehicle to enter the intersection, while follow-up time is the time span between two departing vehicles, under the condition of continuous queuing. Thus, a gap acceptance model can help describe how a driver judges whether to accept or reject the available space.

Several studies have been carried out on traffic flow and allied models; Tanner (1962) developed an equation for the minimum average delay by minor stream vehicles given as:

$$D_{min} = \frac{e^{qp\left(t_f - t_m\right)} t_c}{\left(1 - t_m q_p\right) q_c} - \frac{q_p t_m^{\ 2}\left(2t_m q_p - 1\right)}{2\left(1 - t_m q_p\right)} \qquad (1)$$

where: q_p = major stream volume in vehicles/sec; t_c = critical gap time; t_f = follow-up time; and t_m = minimum critical gap.

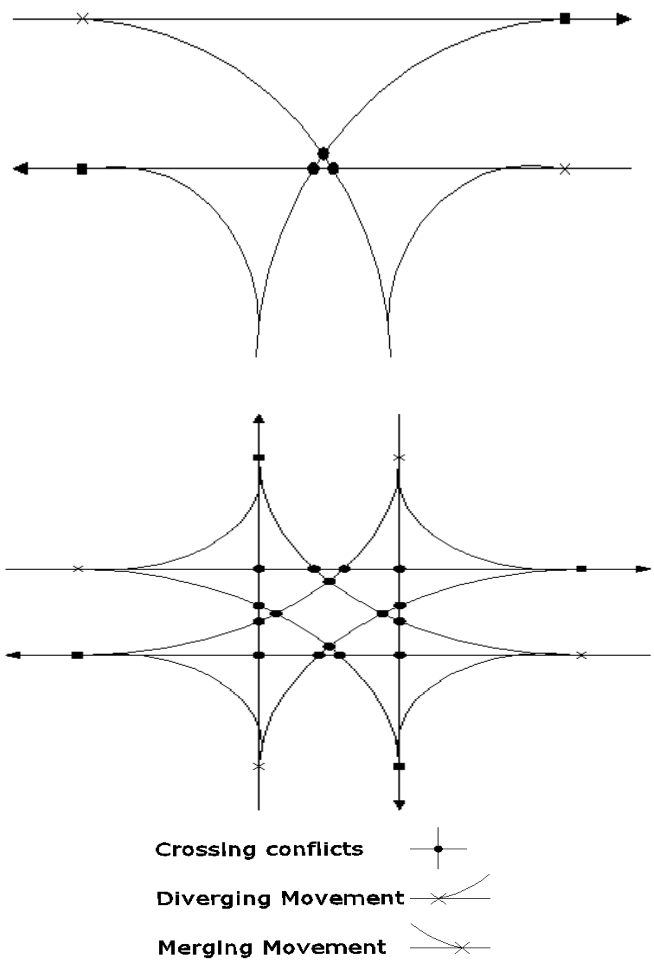

Fig. 1 Types of Movement at Intersections.

Kimber (1980) also developed a general relationship used for the maximum entry flow, Q_e, given as:

$$Q_e = k\left(F - f_c Q_c\right) \qquad (2)$$

where Q_c is the circulating flow in pcu/h. F is the intercept, f_c the slope were functions of the effective entry width with the slope also a function of the roundabout central island size parameter. A correction factor was used for the lesser order terms; viz entry radius and the angle of entry.

Owolabi and Adebisi (1993) also developed models for headway data for single lane-traffic flows at some points along Zaria-Sokoto Road, Nigeria. The study was particularly designed to reflect the traffic situation in developing countries like Nigeria where motorcycles constitute a reasonable proportion of traffic on urban roads. From the Kolmogorow-Smirnow (K-S) goodness of fit test results, the composite exponential model was found to be a sound descriptor of observed headways for flows ranging from 170vph to 750vph; while the shifted negative exponential model was found to be a sound headway model only for low flows. The approximate relationship for cases where motorcycles were included in the observations was given as:

$$a = 1.07 \times 10^{-3} q - 0.06 \quad (3)$$

where: a is the proportion of free vehicles; q is traffic flow expressed in vehicles per hour, while that for data not involving motorcycles was given as:

