DEVELOPMENT OF MACROSCOPIC TRAFFIC FLOW MODELS FOR URBAN ROADS

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ABSTRACT

Availability of adequate mathematical models for vehicular traffic is a pre-requisite for the development of efficient traffic control strategies. Macroscopic traffic flow models, that is, speed-flow-density relationships, are the most useful tools in highway design and planning process. They are useful in predicting the roadway capacity, determining the adequate level-of-service of traffic flow and travel time for a given roadway. The challenge faced by urban traffic in both developed and developing countries is infrastructure deficiency. Consequently, vehicular traffic volume has rapidly outstripped the capacity of the nation's roadways. It has therefore become increasingly necessary to understand the dynamics of traffic flow and obtain mathematical description of the processes in order to address this problem; hence, this study.

Traffic flow data (traffic volume and travel speed) were collected along Post Office – Obafemi Awolowo Teaching Hospital Complex Road in Ile-Ife during the morning and evening peaks using the manual technique at an interval of 15-minutes for seven days. Traffic density was computed using the fundamental traffic flow equation. Speed-density model was thereafter developed using linear regression technique. Flow-speed and flow-density models were then developed from the speed-density model by applying the fundamental traffic flow relationship. Traffic parameters deduced from these models were then used to assess the performance of the road.

The results obtained showed that the traffic flow data collected was composed of 23 % passenger cars, 25 % buses, 2 % trucks and 50 % motorcycles. The speed-density, flow-speed and flow-density models were $U_s = -0.324K + 41.13$, $Q = -3.09{U_s}^2 + 126.94U_s$, and $Q = -0.324K^2 + 41.13K$, respectively. R^2 values of 0.611, 0.942, and 0.431 for speed-density model, flow-speed model, and flow density model, respectively. A free flow speed (U_s) of 41.16 km/h, optimum speed (U_o) of 20.57 km/h, jam density (K_j) of 126.94 pc/km/ln, optimum density (K_o) of 63.5 pc/km/ln, percentage free flow speed (PFFS) of 50.10% , level of service E, traffic capacity (C) of 1306 pc/h/ln (which was found less than the required traffic capacity of 1500 pc/hr/ln for an urban two-way two-lane highway), were obtained for this study.

In conclusion, this study provided a new means of determining the level of service and performance rating of two-lane highways compared to the method provided in the Highway Capacity Manual (HCM). Traffic parameters obtained from this study showed that the road is operating below the required capacity and at the level of service E. This denotes that the capacity of the highway has been reached. And that traffic flow conditions are best described as unstable with any traffic incident causing extensive queuing and even breakdown. Levels of comfort and convenience are very poor and travel speeds are low. Hence, timely effective traffic management plan for urban roads is recommended in order to withstand the increasing travel demand.

Keywords : models for vehicular traffic, macroscopic traffic flow models, efficient traffic control strategies, macroscopic traffic flow models macroscopic traffic flow models.

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xvi, 136p

CHAPTER ONE

INTRODUCTION

1.1 General Background

The challenges faced by urban traffic in both developed and developing countries are infrastructure deficiency, congestion, accidents, and pollution due to environmental and health damages. Though, all of these are important, the problem of traffic congestion is the most visible and affects a large number of motorists directly on a day-to-day basis. Urban areas in most of the developing countries are facing major challenges in traffic management and control in recent decades and Nigeria is no exception. Traffic congestion is a consequence of disparity between transportation demand and supply Highway Capacity manual (2010). Demand for transportation in urban areas is an increasing phenomenon due to continuous increase in urban population and economic activities, while the current rate of road infrastructure development is inefficient. As a result, traffic congestion has already become part of urban transportation system. Such traffic congestion not only causes problems to urban transportation activities but also causes degradation to natural environment by increasing the magnitude and intensity of air pollution. Efforts to eliminate traffic congestion completely from the transportation network may be unrealistic; rather the way to minimize traffic congestion and consequent air pollution is a challenge for transportation engineers and urban planners.

Worldwide urbanization is stimulating a new global economy and vice versa. This interactive process will literally change the face of the planet (Cohen, 2004). More than half of world's population is currently living in cities. This share is expected to grow to almost 70 percent in 2050. The total world population growth in the same period (2010-2050) is estimated to be 33 percent; therefore, the prognosis for the absolute urban growth is 86 percent over the next 40 years (United Nations, 2008). A significant share of the urban growth is taking place in large cities. Especially the number of conglomerates with more than 5 million inhabitants will grow. Middle and low income countries show the highest urban population increase, especially in Sub Saharan Africa (Gwilliam, 2003). Despite some economic benefits, the rapid urban growth in these developing countries is outstripping the capacity of most cities to provide adequate services for their citizens (Cohen, 2004). A high urbanization rate in combination with the intense desire for car ownership in developing countries causes a rapid growth of motorization (Gakenheimer, 1999). In other countries where public transport plays a dominant role, urban population growth goes hand in hand with a significant increase in public transport trips (World Bank, 2002). On the other hand, lack of infrastructure and weak maintenance put extra stress on these growing traffic flows with congestion, pollution and a low road safety level as a result (Rust *et al*., 2008, Cohen, 2004, Gakenheimer, 1999 and Gwilliam, 2003). Improved mobility in urban areas in developing countries is possible by building new infrastructure. However, this is a long term and expensive solution. A short term solution is to improve the traffic management to rationalize the use of existing infrastructure (Gakenheimer, 1999).

The current traffic control and management in developing cities is underdeveloped (Gwilliam, 2003). Hence, vehicular traffic volume has rapidly outstripped the capacities of the nation's highways and the problem of limited road capacity has been growing, it has become increasingly necessary to understand the dynamics of traffic flow and obtain mathematical description of the processes (Otuoze *et al*., 2012).

Availability of adequate macroscopic mathematical models of traffic is a pre- requisite for the development of efficient control strategies (Papageorgiou, 1983). For as the number of vehicles grow and the need for mobility increases on a world-wide scale, the frequency and duration of traffic jams in and around major cities and on major freeways increase (Berg and Vaa 2003). It is therefore, acknowledged that to reduce congestion and improve urban mobility in large cities, better utilization of the existing infrastructure should be encouraged. The construction of the new infrastructure is not an easy solution to decrease congestion, not only because of the tremendous cost to keep pace with population increase and the resulting increase in travel demand, but also because of the phenomenon of induced demand. In the light of high costs and societal impacts, the effective way to relieve congestion and improve accessibility is to carry out dynamic traffic management (DTM), make better use of existing road capacity and improve the production efficiency of larger transportation networks. Previous study revealed that the network-wide DTM will be even more efficient than local traffic control in improving network utilization (Taale, 2008).

The precondition for efficiently implementing network-scale DTM is a timely and accurate description of the state of the network, that is, a macroscopic description of the network traffic flow. The macroscopic description of traffic operation utilized in this study are the Macroscopic Traffic Flow Models (MTFM) as proposed by Greenshields (1935), Obeid (2001), Garber and Sankar (2003) etc. The MTFM describe the network-average relationships between the number of vehicles in the network and performance, and between the number of vehicles in the network and outflow (Jiyang *et al*., 2010). When the MTFM is known, transportation managers can continuously monitor whether the road network is in the ideal and preferred state.

1.2 Statement of Research Problem:

Urban transport problems remain one of the most nagging problems in urban transportations today. All over the world, attempts have been made to tackle the problems, yet the situation seems to get worse. According to Ogunsanya (1988), Nigeria had a rate of urbanization that was ranked as one of the highest in the world. Available statistics, as at that time, indicated that by 1988, more than 30% of the country's population lived in urban centers of 100,000 and above. It was also estimated that by the year 2000, the proportion of population living in cities would have increased to 50% from its 1990 rate of 35%. This shows that a major feature of Nigeria population structure is an increasing tendency for people to concentrate in the urban area. This is because the major area of settlement change and development is the city. Since most of the urban areas remained the major points of administrative, commercial, educational, social and recreational centers of the country (as seen in Ile-Ife). The multi-functions performed by these cities make them to generate and attract large number of intra and inter-urban road traffic (Ogunbodede, 2008). Thus, travel demand has been on the increase while the rate of road infrastructure development is insufficient. The need to optimize the existing road infrastructure is therefore highly expedient, considering the sociopolitical and economic factors which limit the construction of new road infrastructure; hence, this study.

1.3 Justification

Nigeria is one of the countries in the developing world with rapid urbanization and fast growing cities. A study of the changing morphology of many Nigerian cities gives an insight into the evolution of urban transport problems in Nigeria. There has not been any comprehensive transportation study for many urban centers in Nigeria (Aderamo, 2012). Thus the volumes of traffic along many of the urban routes in our cities are not known. A time-series data on the various components of urban traffic peculiar to Nigerian socioeconomic and demographic environment is of great importance to city planners interested in future transport planning. Traffic flows along major roads in our cities need to be monitored regularly so that the design capacities of those roads are not exceeded, hence; this study.

1.4 Aim and Objectives of the Research

The aim of this study is to develop macroscopic traffic flow models that could be used to continuously monitor the state of traffic on two way two lanes urban roads in Nigeria; a case study of a selected major urban road in Ile-Ife.

The specific objectives of this research are to

(a) collect traffic flow data for selected urban road in Ile-Ife;

(b) develop macroscopic traffic flow models for the selected road; and

(c) evaluate the developed models.