$$a = 4.510^{-4} q + 0.13 \quad (4)$$

Abdelwahab *et al.* (1994) developed models that can be used to predict the number of crossing opportunities in a traffic stream under various roadway and traffic conditions through an empirical study of vehicular headways in urban areas. Number of lanes, directionality (i.e. one-way or two-way road), location of study area within a signalized and coordinated corridor and other traffic as well as road features were considered. The general form of the model relating the number of crossing opportunities to the traffic flow rate is given as:

$$N = \alpha 0 + \sum_{i=1}^{k} \alpha_i v^1 + \in \quad i = 1,2,...,k \quad (5)$$

where: N is the number of crossing opportunities; v is traffic flow rate; α_0 and α_I are regression coefficients; k is the degree of the polynomial; ε is the random error term.

The Highway Capacity Manual (HCM) 2000 gives an equation for estimating entry capacity of a single-lane roundabout based on the conflicting flow, the critical headway and the follow-up time. The equation is given as:

$$C_{ex} = \frac{V_{cx} e^{-V_{cx}{}^{t_c}/3600}}{1 - e^{-V_{cx}{}^{t_f}/3600}} \quad i = 1,2,...,k \quad (6)$$

where: $C_{e,x}$ = entry capacity for the entry x in pcu/h; $v_{c,x}$ = conflicting flow in front of entry x in pcu/h; t_c = critical headway in sec; and t_f = follow-up time in sec.

The capacity model given above was based on data collected at roundabouts with single circulating lane; however, it has two parameters for calibration: the critical headway, t_c, and the follow-up time, t_f. The HCM 2000 also gives the capacity of the critical lane of a multilane roundabout entry as follows:

$$C_{crit} = 1230 e^{(-0.0009 V_c)} \quad i = 1,2,...,k \quad (7)$$

where C_{crit} is capacity of the critical lane on the approach, veh/h; and V_c is Conflicting flow, veh/h.

The capacity of the non-critical lane is assumed to be the same as that of the critical lane. The coefficient preceding the exponential term is equivalent to the follow-up time and can be readily measured in the field. However, a conspicuous gap left by past researchers is the outright neglect of the distance of intersection before and after the one under study in determining flow; this now forms a major thrust of this research. Thus, this research aimed at formulating predictive models of traffic flow at road intersections while incorporating the effect of the intersections before and after the ones studied as correction factors.

Akure, the study site is the capital city of Ondo State with a population of 387 087 according to 2006 census and is one of the fastest growing urban settlements in the South Western region of Nigeria. It is located on latitude 70° 20′ N and longitude 50° 15′ E. The existing land use is characterized by a medium density structure within the inner core areas. Akure is composed mainly of residential areas forming over 90% of the developed area but additional activities such as warehousing; manufacturing, workshops and other commercial activities are commonly located within the residential neighborhoods.

Over the years, the number of vehicles on its roads has increased greatly due to increasing socioeconomic activities. Increase in infra-structural facilities such as housing, electricity, water supply and transportation caused migration into the cities has imposed serious strains on existing transport infrastructure and brought about various traffic problems. The natural pattern of development is linear along its main roads; Oyemekun-Oba Adesida road and Arakale-Oda road. These roads connect other street roads from Aiyedun, Isolo, Araromi, Oke-Ijebu, Elerinla, Fanibi, Isikan and Adegbola residential áreas (**Fig. 3**).

In Akure metropolis, unsignalized intersections are the most common forms of intersection where it is controlled by Stop and Yield signs. The traffic composition in the metropolis is mixed comprising of motorcycles, taxis, minibuses, Lorries and trucks (trailers); however, the traffic composition of Akure is dominated by taxis, motorcycles (Okadas) and minibuses (Owolabi, 2009). **Figure 2** is the map of Ondo State in relation to Nigeria and that of Ondo state indicating the study area.

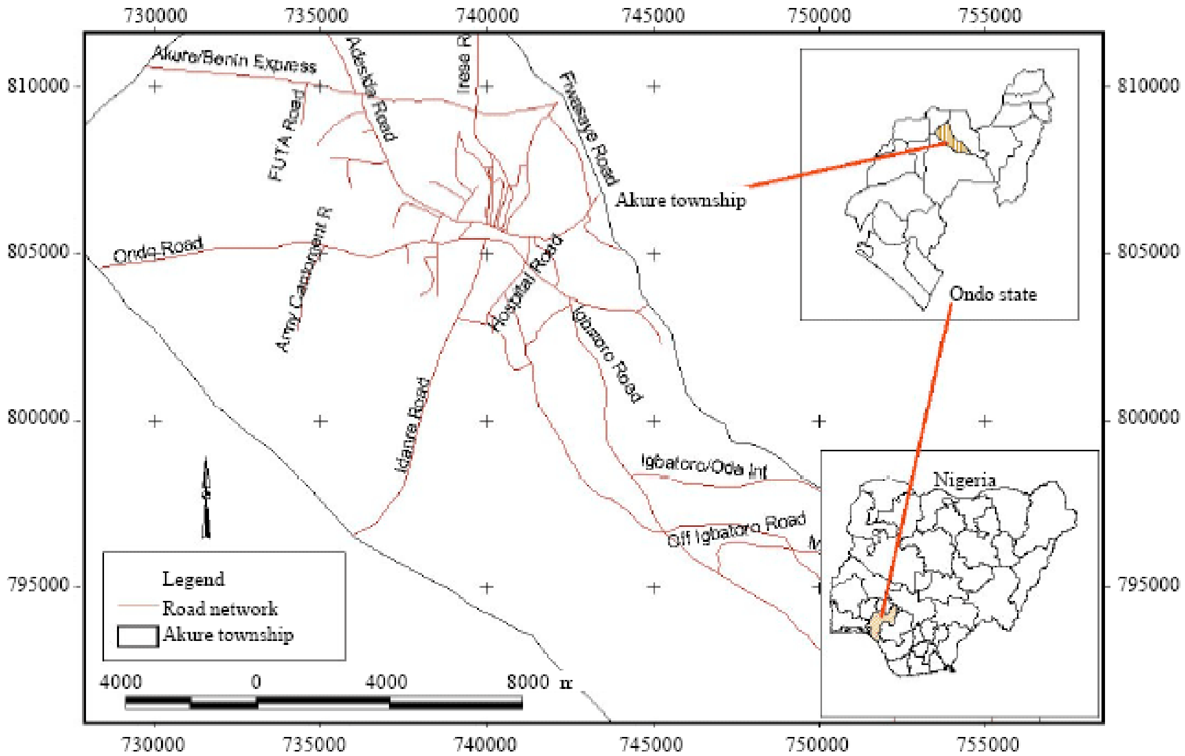

Fig. 2 Map of Ondo State in Relation to Nigeria and Map of Ondo State Indicating the Study Area.
Source: Ayeni (2011)

METHODOLOGY

Five intersections shown in **Fig. 3** consisting of three Tee intersections Road block (RN1), Cathedral (RN2), Akure-Town Hall -Araromi Junction (RN3) and two Cross intersections NEPA (RN4**),** Odole (RN5)) that are critical to traffic flow in the study area were selected for study. Data on geometric features of the intersections were collected using odometer, while traffic parameters such as volume (q), speed (V_s), density (K), headway (h) and delay (d_a) were captured and metered using cine camera placed at vantage points from the road sections during the morning and evening peak periods between 6:30−7:30 GMT and 15:30−16:30 GMT, respectively, and off-peak periods between 10:30−11:30 GMT during week days. The headways were measured while replaying the cine camera and observing the interval in time from head to head of vehicles as they passed a given point at the intersections' approaches. Control delays were measured by taking note of how long vehicles waited at particular approaches before having the right-of-way. Traffic flows at intersections were expressed as functions of traffic characteristics and geometric features of the roads; while distances of intersections before and after the intersections studied were also incorporated as a correction factors in the models. The models were developed using multiple linear regression technique with the aid of SPSS software and validated with empirical data other than those used for model calibration The descriptive statistics of traffic data for rotary and tee intersections studied during peak and off peak periods are shown in **Table 1** to **Table 4.**

RESULTS AND DISCUSSION

The Predictive Model

The traffic flow q was modelled as a function of traffic characteristics and geometric features of intersections. The geometric features include major approach width (m_{sw}) in metres; number of lanes of minor road movement (m_n); and minor road approach width (m_w) in metres. The traffic characteristics include average delay (d_a) in sec; follow up time (t_f) in sec, density (k) vehicles/km, headway (h) in sec, and vehicle speed (v_s) in m/s.