1.5 Contribution to Knowledge

This study will provide macroscopic traffic information suitable for predicting the performance of two way two lane urban roads systems in Nigeria.

1.6 Scope of the Research

This study involves the collection, analysis and macroscopic modelling of traffic flow parameters along Post-Office / Teaching Hospital road, located at the central part of Ile-Ife City.

CHAPTER TWO

LITERATURE REVIEW

2.1 General Review

Traffic flow models can generally be grouped as microscopic (e.g, car following model) and macroscopic models. The microscopic traffic flow models simulate single vehicle-driver units and analyze microscopic properties like the position and velocity of each individual vehicle. In contrast, macroscopic models consider the traffic stream from a macroscopic perspective and can be grouped as continuum and non-continuum models. The most popular continuum models are the hydrodynamic models, the Lighthill-Whitham-Richards (LWR) models (Vasantha *et al*., 2011). The fundamental relationship between flow and density was first proposed by Greenshields (1935). Traffic phenomena have been modelled as flow from the analogy between vehicular traffic and flow which is continuum (Markos, 1991 and Amadou *et al*., 2010). Macroscopic flow variables, such as flow, density, speed and speed variance, reflect the average state of the traffic flow in contrast to the microscopic traffic flow variables, which focus on individual drivers. Macroscopic modelling looks at traffic flow from a global perspective, whereas microscopic modelling gives attention to the details of traffic flow and the interactions taking place within it (Mathew, 2006). The modelling approaches have provided useful tools to understand traffic phenomena such as the formation and dissipation of traffic jams (Hwasoo, 2008). The major advantage of macroscopic models is their tractable mathematical structure and their low number of parameters (Gunnar, 2010).

The idea of a macroscopic description of the traffic flow is quite old. Many traffic researchers (such as Greenshields (1935), Greenberg (1959), Underwood (1961), Drew (1965) and Drake et al. (1965), Thomson (1967), Wardrop (1968), Zahavi (1972), Herman and Prigogine (1979), Olszewski *et al.* (1995), Lum *et al.* (1998), Roux and Bester (2002), Daganzo (2005), Niesen and Jorgensen (2008), Alkubaisi and Abbas (2011), Xie *et al.* (2011), Janvier (2013), etc.) have investigated and estimated these three variable relations. Some of these researchers agreed with Greenshields (1935), who proposed that speed and density are linearly related. However, other researchers postulated that speed and density are not always linearly related. Many macroscopic traffic flow models have been developed in the past few decades on the basis of empirical data collected in the United States, Canada, Turkey, China, Japan, etc. However, research carried out by Roux and Bester (2002) on South African freeways discovered that models obtained from overseas studies are in most cases not readily applicable to African roadways due to differences in population, transportation infrastructure and driving culture. Greenshield s (1935) proposed a parabolic function for the flow- density relation by suggesting linear speed-density curves under the free-flow condition. Based on notions like fluid dynamics and car-following decisions, varieties of functions forms for the flow-density relationships have been studied. For example, Greenberg (1959) used a logarithmic form for speed-density relations, and Underwood (1961) proposed an exponential form to show the relations. Drake *et al.* (1967) extended the specification to the bell-shaped curve model, which corresponds to a speed-flow specification associated with the normal distribution curve. Thomson (1967) used data from central London to obtain a linear speed-flow model. Wardrop (1968) developed a similar relation between average speed and flow by directly incorporating average street width and average signal space into his model. Zahavi (1972) enriched Wardrop's by analyzing real traffic data collected in various cities in the England and the United States. But all of the described models cannot be used to describe the rush hour in a congested city. Later, Herman and Prigogine (1979) put forward the two-fluid model which establishes macroscopic relations in vehicular traffic in large cities. They assumed that the speed distribution splits into two parts: moving vehicles and stopped vehicles. In most models, the relationship of speed-flow or speed-density is dependent only on road type and a free flow speed. Researchers in many countries have investigated the relationship of the traffic flow, speed and density since 1934. Hall and Persaud (1986) have studied the flow-density relationship of Canada using an extensive data set collected on the Queen Elizabeth Way in Ontario. Harata (1989) developed the regression model for the three variable relationships for Japanese cities. Olszewski *et al.* (1995) developed an area-wide traffic speed-flow model for the Singapore to get an analytical framework for traffic management measures evaluation. Lum *et al*. (1998) analyzed the speed-flow relationship of the arterial road using the traffic volume and travel time data at a number of arterial roads in Singapore. Daganzo (2005) proposed the relation between traffic flow and traffic speed as a part of an analytical model describing the dynamics of a traffic network. The experiments and simulations made by Geroliminis and Daganzo (2008) suggested that at least in some instances average flow and density were indeed related by a mathematical expression, which has come to be known as the 'Macroscopic Traffic Flow Model' (MTFM).

Nielsen and Jorgensen (2008) explored the speed-flow and flow-density relations based on a large data sample at the motorway network of the greater Copenhagen region in Denmark. Xie *et al.* (2011) investigated the traffic speed-flow relationship of different lanes under a given traffic condition and general levels of service based on Chengdu expressway data of China. However, most researchers focus on the character of the network roads, lacking the comparison of different roads of different type. The aim of this study is to develop flow-speed-density models to estimate functional relationships based on the data collected from Post Office -Teaching Hospital Road in Ile-Ife. Based on these proposed classical models, the basic variables such as the jam density, free flow speed, and density or speed at the maximum level can also be derived for performance evaluation of the selected road. Display of the appropriate speed-flow-density models can provide the traffic managers the traffic flow characters of different roads, which can help them to do more research on the travellers' travel behaviours, traffic ramp control, congestion analysis, and so on.

2.2 Urban Roads

Cities and traffic have developed hand-in-hand since the earliest large human settlements and forcing inhabitants to congregate in large urban areas and in turn enforcing need of urban transportation (Mathew, 2014). To develop efficient road transportation, to serve effectively various land use in an urban area, and ensure community development, it is desirable to establish a network of roads divided into systems, each system serving a particular function or particular purpose. Accordingly, a community should develop an ultimate road-classification in which each system has a specific transportation service function to perform. There are several operational performance measures and level of services (LOS) which have to be taken into account to evaluate the system of roads. Increasing population of urban areas due to shifting of people from rural to urban areas and thus certainly increasing vehicular population on urban roads have caused problems of congestion in urban areas. Road traffic congestion poses a challenge for all large and growing urban areas. This section provides a summary of urban roads with respect to their classification, related operational performance measures and level of services (LOS) involved in each class of urban roads and it also provides strategies necessary for any effective congestion management policy to curb the congestion. The LOS of urban roads with respect to the prevailing traffic density is as given in Table 2.1.

Level of service is the "grading system" used to rate highways and freeways. It is used to justify improvements to roads once they become congested.

Highway Capacity Manual (HCM) (2000) defines the level of service as a qualitative measure describing operational conditions within a traffic stream generally in terms of such service measures as speed and travel time, freedom to maneuver, traffic interruptions, and comfort and convenience.

Letters A-F "grades" are used to signify the level of service.

Table 2.1: Level of Service (LOS) for Urban Roads

Density(veh/ml/ln)	Level of Service	Flow Conditions	
0-12	A	Free-Flow	Uncongested Flow
12-20	B	Reasonable Free Flow	
20-30	C	Stable	
30-42	D	Borders on Unstable	
42-67	E	Extremely Unstable Flow	Near-Capacity Flow

67-100	F	Forced or Breakdown	
>100		Incident Situation	Congested Flow

Source: (May, 1990)

• A = excellent level of service (little interaction between cars)

• E = level of service near highway or freeway capacity

• F = recurrent congestion - unstable traffic flow (stop and go traffic)

2.2.1 Theory on two-lane two-way highways

Two-lane two-way highways form one of the type of highways which present certain particularities, since the vehicles behind are constrained to use the lane of oncoming vehicles during overtaking process due to low speed of the vehicles ahead. Moreover, since urban roads serve mainly two functions in many national highway networks namely; mobility and accessibility, and that two-lane two-way highways constitute the largest portion of urban streets mileage in those networks, this result in their carrying a large portion of urban traffic in most developed and developing countries including Nigeria. Again due to today's growing awareness of issues such as road safety and environment protection, the performances of urban roads have attracted much attention of many researchers across the globe (Tapani, 2008).

By definition, two-lane two-way highways are defined as undivided roadways composed of two lanes, one for use by traffic in each direction, and which overtaking process takes place in the lane of oncoming traffic, if adequate sight distance and a safe gap in the conflicting traffic are available(O'flaherty, 1997). Plate 2.1 illustrates a two way two lane highway while Plate 2.2 illustrates a typical two-lane two-way urban road in Nigeria.

The highway capacity manual classifies two-lane highways into three broad categories, where the first two categories are found in less developed environment and the last category is found in developed areas (TRB, 2010). The three classes of two-lane roads are defined as follows (TRB, 2010):

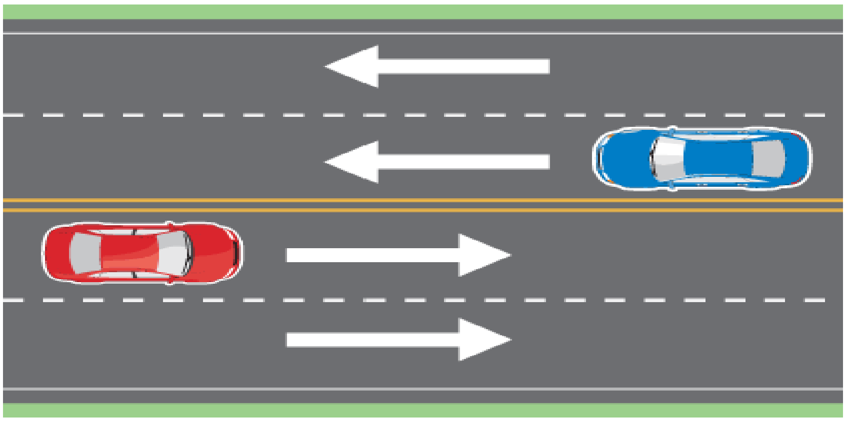


Plate 2.1: A typical Two-way Two-lane Highway (Google Search, 2014)

(a) Examples of Class I Two-Lane Highways

(b) Examples of Class II Two-Lane Highways

(c) Examples of Class III Two-Lane Highways

Plate 2.2: A Typical Two-way Two-lane Urban Road in Nigeria (Google earth, 2014)

(i) Class I: are two-lane highways in which the motorists' expectations are that they will drive at relatively high speeds. Those highways are generally major intercity routes, primary arterials connecting major generators of traffic, daily commuter routes, or primary links to the other arterial highways. These facilities are mostly used when long distance trips are to be undertaken.

(ii) Class II: are two-lane highways on which motorists' expectations are such that they can travel at relatively lower speeds than for class I. Those are two-lane highways whose function is to provide access to class I two-lane highways and those which serve as scenic byways or recreational routes that are not primary arterials. In general, these routes serve relatively short trips, and their average trip lengths are relatively shorter than those for Class I two-lane highways.

(iii) Class III: Two-lane highways which serve moderately developed areas. They may be part of the class I or class II highway which pass through a developed area or small town and are more associated to reduced speed limits. The selected urban road is found under this class.

2.2.2 Analysis and performance evaluation of two-lane two-way highways

The highway capacity manual being a document that provides capacity analysis procedures and methodologies for different types of highways in the United States is adopted in Nigeria``. In this manual, a study of performance measures for two-lane two-way highways could be conducted either by directional segment procedure, that is, an operational assessment for one direction of travel at time, or by two-way segment methodology where performance measures are studied over both directions of travel combined (TRB, 2000). However, the latest version of HCM came out with a method of analysis of two-lane highways, in which the performance of these highways can only be studied using only one direction methodology, and that results obtained are to be expanded to two lanes by applying a weighted average procedure (Transportation Research Board, 2010).

The analysis and modelling of traffic performance for two-lane two-way highways is one of the most important aspects to consider while undertaking the planning, design and operation of these facilities. The evaluation of traffic performance is therefore, a major input to important decisions on public investments made regarding different stages of the life of these highways. However, highway performance study is normally done within the capacity analysis for various highway facilities, by the use of Highway Capacity Manual procedures, where highway performance measures are described in terms of level of service (Transportation Research Board, 2010).

Two-lane two-way highways exhibit operation characteristics different to that of other uninterrupted facilities, resulting from higher interactions between vehicles travelling not only in the same direction but also in the direction of oncoming vehicles. The principal characteristic that separates motor vehicle traffic on two-lane highways from other uninterrupted-flow facilities is that passing maneuvers take place in the opposing lane of traffic. Passing maneuvers are limited by the availability of gaps in the opposing traffic stream and by the availability of sufficient sight distance for a driver to discern the approach of an opposing vehicle safely (Transportation Research Board, 2010). As travel demand and geometric restrictions increase, opportunities to pass decrease. This creates platoons within the traffic stream, with trailing vehicles subject to additional delay because of the inability to pass the lead vehicles. Because passing capacity decreases as passing demand increases, two-lane highways exhibit a unique characteristic: operating quality often decreases precipitously as demand flow increases, and operations can become “unacceptable” at relatively low volume-to-capacity ratios. For this reason, few two-lane highways ever operate at flow rates approaching capacity; in most cases, poor operating quality has led to improvements or reconstruction long before capacity demand is reached (Transportation Research Board, 2010). Due to these unique characteristics, measuring traffic performance at such highways becomes a complex problem.

Currently, three performance measures are proposed by Highway Capacity Manual (Transportation Research Board, 2010). Those performance measures are respectively Average Travel Speed (ATS), Percent Time-Spent-Following (PTSF) and Percent of Free-Flow Speed (PFFS). In this study, the proposed models will utilize the ATS and PFFS as the performance measure for the selected two lane two way urban road.

ATS reflects mobility on a two-lane highway. It is defined as the highway segment length divided by the average travel time taken by vehicles to traverse it during a designated time interval (TRB, 2010); this measure is used in this study. Figure 2.1 illustrates typical average travel speeds and flow relationships for two-lane highways in United States obtainable under the ideal conditions namely lane width greater than or equal to 3.6 m, clear shoulders with width equal or greater than 1.8 m, absence of no-passing zones, traffic stream composed of only passenger cars, level terrain, and absence of obstruction to through traffic (TRB, 2010).

PTSF represents the freedom to maneuver and the comfort and convenience of travel. It is the average percentage of time that vehicles must travel in platoons behind slower vehicles due to the inability to pass. Because this characteristic is difficult to measure in the field, a surrogate measure is the percentage of vehicles traveling at headways of less than 3.0 s at a representative location within the highway segment. PTSF also represents the approximate percentage of vehicles traveling in platoons (Transportation Research Board, 2010). Figure 2.2 illustrates typical percent time-spent-following and flow relationships under the ideal conditions as stated in previous section, for two-lane highways in the United States.

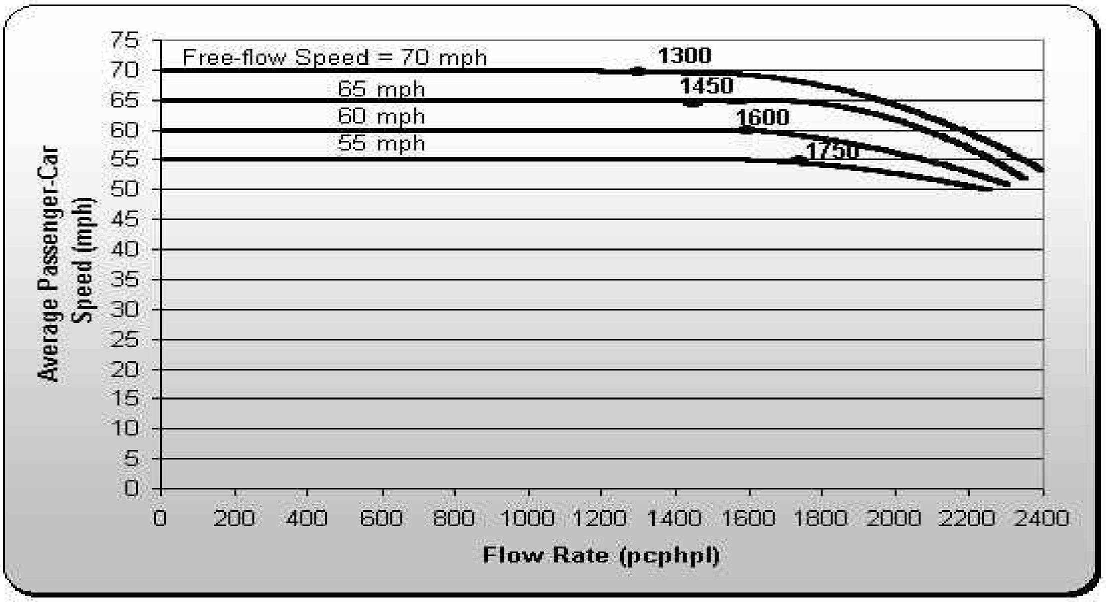


Figure 2.1: Average Travel Speed versus direction flow rate relationships (TRB, 2010)

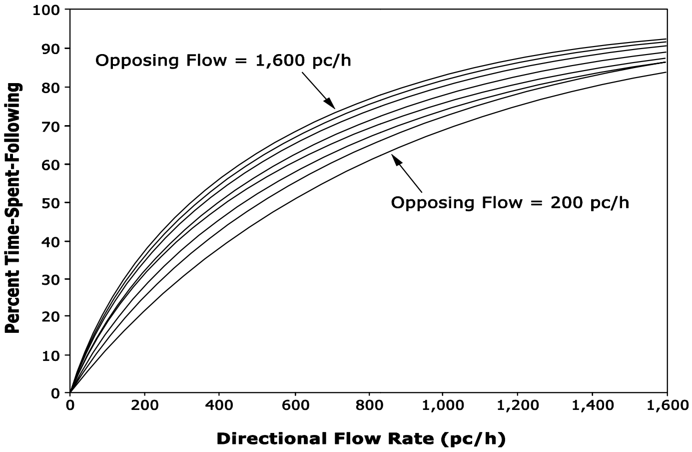


Figure 2.2: Percent -Time-Spent-Following versus direction flow rate relationships (TRB, 2010)

However, the use of percent followers alone was found inadequate in decision making with respect to the upgrading and improving processes of two-lane highways since it does not accurately reflect the effect of traffic level (Shawky and Hashim, 2010).