Table 1. Descriptive Statistics of Traffic Data for Tee Intersection during Peak Periods

Traffic Characteristic	Mean	Median	Mode	Kurtosis	Skewness	Standard Deviation	Variance	Coefficient of Variance
Q	1176.29	1292	1384	-1.15	-0.47	346.48	120049.77	0.29
V_s	6.789	6.72	6.95	-0.31	0.25	0.85	0.72	0.12
K	11.56	11.5	12	-0.26	0.03	2.05	4.19	0.18
D_a	53.11	54	54	1.35	0.47	11.38	129.55	0.21
H	3.42	3.32	3.09	0.20	0.56	0.78	0.60	0.23

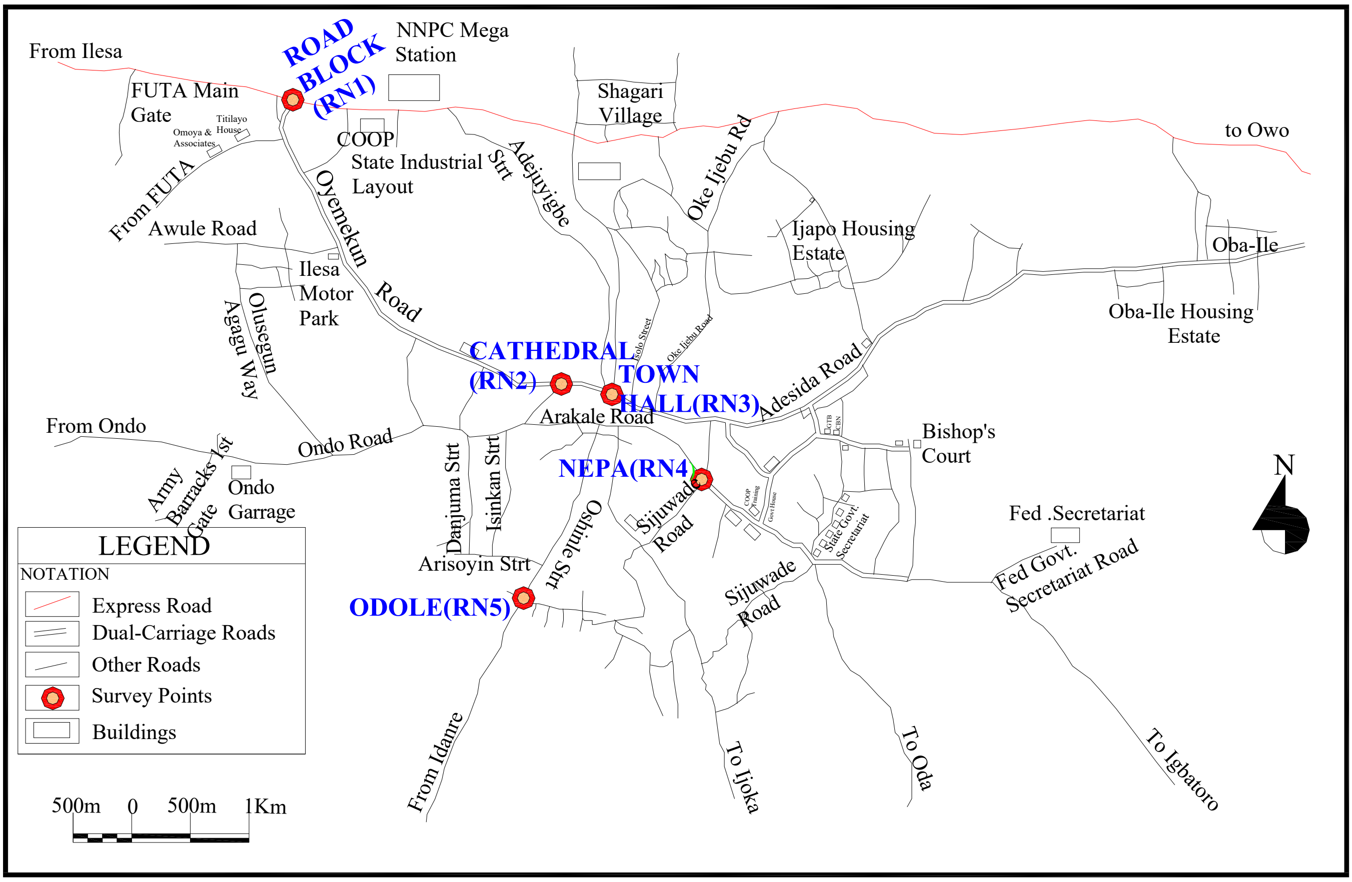

Fig. 3 Street Guide Map of Akure Showing the Survey Points.
Source: Ministry of Lands and Housing (2010)

Table 2. Descriptive Statistics of Traffic Data for Tee Intersection during Off-Peak Periods

Traffic Characteristic	Mean	Median	Mode	Kurtosis	Skewness	Standard Deviation	Variance	Coefficient of Variance
Q	1127.89	1302	1300	-1.44	-0.54	375.85	141262.16	0.33
V_s	7.06	7.18	6.15	-0.57	-0.68	0.93	0.86	0.13
K	10.56	11	11	-0.55	-0.52	2.44	5.97	0.23
D_a	44.92	41.5	32	-0.69	0.60	10.63	112.94	0.24
H	3.80	3.34	3.13	0.56	0.93	1.14	1.29	0.30

Table 3: Descriptive Statistics of Traffic Data for Cross Intersection during Peak Periods

Traffic Characteristic	Mean	Median	Mode	Kurtosis	Skewness	Standard Deviation	Variance	Coefficient of Variance
Q	820.04	800	980	-0.22	0.40	124.03	15383.317	0.15
V_s	5.45	5.3	6.38	-0.95	0.20	0.91	0.83	0.17