As percent follower measure is dependent on increase of flow and speed variation, it may be possible that in certain conditions, low traffic levels associated with high variations in speed will result in reduced passing opportunities and consequently, they will present high values of percent followers, in these cases the road upgrading or improvement decision based on PTSF measure alone can be proved erroneous and unjustified (Shawky and Hashim, 2010).

Due to the above mentioned limitations in determination of the PTSF measure, many researchers in different countries have introduced different alternative performance measures other than PTSF to fit local conditions found in their countries. Examples of those alternative measures are follower density, average travel speed of passenger cars, platoon percentage, density, percentage of vehicle impeded, and so on (Hashim and Abdel-Wahed, 2011).

PFFS represents the ability of vehicles to travel at or near the posted speed limit. It reflects the degree at which drivers are travelling in accordance to the posted speed limit. This measure is used to evaluate service quality on class III two- lane highways since high speeds are not expected on these highways, and drivers are more likely to travel at or near the speed limit (TRB, 2010).

$$PFFS = \frac{ATS}{FFS} \quad (2.1)$$

And $ATS = FFS - 0.00776(Q_{d,ATS} + Q_{o,ATS}) - f_{np,ATS}$ (2.2)

Where

ATS = average travel speed in the analysis direction (mi/h);

FFS = free-flow speed (mi/h);

$Q_{d,ATS}$ = demand flow rate for ATS determination in the analysis direction (pc/h);

$Q_{o,ATS}$ = demand flow rate for ATS determination in the opposing direction (pc/h); and

$f_{np,ATS}$ = adjustment factor for ATS determination for the percentage of no-passing zones, from Table 2.2.

In summary, on Class I two-lane highways, speed and delay due to passing restrictions are both important to motorists. Therefore, on these highways, level of service (LOS) is defined in terms of both ATS and PTSF. On Class II highways, travel speed is not a significant issue to drivers. Therefore, on these highways, LOS is defined in terms of PTSF only. On Class III highways, high speeds are not expected, because the length of Class III segments is generally limited, passing restrictions are also not a major concern. In these cases, drivers would like to make steady progress at or near the speed limit. Therefore, on these highways, PFFS is used to define LOS. In this study, the selected road is classified under Class III highway, so the proposed models will utilize PFFS as a performance measure. Note, the use of the average measure of effectiveness over the full length of a two-lane highway is not justified because it is not evident that the total road length shows signs of poor performance, this however, implies that road improvement or upgrading may only be required for the section of the two-lane road where the level of service was found poor not the road as a whole (Van As, 2003). Table 2.3 shows the LOS of two lane highways based on ATS, PSTF, and PFFS.

2.2.3 Capacity determination of two-lane two-way highways

The capacity of two-lane two-way highway in a given condition can be determined either by analytical methods, which include methods provided by Highway Capacity Manual published in 2000, Models developed by the Finnish National Road Administration published in 2000,

Table 2.2: ATS Adjustment Factor for No-Passing Zones ($f_{np,ATS}$)

Flow Rate, Q_o (pc/h)	Percent No-Passing Zones				
	≤ 20	40	60	80	100
FFS ≥ 65 mi/h					
≤100	1.1	2.2	2.8	3.0	3.1
200	2.2	3.3	3.9	4.0	4.2
400	1.6	2.3	2.7	2.8	2.9
600	1.4	1.5	1.7	1.9	2.0
800	0.7	1.0	1.2	1.4	1.5
1,000	0.6	0.8	1.1	1.1	1.2
1,200	0.6	0.8	0.9	1.0	1.1
1,400	0.6	0.7	0.9	0.9	0.9
≥1,600	0.6	0.7	0.7	0.7	0.8
FFS = 60 mi/h					
≤100	0.7	1.7	2.5	2.8	2.9
200	1.9	2.9	3.7	4.0	4.2
400	1.4	2.0	2.5	2.7	3.9

600	1.1	1.3	1.6	1.9	2.0
800	0.6	0.9	1.1	1.3	1.4
1,000	0.6	0.7	0.9	1.1	1.2
1,200	0.5	0.7	0.9	0.9	1.1
1,400	0.5	0.6	0.8	0.8	0.9
≥1,600	0.5	0.6	0.7	0.7	0.7

Source: HCM (2010)

Table 2.2: ATS Adjustment Factor for No-Passing Zones ($f_{np,ATS}$). Contd.

FFS = 55 mi/h					
≤100	0.5	1.2	2.2	2.6	2.7
200	1.5	2.4	3.5	3.9	4.1
400	1.3	1.9	2.4	2.7	2.8
600	0.9	1.1	1.6	1.8	1.9
800	0.5	0.7	1.1	1.2	1.4
1,000	0.5	0.6	0.8	0.9	1.1
1,200	0.5	0.6	0.7	0.9	1.0
1,400	0.5	0.6	0.7	0.7	0.9
≥1,600	0.5	0.6	0.6	0.6	0.7
FFS = 50 mi/h					
≤100	0.2	0.7	1.9	2.4	2.5
200	1.2	2.0	3.3	3.9	4.0
400	1.1	1.6	2.2	2.6	2.7

600	0.6	0.9	1.4	1.7	1.9
800	0.4	0.6	0.9	1.2	1.3
1,000	0.4	0.4	0.7	0.9	1.1
1,200	0.4	0.4	0.7	0.8	1.0
1,400	0.4	0.4	0.6	0.7	0.8
≥1,600	0.4	0.4	0.5	0.5	0.5

Source: HCM (2010)

Table 2.2: ATS Adjustment Factor for No-Passing Zones ($f_{np,ATS}$). Contd.

FFS ≤ 45 mi/h					
≤100	0.1	0.4	1.7	2.2	2.4
200	0.9	1.6	3.1	3.8	4.0
400	0.9	0.5	2.0	2.5	2.7
600	0.4	0.3	1.3	1.7	1.8
800	0.3	0.3	0.8	1.1	1.2
1,000	0.3	0.3	0.6	0.8	1.1
1,200	0.3	0.3	0.6	0.7	1.0
1,400	0.3	0.3	0.6	0.6	0.7
≥1,600	0.3	0.3	0.4	0.4	0.6

Source: HCM (2010)

Table 2.3: LOS for Two-Lane Highways

LOS	Class I Highways		Class II Highways	Class III Highways
	ATS (mi/h)	PTSF (%)	PTSF (%)	PFFS (%)
A	>55	≤ 35	≤ 40	>91.7
B	>50-55	>35-50	>40-55	>83.3-91.7
C	>45-50	>50-65	>55-70	>75.0-83.3
D	>40-45	>65-80	>70-85	>66.7-75.0
E	≤ 40	>80	>85	≤ 66.7

Source: (Highway Capacity Manual, 2010)

methods developed by Brilon and Weisner in 1994 and 1998, and Luttinen in 2001 or by the use of simulation tools, since there are some complex situations which require simulation as the only solution for their evaluations (Van As, 2003).

However, capacity conditions of two-lane two-way highway are difficult to be observed in the field, since very few two-lane roads operate at or near capacity and most of times road improvement or upgrading are done prior to operations at capacity conditions are at hand (Harwood et al., 1999 and TRB, 2000).

The second version of HCM (1965 HCM publication) provides a value of 2000 pc/h as capacity of two-lane rural highways, also, many studies have been conducted to determine the capacity of these highways across the globe (Rozic, 1992). The third version of HCM published in 1985 defined the value of capacity of two-lane highways as 2800 pc/hr with corresponding speed of 72 km/h and critical density of 19.4 veh/km (TRB, 1985). Furthermore, the 2000 HCM proposed the capacity values under ideal conditions for two-lane two-way roads as 1700 pc/h for each direction of travel and 3200 pc/h for both direction of travel combined (TRB, 2000). Mathematically, the capacity of the two- lane two-way highways was found to take the following form (Luttinen, 2001).

$$C = \min\{\frac{1700}{P_d}; 3200\} \tag{2.3}$$

Where

C = Capacity of two-lane two-way highways,

P_d = The proportion of traffic in the major direction.

Prior to the publication of the latest version of HCM, the capacity of a two-way two-lane highway was defined in terms of the total two-way traffic that a given facility is likely to accommodate: since it was believed that the traffic in one direction being in close interaction with traffic moving in the other direction more often when the traffic is heavy, the determination of the capacity for single direction only was very difficult (Mc Shane and Roess, 1990).

The latest version of HCM underlined that the capacity can only be determined separately for each direction and that total roadway capacity is to be determined by weighted average of the one-direction results (TRB, 2010). Therefore in this manual, the capacity under ideal condition is given as 1700 veh/h per one direction and a limit of 3200 veh/h for the total of two directions as previously mentioned in the 2000 HCM version (TRB, 2010). From the value of capacity under ideal condition, some adjustments are applied on it to obtain the capacity under prevailing conditions of a given roadway (TRB, 2010).