K	9.86	10	9	0.63	0.69	1.53	2.35	0.16
D_a	48.95	47	46	-0.65	0.42	5.77	33.31	0.12
H	3.67	3.78	3.79	1.79	0.00	0.99	0.98	0.27

Table 4. Descriptive Statistics of Traffic Data for Cross Intersection during Off-Peak Periods

Traffic Characteristic	Mean	Median	Mode	Kurtosis	Skewness	Standard Deviation	Variance	Coefficient of Variance
Q	783.66	760	720	0.69	1.03	95.84	9185.97	0.12
V_s	6.37	6.38	6.53	0.11	-0.20	0.82	0.66	0.13
K	7.91	8	8	-0.69	-0.02	1.85	3.43	0.23
D_a	38.91	38	38	-0.86	0.15	4.96	24.60	0.13
H	3.42	3.54	3.54	-0.95	0.05	1.16	1.34	0.34

Table 5. Summary of Coefficients for Tee Intersection for Peak Period

Model		Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.	95% Confidence Interval for B Lower Bound	95% Confidence Interval for B Upper Bound	Correlations Zero-order	Correlations Partial	Correlations Part
1	(Constant)	3.469	.015		186.400	.000	2.837	2.898			
	H	-.939	.029	-.912	-32.364	.000	-.996	-.882	-.912	-.912	-.912
2	(Constant)	3.169	.038		66.988	.000	2.491	2.642			
	H	-.866	.027	-.841	-32.500	.000	-.919	-.814	-.912	-.913	-.795
	K	.259	.031	.217	8.371	.000	.198	.320	.490	.498	.205
3	(Constant)	2.975	.059		40.170	.000	2.257	2.490			
	H	-.839	.026	-.815	-31.739	.000	-.892	-.787	-.912	-.909	-.748
	K	.338	.030	.233	9.228	.000	.219	.338	.490	.536	.218
	Vs	.197	.047	.102	4.195	.000	.104	.290	.251	.277	.099