However, some researchers have shown that the capacity values provided by HCM underestimate the capacity likely to be obtainable on two-lane highways since it can ideally be estimated by 3600 pcu/h, if the demand is high in both directions (Yagar, 1983). And also the value of capacity of 4000 pcu/h can be achieved on these highways under base conditions in both directions (Rozic, 1992). In this regard, Kim and Elefteriadou (2009) in their study investigating the capacity of two-lane two-way highways using simulation method, found that the capacity under ideal conditions as defined previously vary with the average free-flow speeds, since capacity is likely to be 1800 pcu/h/ln when the average free-flow speed is 64 km/h, while it is 2000 pcu/h/ln when the average free-flow speed is 80 km/h.

In summary, the Nigerian Highway Design Manual (2006) recommends an average traffic capacity of 1500 pcu/h/ln and average travel speed of 50-60 km/h for two lane two way urban highway.

2.3 Traffic Congestion

When the traffic capacity of a roadway is lagging behind the travel demand, traffic congestion set in. Transportation system consists of a group of activities as well as entities interacting with each other to achieve the goal of transporting people or goods from one place to another. Hence, the system has to meet the perceived social and economical needs of the users. As these needs change, the transportation system itself evolves and problems occur as it becomes inadequate to serve the public interest. One of the negative impacts of any transportation system is traffic congestion. Levinson (2003) said, traffic congestion management has the goal to optimize transportation flow of people and goods particularly in the metropolitan area .To define what is meant by traffic congestion, the executives report of Organization of Economic co-operation and Development (OCED) and European Conference of Ministers of Transport (ECMT), January 2004, stated that there is no single definition for congestion since congestion takes on many phases, which occur in many different contexts and caused by many different process. However, the following are some of their definitions of congestion:

i.) Congestion is a situation in which demand for road spacing exceeds supply

ii.) Congestion is the impedance vehicles impose on each other due to the speed-flow relationship, in condition where the use of a transport system approaches capacity.

iii.) Congestion is essentially a relative phenomenon that is linked to difference between the road system performance that users expect and how the system actually performs.

Grant-Muller and Laird (2007) described traffic congestion as a widely recognized transport cost and is a significant factor in transport system performance evaluation which affects transport planning decisions. To individual motorists, congestion is a cost they bear, but each motorist also imposes congestion on other road users. The existence of congestion can be explained by the fact that each additional vehicle imposes more total delay on others than they bear, resulting in economically excessive traffic volumes.

The travel time or delay is in excess of that normally incurred under light or free flow traffic condition. The travel time or delay is in excess of agreed upon norm which may vary by type of transport facility, travel mode, geographical location, and time of day. In the procedure for congestion management the root cause of congestion and the remedies for managing the congestion have to be found out, updating the signalization if it is needed. It is always better to use good signalization for minimizing impact of congestion. More space can be provided by making use of 'turn bays' if geometry permits. Parking restrictions also help in congestion management on urban streets. Now, some important strategies to manage the congestion on urban streets will be discussed. Congestion can be recurrent (regular, occurring on a daily, weekly or annual cycle) or non-recurrent (traffic incidents, such as accidents and disabled vehicles) as revealed by ECMT (2000):

(i) Recurrent congestion: Recurrent congestion generally occurs at the same place, at the same time every weekday or weekend day. This is generally the consequence of factors that act regularly or periodically on the transportation system such as daily commuting or weekend trips. Recurrent congestion is predictable and typically occurs during peak hours. It displays a large degree of randomness in terms of duration and severity.

(ii) Non-recurrent congestion: Non-Recurrent congestion is the effect of unexpected, unplanned large events (road woks, accidents, special events and so on) that affect transportation system more or less randomly and as such, cannot be easily predicted.

Basically at arterial level, congestion can be countered by any of the following ways:

2.3.1 Signal based remedies

Signal based remedies for congestion management can be achieved by implementing the following two strategies:

(i) Metering plans

It is the congestion management policy for road congestion to limit the volumes arriving at critical locations. It uses some control strategies within the congestion networks by storing vehicles at links defined to be part of system under control. It should be noted that metering concept does not explicitly minimize delays and stops but manages queue formation (Mathew, 2014).

(ii) Shorter cycle length

If on any intersection higher cycle time is provided then it will certainly create problems like increase in queue length and platoon length discharged and it will lead to increase in blockage of intersection, with substantial adverse impact on system capacity. This is particularly when short link lengths are involved Mathew (2014).

2.3.2 Non-signal based remedies

If the problem of congestion does not get resolved by signalization the next set of action is to add more space. *More space* means there is need of provision of additional lanes or some other facility. This can be achieved by adding left turn bays / right turn bays, removing obstructions to through flows by adding more space and free movements Mathew (2014). Some non-signal based remedies are:

(i) Two-way turn lanes

On suburban and urban arterials dedication of a central lane for turns in either direction is provided. This also allows for storage and for vehicles to make maneuvers in two distinct steps. Leaving the arterial and entering it is separated into two distinct steps. Vehicles leaving the arterial do not have to block a moving lane while waiting for a gap in the opposing flow. Entering vehicles do not have to wait for a gap simultaneously in both directions (Mathew, 2014).

(ii) Reversible Lane

Reversible lanes have great advantage of matching lane availability to the peak demand. Lanes that are reversible means can be split into various combinations for different times of day to match the demand. E.g. eight lanes can be split into 6:2 or 5:3 and so forth if required to match up for the demand. It should be noted that some jurisdictions have combined two-way lanes and reversible lanes on same arterial 'because combination of peak- period congestion and increased road side development'. The concerns with reversible lanes and relates to the misuse and lanes by the driver (particularly the unfamiliar driver), despite the signalization over the lanes Mathew (2014).

(iii) High occupancy vehicle lanes

High occupancy vehicle (HOV) lanes are designed to help move more people through congested areas. HOV lanes offer users a faster, more reliable commute, while also easing congestion in regular lanes by moving more people in fewer vehicles. HOV lanes on provincial highways are reserved for any of the following passenger vehicles carrying at least two people (often referred to as 2+):

- Car
- Commercial truck less than 6.5 meters long
- Minivan
- Motorcycle
- Taxi or limousine

In addition, vehicles with a special green license plate (plug-in hybrid electric or battery electric vehicle) can use an HOV lane, even without passengers. This helps buses keep to their schedules and provide reliable, efficient service. Emergency vehicles are permitted to use the HOV lanes at all times (Mathew, 2014).

(iv) Kerb parking prohibition

Congestion can be managed by prohibiting the kerb parking. Kerb parking means on street parallel parking. If such parking is avoided it implies oblique and right angled parking is also prohibited and hence provides more space for traffic flow so congestion is minimized (Mathew, 2014).

(v) Lane marking

Longitudinal lane markings such as solid white lines and broken white lines restrict overtaking maneuver of vehicles which encourages mix through traffic flow unobstructed resulting in reducing the congestion (Mathew, 2014).

(vi) Equity offset

Offset on an arterial are usually set to move vehicles smoothly along the arterial, as is logical. Equity offset allows the congested arterial to have its green at upstream intersection until the vehicle just begin to move , then switches the signal, so that these vehicles flush out the intersection, but no new vehicles continue to enter Mathew (2014).

(vii) Imbalanced Split

This is the procedure of allocating the 'available green' in proportion to the relative demands. It is sometimes desirable to split green as per demand of various routes to meet peak hour demands of respective routes Mathew (2014).

2.4 Traffic Flow Theory

The performance evaluation and occurrence of traffic congestion can be best understood by having in-depth knowledge of the traffic flow theory. Traffic Flow theory comprises the study of the movement of individual drivers and vehicles between two points and the interactions they make with one another and plays a vital role in the progress of overall social productivity. In the 1950s James Lighthill and Gerard Whitham, two experts in fluid dynamics, (and, independently P. Richards), modelled the flow of car traffic along a single road using the same equations describing the flow of water (Lighthill and Whitman, 1955 and Richard, 1956). The basic idea is to consider traffic on a large scale so that cars are taken as small particles and to assume the conservation of the number of cars. The Lighthill-Whitman-Richard (LWR) model is described by a single conservation law, a special partial differential equation where the dependent variable, the car density, is a conserved quantity, i.e. a quantity which can neither be created nor destroyed. Traffic flow when likened to fluid flow has several parameters associated with it. These parameters could then provide information regarding the nature of traffic flow, which would help a traffic analyst or modeller in detecting any variation in flow characteristics. Thus, understanding traffic behaviour would require a thorough knowledge of these traffic stream parameters and their mutual relationships.

The knowledge of traffic flow is useful to the road engineers in developing roads, motorways and transportation plans, performing economic analyses, establishing geometric design criteria, selecting and implementing traffic control measures and evaluating the performance of transportation facilities. In this study, traffic flow characteristics – traffic speed, travel time, volume and density – are fundamental for planning, design and operation of roads and motorways (highways) and transport facilities. Determination of relationships between concentration, density, speed and volume is of primary interest in traffic flow theory, which involves the development of mathematical relationships among the primary elements of a traffic stream – flow, speed, density. The relationships help the traffic engineer in planning, designing and evaluating the effectiveness of implementing traffic-engineering measures on a road or highway system. The basis for further analyses is data collection on several elements of traffic stream. One example of the use of traffic flow theory in design is the determination of adequate lane lengths for storing left-turn vehicles on separate left-turn lanes. The determination of average delay at intersections and freeway ramp merging areas is another example of the application of traffic flow theory. Another important application of traffic flow theory is simulation, where mathematical algorithms are used to study the complex interrelationships that exist among the elements of a traffic stream or network to estimate the effect of changes in traffic flow on factors such as accidents, travel time, air pollution and fuel consumption. Occasionally, single vehicles traverse the transportation facilities without significant interference from other vehicles. But the same facilities also experience simultaneous usage by streams of vehicles. The resulting traffic conditions range from almost free flow when only a few relatively unconstrained vehicles occupy a roadway to highly congested conditions when the roadway is jammed with slow-moving vehicles. The determinant of these traffic flow models is the car-following rule adopted by drivers in an attempt to maximize their speeds while maintaining an acceptable level of safety. They accomplish this by adjusting the distance between vehicles, depending on their speed. The basic variables that describe the prevailing conditions within a vehicular stream (flow, concentration, and mean speed) are introduced and the fundamental relationship between the three stream variables is postulated and applied to several traffic phenomena, including the propagation of shock waves in traffic. Consider the case of vehicles following each other on a long stretch of roadway or guide way. Furthermore, assume that these vehicles are not required to interrupt their motion for reasons that are external to the traffic stream, such as traffic lights, transit stations, and the like. As a general rule, the spacing between vehicles should be such that if a sudden deceleration becomes necessary for a leading vehicle, the following vehicle has ample time and distance to perceive the situation, react to it, and be able to decelerate safely without colliding with the stopping, leading vehicle. Parenthetically, the term vehicle may be taken to mean a vehicular train consisting of a number of articulated vehicles rather than a single vehicle. Using the following notation, a relationship between spacing, speed, and deceleration (assumed constant) can be developed. In formulating a mathematical model for a continuum traffic flow, there are basic steps that are often used as given by Haight (1963); Haberman (1977); Banks (1992); Michalopoulos *et al*. (1993); Bellomo *et al*.(2002) and Bellemans *et al.* (2002) below:

Identify appropriate conservation laws (e.g. mass, momentum, energy, etc) and their corresponding densities and fluxes.

Write the corresponding equations using conservation law and close the system of equations by proposing appropriate relationships between the fluxes and the densities.

2.4.1 Overview on basic terms used in traffic flow theory

Traffic flow theory is one of the disciplines of transportation engineering which uses mathematical analysis and modelling to explain road traffic flow mechanisms. The theory of traffic flow uses mainly three interrelated parameters for which relationships are worthy to be understood. In order to establish these relationships on two-lane highways, it is essential to understand those parameters for which the steady- state flow fundamental relationship is shown in the following equation:

$$Q = U_s K \tag{2.4}$$

Where:

Q = Flow (veh /h)

U_s= Macroscopic speed (Km/h)

K = Density (Veh/km)

Since density cannot be measured directly from the field, it is computed using its relationship with speed and flow as:

$$\text{Density} = \frac{\text{Flow}}{\text{Speed}} \tag{2.5}$$

Different terms used in analysis of traffic flow are defined in the following part as described in the Highway Capacity Manual (TRB, 2000). These terms are grouped into two categories according to the relative approach in which they are used, as shown in Figure 2.3.

The functional effectiveness of a highway is measured in terms of its ability to assist and accommodate the flow of vehicles with both safety and efficiency. In order to measure its level of effectiveness, certain parameters associated with the highway must be measured and analyzed. These properties were considered in this study and they include:

- The quantity of traffic
- The type of vehicles within the traffic stream
- The distribution of flow over a period of time
- The average speed of the traffic stream
- The density of the traffic flow.

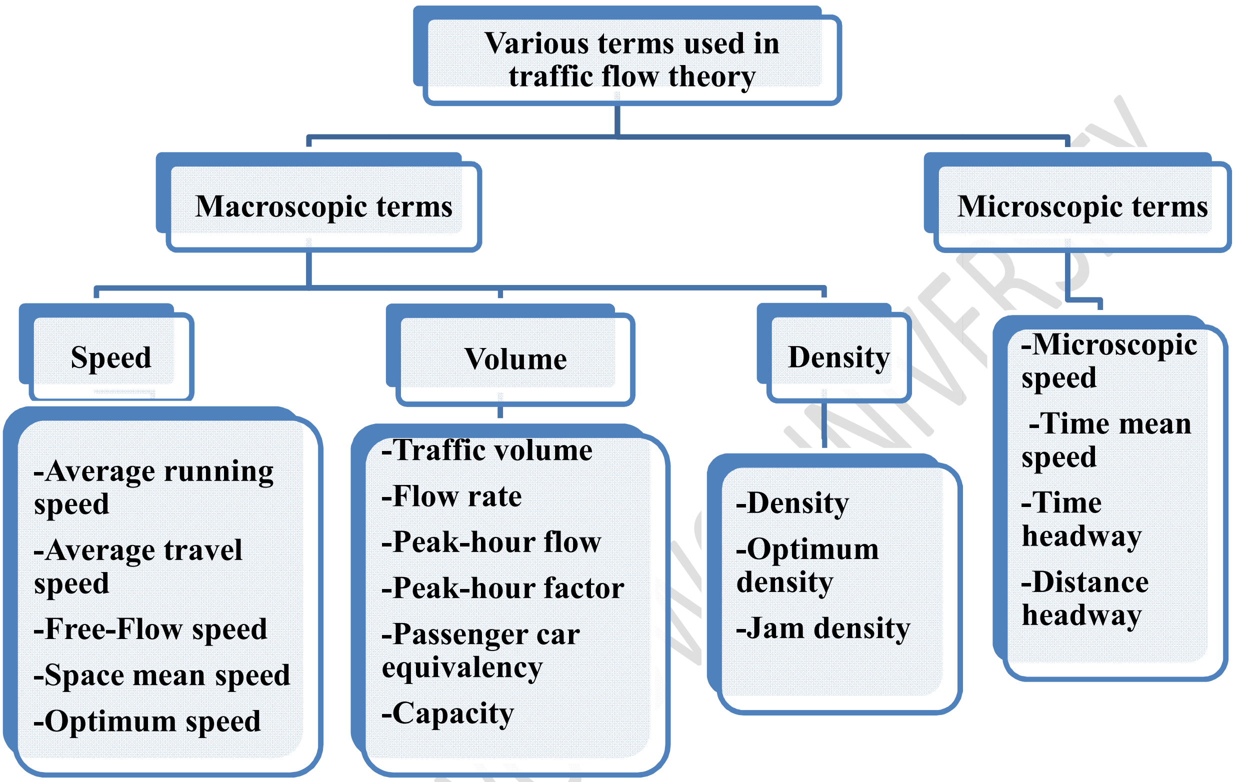


Figure 2.3: Most used terms in traffic flow theory (Twagirimana, 2013)

Analysis and modelling of these parameters will directly influence the scale and layout of the proposed highway, together with the type and quantity of materials used in its construction. This process of examination is termed traffic analysis and modelling and the following sections deal with the parameters and how they are related.

Traffic volume

Traffic volume is the total number of vehicle that pass over a given point or section of a lane or 1roadway during a given time interval; volumes are expressed in terms of annual, daily, hourly or sub hourly periods HCM (2000).

The following terms are used to characterize traffic volume on a given segment of the roadway:

- **Flow rate (q)** is defined as a rate at which vehicles pass over a given point of roadway during the sub hourly time period, usually 15 minutes. And it is expressed as number of vehicles per unit of time mostly hour or second.

  HCM (2000) defines the flow rate as the equivalent hourly rate at which vehicle pass over a given point or section of a lane or roadway during a given time interval of less than 1 hour, usually 15minutes.
  Volume and flow rate are variables that quantify demand, that is, the number of vehicles occupants or drivers (usually expressed as the number of vehicles) who desire to use the roadway during a specific time period. Congestion can influence demand, and the observed volume sometimes reflects the capacity constraints rather than true demand.

  $$q = 4V_{15} \qquad (2.6)$$

  Where V_{15} = traffic volume at time interval of 15minutes.
  For a roadway of two or more segments, Tsubota *et al*. (2014) proposed that the area average flow, q can be computed by:

  $$q = \frac{\Sigma q_i l_i n_i}{\Sigma l_i n_i} \qquad (2.7)$$

  Where q_i is the average flow of segment *i,* the flow is measured at the downstream of each segment i= 1,2,…….n and l_i is the length of segment *i* and n_i is the number of lanes of segment *i*.
- **Peak hour flow** is defined as the highest traffic flow which is obtained during any successive 60 minutes. This flow is mostly considered in capacity and other traffic studies since it provides the most critical period which can affect the operation of a given highway, thereby capacity.

- **Peak hour factor (PHF)** is defined as the ratio of total hourly volume to the maximum rate of flow within the hour. The peak hour factor can be computed as shown in the following equation: Peak hour factor

$$PHF = \frac{V}{4 \times V_{15}} \quad (2.8)$$

Where: PHF = Peak hour factor, V = Hourly volume (veh/h), and

V_{15} = Volume during peak 15minutes of the peak hour (veh/15minutes).

- **Capacity** can be defined as maximum rate of flow that can be achieved on a given roadway facility under prevailing roadway, traffic and control conditions.
- **Passenger car equivalency** can be defined as the equivalent value which is representative of a number of passenger cars that would use the same amount of capacity of a given highway as heavy vehicles under the prevailing conditions. The passenger car equivalents for different types of vehicles are presented in Table 2.4 as recommended in the Nigerian Highway Design manual (2007).