Dependent Variable: *Q*

The traffic flow at tee and cross intersection is given as $q = f\left(v_s, k, d_a, m_{sw}, m_n, m_w\right)$, but q is inversely proportional to h, this gives:

$$q = \frac{v_s.k.d_a.m_{sw}, m_n, m_w}{h} \tag{8}$$

where q is the traffic volume in pcu/hr. Taking the log to base 10 on both sides gives: $log\, q = log\left(\frac{v_s.k.d_a.m_{sw}, m_n, m_w}{h}\right) = log\left(v_s.k.d_a\right) - log\, h$

$\left(m_{sw}.m_n.m_w\right) + log_q = log\, v_s + log_k + log\, d_a + \left(-log\, h\right)$

Let *logq* be denoted by *Q*, *logv*$_s$ by *V*, logk by *K*, logd$_a$ by D_a,, and –log*h* by *H*. Using multiple linear regressions given as:

$$Q = a_0 + a_1 V + a_2 K + a_3 D_a + a_4 H \tag{9}$$

where a_0 is the regression constant, a_1, a_2, a$_3$, and a_4 are the regression coefficients. Note: m_{sw}, m_n, m_w are constant values. Hence they fizzle into the constant term a$_0$ in the regression equation. The summary of the coefficient at intersections obtained using regression analysis are shown in **Tables 5– 8**. The model equations are given in **Eqs 10–13**. For Tee Intersection during Peak Period: Substituting the coefficients in **Table 5** in the model **Eq. (9)** above gives: $Q = 2.975 + 0.197V + 0.338K - 0.839H$. Substituting for *Q*, *V*, *K*, and *H*: $log\, q = log\, 10^{2.975} + log\, v_s^{0.197} + log_k^{0.338} + log\, h^{-0.839}$

$log\, q = log\left(10^{2.975} v_s^{0.197}\, {}_k^{0.338} h^{-0.839}\right)$

The model equation for Tee Peak Period is given as:

$$q = 944.06 v_s^{0.197} k^{0.338} h^{-0.839} \tag{10}$$

Equation (10) shows that vehicular speed (v_s), density (k) and headway (h) have significant impact in predicting the traffic flow at Tee intersections during the peak periods at the study locations. Similarly, the coefficients obtained from the statistical analysis were substituted to obtain the model equations for Tee intersections during off-peak periods and Cross intersection during the peak and off-peak periods.

Table 6. Summary of Coefficients for Tee Intersection for Off-Peak Period

Model		Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.	95% Confidence Interval for B Lower Bound	95% Confidence Interval for B Upper Bound	Correlations Zero-order	Correlations Partial	Correlations Part
1	(Constant)	3.489	.014		199.435	.000	2.860	2.918			
	H	-.970	.026	-.964	-37.352	.000	-1.026	-.922	-.964	-.964	-.964
2	(Constant)	3.355	.044		63.088	.000	2.669	2.842			
	H	-.928	.028	-.922	-32.865	.000	-.987	-.875	-.964	-.955	-.813
	K	.114	.035	.090	3.223	.002	.044	.183	.525	.301	.080
3	(Constant)	3.220	.075		34.886	.000	2.469	2.766			
	H	-.807	.030	-.897	-30.278	.000	-.966	-.847	-.964	-.948	-.735
	K	.173	.037	.114	3.872	.000	.070	.216	.525	.356	.094
	Vs	.115	.051	.060	2.246	.027	.014	.218	.239	.216	.055

Dependent Variable: *Q*

The model equation for Tee Intersection Off-Peak Period is given as:

$$q = 1659.59 v_s^{0.115} k^{0.173} h^{-0.807} \quad (11)$$

Equation (11) indicates that variables “v_s”, “k” and “h” have significant contribution in predicting traffic flow.

The model equation for Cross Intersection is given as:

$$q = 814.70 k^{0.122} d_a^{0.232} h^{-0.965} \quad (12)$$

For the Cross intersection, the predictor variables *k*, d_a and *h* made significant impact in predicting the traffic flow. On the contrary, speed (v_s) has no impact in predicting the flow; this may be due to large volume of motorcycle which impedes smooth flow of traffic.