Travel speed

Although traffic volumes provide a method of quantifying capacity values, speed is an important measure of the level of service provided by a roadway. It is an important measure of effectiveness defining level of service for many types of facilities, such as rural two-lane highways, urban streets, freeway weaving segments and others HCM (2000).

Speed can be defined as the rate of motion of vehicle along a given roadway and it is expressed as distance per unit of time, generally as kilometer per hour (km/h).

In the analysis of traffic stream, the following speed parameters can be considered depending on the purpose of the study:

- **Average running speed** is a speed computed when vehicles are only in motion and is obtained by dividing the length travelled by the time a platoon of vehicles uses to travel a given length.

$$S = \frac{nL}{\sum_{i=1}^{n} t_{\mathrm{i}}} = \frac{L}{\frac{1}{\mathrm{n}}\sum_{i=1}^{n} t_{\mathrm{i}}} = \frac{\mathrm{L}}{t_{\mathrm{a}}} \quad (2.9)$$

S = average travel speed (km/h)

L = length of the highway segment (km)

t_i = travel time of the ith vehicle to traverse the section(hour)

n = number of travel times observed, and

$t_{\mathrm{a}} = \frac{1}{\mathrm{n}}\sum_{i=1}^{n} t_{\mathrm{i}}$ = average travel time over segment, L, (hour)

- **Free-flow speed** is defined as the average speed of vehicles travelling over a roadway, measured under low-volume traffic conditions, that is, when density and flow rate on particular section of the roadway are both zero. In this case, the drivers are free to drive at their desired speed and are not embedded by the presence of others.

Table 2.4: Passenger Car Equivalent (Nigerian Highway Manual, 2007)

Vehicle Type	Passenger Car Equivalent Units			
	Rural	Urban	Roundabouts	Traffic Signals
Cars and Light Vans	1.00	1.00	1.00	1.00
Commercial Vehicles	3.00	1.75	2.80	1.75
Buses and Coaches	3.00	3.00	2.80	2.25
Motorcycles	0.75	0.75	0.75	0.33
Pedal Cycles	0.50	0.50	0.50	0.20

- **Space mean speed** also termed macroscopic speed is defined as a speed of a traffic stream measured under basis of the average travel time of vehicles traveling over a given length. This speed draws this name from the fact that the average travel time weights the average to the time each vehicle spends in a given roadway. The space-mean speed is the average speed that has been used in the majority of traffic models.
  - Considering a segment of the highway whose length equals L. The time needed by the i-th vehicle to travel along this highway section is denoted by t_i. The space-mean speed u_s is defined in the following way:

$$\bar{u}_s = \frac{L}{\frac{1}{n}\sum_{i=1}^{n} t_i} = \frac{L}{t_a} \tag{2.10}$$

Where:

U_s = Space mean speed (km/h)

L = length of the highway segment(km)

t_i = travel time of the ith vehicle to traverse the section(hour)

n = number of travel times observed, and

$t_a = \frac{1}{n}\sum_{i=1}^{n} t_i$ = average travel time over segment, L, (hour)

  - **Optimum speed** is defined as the speed which occurs when the level of traffic flow is at capacity.

Traffic density

Density is defined as average number of vehicles occupying a given length of a roadway at a particular instant; density is expressed as vehicle per kilometre.

Density is an important parameter for uninterrupted-flow facilities since it characterizes the quality of traffic operations of a given facility, also describing the proximity between vehicles and reflecting the freedom to maneuver within the traffic stream. Following density parameters are of great importance in defining relationship between traffic characteristics:

- **Optimum density** can be defined as a density which corresponds to the maximum flow.
- **Jam density** is defined as the maximum density that may be found on any road. This density is obtainable for stopped vehicles on a given road, that is, when flow rate is zero.

Microscopic speed

This is also called spot speed; it is defined as a rate of motion at which an individual vehicle uses travelling a certain distance over time. It can be computed as shown in the following formula:

$$U_i = \frac{dX}{dT} \qquad (2.11)$$

Where:

U_i = Microscopic speed of vehicle i

dx = Short distance travelled

dT = Short time interval

Time mean speed

This is defined as the arithmetic mean of the speeds of vehicles passing a point on a highway during an interval of time. The time mean speed is computed as follows:

$$\bar{u}_t = \frac{1}{n}\sum_{i=1}^{n} u_i \qquad (2.12)$$

Where:

$\bar{u}_t$ = time mean speed (km/h)

u_i = speed of the ith vehicle (km/h)

From the field data, the research carried out by Wardrop (1952) has shown that a relationship exists between time mean speed and the space mean speed. That relationship is formulated as follows:

$$\bar{u}_t = u_s + \frac{\sigma^2}{u_s} \qquad (2.13)$$

With σ is the standard deviation of macroscopic speed which is obtained as shown in the following formula:

$$\sigma = \frac{\sum_{i=1}^{n} k_i (u_i - u_s)^2}{K} \quad (2.14)$$

K_i = Density of vehicles in each individual stream

K = Total density

u_i and u_s are as previously determined.

Time headway

This is defined as a difference between the time the front of a vehicle arrives at a point on a highway and the time the front of the next following vehicle arrives at the same point. Time headway is expressed in seconds.

Distance headway

This can be defined as the longitudinal distance between the front bumper of lead vehicle and the front bumper of the following vehicle. This distance includes the length of the lead vehicle and the gap distance between the lead and the following vehicles. Space headway is expressed in meters.

Moreover, it is worthy to note that the distance headway can be obtained either photographically or by computation using individual speed measurement and time headway as follows:

$$e_{n+1} = h_{n+1} + \dot{X}_n \quad (2.15)$$

Where:

e_{n+1} = Distance headway of following vehicle (m)

h_{n+1} = Time headway of the following vehicle at a given point (sec)

$\dot{X}_\text{n}$ = Speed of lead vehicle during the time period (m/sec)

It can also be noted that there is a relation between average distance headway between successive vehicles and density as shown in the following formulae (Leutzbatch, 1988):

$$K = \frac{1}{a} \quad (2.16)$$

Where:

K= density

a = average distance headway

With average distance headway computed by averaging the individual distance headways as shown in the following equation:

$$a = \frac{\sum a_n}{N} \quad (2.17)$$

Where:

a_n =Individual distance headway (m/ Veh)

N = Number of observed distance headways

2.5 Traffic Flow Modelling

Since the 1935's pioneering work of Greenshields in traffic studies, many researchers have developed various models aiming to describe the relationships between traffic characteristics on uninterrupted flow facilities Janvier (2013). These models were developed following two main approaches, macroscopic approach and microscopic approach. The first approach is macroscopic approach that uses traffic average speed, flow and density in flow analysis. The second approach which is microscopic approach also termed car-following theory deals with individual vehicle spacing and speed. Macroscopic stream models, being the area of focus for this study, represent how the behaviour of one parameter of traffic flow changes with respect to another. Most important among them is the relation between speed and density. The first and most simple relation between them were proposed by Greenshields. Based on the relationship between the three parameters (speed, flow and density), many observations have been carried out to determine the relationship between any two of these parameters, for, with one relationship established, the relationship between the three parameters is determined as mentioned by Salter (2012).

2.5.1 Macroscopic traffic flow modelling

First and foremost, in order to simplify the representation of relationships between traffic flow parameters, Greenshields (1934) found that, for two-way highways, speed decreases linearly as density increases on such highways. He subsequently proposed a model under the assumption of a linear relationship between speed and density.

The resulting ***Speed-Density*** relationship is shown in the following equation:

$$U_s = U_f \left(1 - \frac{K}{K_j}\right) \quad (2.18)$$

Where:

U_s = Space mean speed

U_f = Free flow speed (km/h)

K = Density (veh/km)

K_j = Jam density (veh/km)

Greenshields (1934) showed that the speed-density relationship presents significant changes when the density on highway is heavy, which is logical since entire stream of vehicles is likely to be hindered by any form of delay or slow vehicles. The Greenshields (1934)'s study constitutes one single and general case. This model was found to be simple and satisfy all boundary conditions ($u = 0$ at $k = k_j$ and $u = u_f$ at $k = 0$).

However, many other researchers found non-linear relationship between speed and density and they developed more complex models which are ranged from single to three regime models using either statistical approach or hydrodynamic analogy to traffic flow phenomenon (Janvier, 2013). Using the applied fluid dynamic principles, as proposed by Lighthill and Withmam (1955) who postulated that traffic flow can be treated as a fluid flow in different traffic situations, Greenberg (1959) obtained a logarithmic flow-density curve which presents a maximum value of flow when $k_j/_k = e$ and optimum speed (u_o) when speed = c.