Table 7. Summary of Coefficients for Cross Intersection for Peak Period

Model		Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.	95% Confidence Interval for B Lower Bound	95% Confidence Interval for B Upper Bound	Correlations Zero-order	Correlations Partial	Correlations Part
1	(Constant)	3.410	.018		160.384	.000	2.773	2.842			
	H	-.983	.029	-.924	-33.339	.000	-1.041	-.925	-.924	-.924	-.924
2	(Constant)	3.007	.072		33.217	.000	2.262	2.548			
	H	-1.020	.028	-.959	-36.350	.000	-1.075	-.965	-.924	-.935	-.933
	Da	.252	.044	.151	5.708	.000	.165	.340	-.070	.383	.147
3	(Constant)	2.911	.074		31.094	.000	2.163	2.456			
	H	-.965	.031	-.907	-31.471	.000	-1.026	-.905	-.924	-.917	-.780
	Da	.232	.044	.131	5.045	.000	.134	.306	-.070	.345	.125
	K	.122	.032	.107	3.816	.000	.059	.186	.514	.268	.095

Dependent Variable: *Q*

Table 8. Summary of Coefficients for Cross Intersection for Off-Peak Period

Model		Unstandardized Coefficients B	Unstandardized Coefficients Std. Error	Standardized Coefficients Beta	t	Sig.	95% Confidence Interval for B Lower Bound	95% Confidence Interval for B Upper Bound	Correlations Zero-order	Correlations Partial	Correlations Part
1	(Constant)	3.500	.024		119.250	.000	2.849	2.946			
	H	-1.142	.039	-.948	-28.938	.000	-1.221	-1.064	-.948	-.948	-.948
2	(Constant)	3.140	.063		40.128	.000	2.412	2.664			
	H	-1.072	.036	-.890	-30.118	.000	-1.143	-1.002	-.948	-.952	-.842
	Da	.202	.034	.178	6.017	.000	.135	.269	.468	.529	.168
3	(Constant)	2.896	.076		30.294	.000	2.143	2.444			
	H	-1.096	.032	-.893	-33.692	.000	-1.139	-1.012	-.948	-.962	-.844
	Da	.261	.032	.230	8.043	.000	.197	.326	.468	.643	.202
	Vs	.193	.040	.133	4.871	.000	.115	.272	-.089	.453	.122

Dependent Variable: *Q*

The model equation for Cross Intersection Off-Peak Period is given as:

$$q = 787.058 k^{0.193} d_a^{0.261} h^{-1.096} \quad (13)$$

For the model **Eq. (13)**, the standardized beta-value in **Table 8** also indicates that “v_s” and “d_a” have positive relationship between the predictors and the outcome variable; while predictor variable h has inverse relationship between the variable and the outcome variable. **Table 9** shows the summary of the model parameters from statistical analysis output.

Effect of Distance of Next Intersection

The distance of next intersection plays a crucial role in determining flow at a particular intersection; this effect comes into play for intersections linking the same roads. Drivers tend not to afford to miss an intersection if the next one is far away and his destination is before the next intersection. On the other hand, drivers tend to

Table 9. Summary of the model parameters for peak period

Intersection	R	R^2	Adj R^2	SEE	Source	SS	df	MS	F	Sig.
Tee Peak Period	0.940	0.883	0.881	0.07602	Regression	9.185	3	3.062	529.623	0.000
					Residual Error	1.220	211	0.006		
					Total	10.404	214			
Tee Off-peak period	0.969	0.939	0.938	0.05714	Regression	5.207	3	1.736	531.625	0.000
					Residual Error	0.337	103	0.003		
					Total	5.543	106			
Cross Peak Period	0.940	0.884	0.882	0.04556	Regression	2.972	3	0.991	479.590	0.000
					Residual Error	0.388	188	0.002		
					Total	3.360	191			
Cross Off-peak period	0.971	0.942	0.940	.03209	Regression	1.546	3	0.515	500.370	0.000
					Residual Error	0.095	92	0.001		
					Total	1.640	95			

Table 10. Intersection under study

Intersection Under Study	Major Roads Linked	Intersections Before and After
Cathedral(RN2)	Oba-Adesida and Arakale roads	[a]By-pass and [b]Car street intersections
Town Hall(RN3)	Oba-Adesida road and Ilesha-Owo Express way	[b]Odo-Ijoka and [b]Odo-Ikoyi intersections
Road Block(RN1)	Oyemekun road and Ilesha-Owo Express way	[a]OkeIyanu Junction
Odole(RN5)	Iromu/Adebowale streets and Oke Aro	[b]Alafe Junction
Nepa(RN4)	Oba-Adesida/Alagbaka and Arakale/Oda road	[a]Government house Junction and [a]Alafiatayo Junction

Note: a >300m; b <300m

prefer major intersections to other minor ones especially if they are at short distances apart. Generally, the longer the spacing between the intersections, the less will be the interference to through traffic and the higher will be the speeds on the arterial. However, longer spacing brings about longer travel distances for side road traffic entering or leaving the arterial and also increases the volume of side road traffic concentrated at each intersection. While studying the effect of distance of next intersection on flow at intersections in the study area, intersections linking the same major roads were considered as shown in **Table 10**.

To generate correction factors for the general flow equations developed for Tee and Cross intersections, the following steps are taken:

(a) Intersections before and after those under study were grouped into two categories, those within 300m and those beyond 300m;
(b) The factors were obtained by determining the ratio of predicted and observed flows and finding the average for all intersections whose next intersections fall into the same distance category;
(c) Two sets of values were obtained: one to cater for intersections before those under consideration and the other to cater for intersections after. The distance factors were then determined by taking the averages of those two set of values.

The analysis was carried out for both peak and off-peak periods. The computed distance factors are shown in **Table 11**.

The flow models which incorporate the effect of distance of next intersection on flow at Tee and Cross intersections are given by **Eqs** (**14)** and **(15)** for the peak period, while **Eqs** (**16)** and **(17)** are the off-peak period flow models.

$$q = 944.06\, f_d {v_s}^{0.197} k^{0.338} h^{-0.839} \tag{14}$$

$$q = 814.70\, f_d k^{0.122} {d_a}^{0.232} h^{-0.965} \tag{15}$$

$$q = 1659.59\, f_d {v_s}^{0.115} k^{0.173} h^{-0.807} \tag{16}$$

$$q = 787.058\, f_d {v_s}^{0.193} {d_a}^{0.261} h^{-1.096} \tag{17}$$

In applying **Eqs 14–17**, the analyst would select the appropriate distance factor from **Table 11** based on the distance categories in which those next intersections fall. From **Table 11**, it could be observed that when the distance of a major intersection to the next ones, both before and after it is less than 300m, the flow at that intersections with similar geometric characteristics, but which fall under a different distance category. This is shown by the distance factors 1.6 and 2.527 for peak and off peak periods respectively. Drivers see no point

Table 11. Distance factors (f_d)

Before / After	Peak Period		Off-Peak Period	
	< 300 m	> 300 m	< 300 m	> 300 m
< 300m	1.60	1.41	2.527	1.626
> 300m	1.30	1.11	1.626	0.725

in plying a minor intersection (which is often linked by a narrow road) when a major one is not far away. The effect is especially more pronounced during the off peak periods as drivers are not under pressure to avoid traffic congestion. On the other hand, when the distance of a major intersection to the next ones before and after is greater than 300 m, flow at that intersection will be less than those at other intersections having similar geometric characteristics, but which fall under different distance categories. This is shown by distance factors of 1.11 and 0.725 for peak and off peak periods respectively. Drivers see no point travelling longer distances before linking a major road and tend to make use of the nearest intersection, especially if their destination is not far away. The effect is less pronounced during peak periods because minor intersections are often served by narrow roads which increase the likelihood of congestion and drivers tend to avoid them at the expense of longer travel distances.

CONCLUSION

The developed models in this research provided insight into the combined effect of speed, density, headway and delay as well as the roadway geometric characteristics on traffic flow at Tee, Cross and Rotary intersections. They also shed more light on the effect of distance of other intersections on flow at an intersection of interest; the models have the potential to accurately predict traffic flow at intersections and will provide a rational basis for planning and design of effective control mechanisms at intersections in the study area and in other developing cities with similar traffic characteristics.

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