To achieve this, Greenberg applied the equation of motion of one-dimensional fluid together with the equation of continuity (conservation of flow) to the traffic flow situation. The two used equations are shown as follows:

$$\frac{du}{dt} = \frac{-C^2}{K}\frac{\partial k}{\partial t} \tag{2.19}$$

$$\frac{\partial k}{\partial t} + \frac{\partial q}{\partial x} = 0 \tag{2.20}$$

Where:

u=Traffic velocity (miles per hour)

k =Traffic density (vehicles per mile)

x = Distance on the roadway (m)

t = Time (hour)

C = Parameter

q = Traffic flow rate (vehicles/hour)

The obtained speed-density curve is shown in the following equation:

$$u = C\ln\frac{k_j}{k} \tag{2.21}$$

Where:

C = u_o (Speed at maximum flow, miles/hour)

K = Traffic density (vehicles per mile)

K_j = Jam density (vehicles per mile)

From Equation 2.16, flow-density curve can be obtained using the fundamental relationship (Equation 2.1).The resultant curve is presented in following equation:

$$u = C\,k\,\ln(\frac{k_j}{k}) \tag{2.22}$$

Underwood (1961) compared the curve established by Greenshields, Norman and Greenberg and found that the Greenberg model provided a best representation of traffic data in a congested regime. However, the Greenberg model was found unsuitable in free-flow regime, due to its inability to provide a realistic free-flow speed. This incited Underwood to propose another model formulated as follows:

$$u = u_f . e^{-\frac{k}{k_o}} \tag{2.23}$$

Where:

u = Speed (km/hour)

u_f = Free-flow speed (km/hour)

k_o = Optimum density (veh/km)

k = Density (veh/km)

While investigating speed-density curve on Eisenhower expressway, Drake *et al.* (1965) found that the curve presents concavity at low density, they then proposed a bell-shaped curve of the form:

$$u = u_f . e^{[-\frac{1}{2}\frac{k}{k_o}]} \tag{2.24}$$

All parameters are as previously determined.

Drew (1968) proposed a model which provides a generalized form of the speed-density curve by introducing a new parameter in the Greenshields' model. The proposed model is formulated as follows:

$$u = u_f[1 - \left(\frac{k}{k_j}\right)^{n+1/2}] \quad (2.25)$$

Where:

n= additional parameter with values -1, 0 and +1, giving the form of the model to be respectively exponential, parabolic and linear model.

All other parameters are as previously determined.

Edie (1960) carried out a study with the purpose of examining different established speed-density curves, he found the existence of discontinuities from the traffic data on many empirical curves. These discontinuities were marked by a sharp capacity drop at some critical densities, where the maximum flow rate achievable in free-flow regime were found to be considerably higher than that in congested regime; this gave birth to a dual-regime traffic stream model. He subsequently hypothesized that Underwood model can be used for the free-flow regime and the Greenberg model be used for the congested-flow regime Edie (1960).

Drake *et al.* (1965) also examined different developed models and proposed that the speed density models are better presented under multi regime models. They suggested three additional representations of speed-density curves. The first representation was the use of separated Greenshields'-type model for the free-flow regime and the congested-flow regime. The second was the use of constant-speed model for the free- flow regime and a Greenberg model for congested-regime. And finally, the third representation which is slightly different from the two previous representations was a three-regime model where the Greenshields' model was applied to free-flow, transitional, and congested-flow regimes separately. However, researchers showed that there exists a problem associated with the use of multi regime models which is to determine the exact point where the breakpoint between regimes occurs.

Applying the steady-state flow relationship between traffic parameters to the linear ***speed-density*** relationship, as given in Greenshields' model, the parabolic shaped ***speed-flow*** curve was obtained as shown in the following formula (Mannering *et al*., 2007):

$$q = K_j\,(U_s - \frac{U_s^2}{U_f}) \quad (2.26)$$

Where:

q= Flow rate, veh/h

U, K_j *and* U_f are defined as previously.

In order to find the speed at maximum flow, Equation 2.26 is differentiated and put equal to zero:

$$\frac{dq}{dU_s} = K_{\rm j}\,(1 - \frac{2{\rm U_s}}{U_{\rm f}}) = 0$$

Since $K_j \neq 0$, the term within the brackets must equal zero, therefore:

$1 - \frac{2{\rm U_m}}{U_{\rm f}} = 0$, thus

$${\rm U_m} = \frac{U_{\rm f}}{2} \tag{2.27}$$

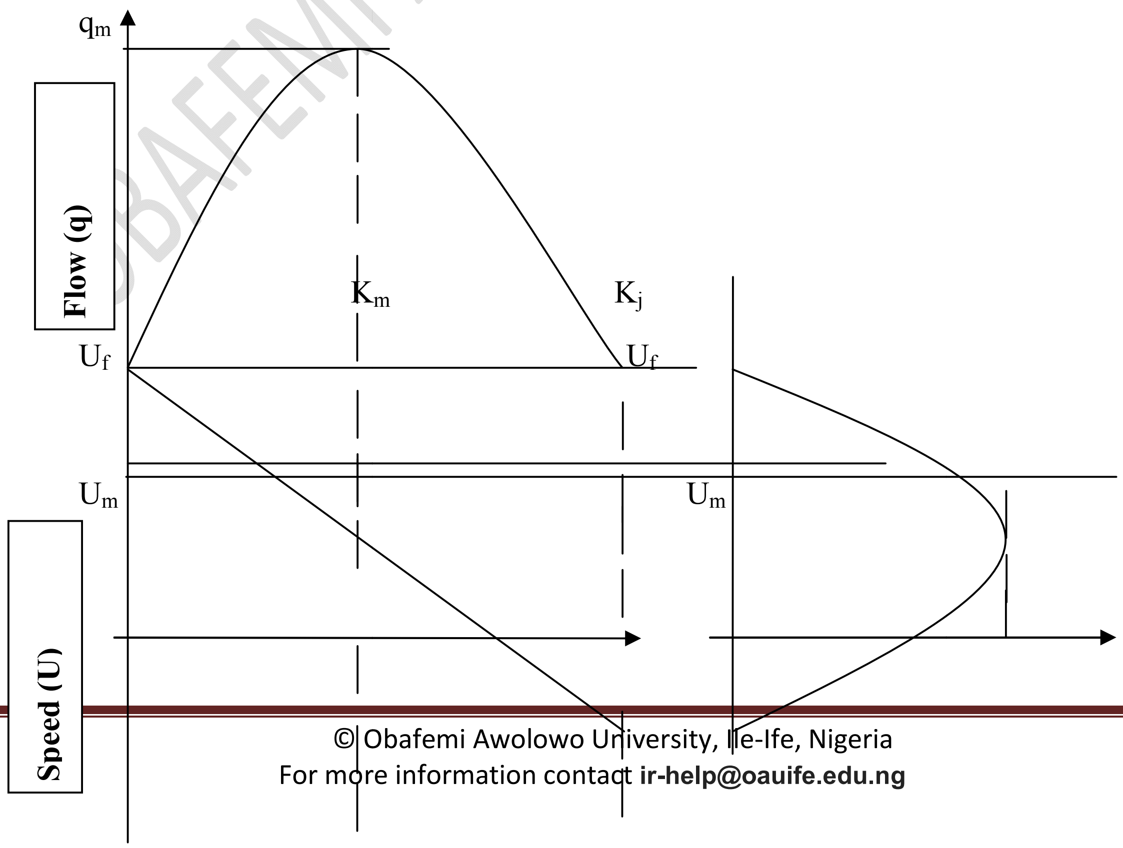
This shows that space mean speed U_m at which the volume is maximum is equal to half the free mean speed. Its location is shown in Figure 2.4.

In a similar way, by the use of the equation 2.26 to the ***speed-density***, relationship shown in equation 2.28, a parabolic shaped ***flow-density*** curve was obtained. The curve presents a peak point corresponding to the maximum flow with intermediate levels of speed and density. The resultant curve is shown in the following equation:

$$q = U_s K = U_{\rm f}\,(1 - \frac{\rm K}{K_{\rm j}}) \times {\rm K} = U_f\,(K - \frac{K^2}{K_j}) \tag{2.28}$$

All parameters are as previously determined.

This is a parabolic relationship and is illustrated below in Figure 2.4. In order to establish the density at which maximum flow occurs, Equation 2.28 is differentiated and set equal to zero as follows:



K_m K_j 0 q_m

Density (K) Flow (q)

Figure 2.4: Generalized Speed-Flow-Density Relationships on Uninterrupted-Flow Facilities (Baerwald, 1976)

$$\frac{dq}{dt} = U_{\mathrm{f}}\,(1 - \frac{2\mathrm{K}}{K_{\mathrm{j}}}) = 0$$

Since $U_{\mathrm{f}} \neq 0$, therefore: $(1 - \frac{2\mathrm{K_m}}{K_{\mathrm{j}}}) = 0$, thus;

$$\mathrm{K_m} = \frac{K_{\mathrm{j}}}{2} \tag{2.29}$$

K_m, the density at maximum flow, is thus equal to half the jam density, k_j. Its location is shown in Figure 2.4.

The speed-density-flow relationships discussed in previous section was illustrated in Figure 2.4. The figure shows a generalized representation of relationships between traffic flow characteristics on uninterrupted flow facilities, developed under assumption of linear speed-density.

Some points on the curves are of great importance in understanding traffic behaviour on a given facility. There are two situations, under which the flow rate is zero. The first situation occurs when there are no vehicles on the highway. Under this condition, the attributed speed is theoretical and corresponds to the maximum speed (U_f) which can occur on a given highway. This speed termed free-flow speed is the speed with which vehicles will be travelling on highway without hindrance by the presence of other vehicles.

The second situation in which flow rate is zero corresponds to a high density in which vehicles arrive at a complete stop. Under this condition, there is no movement of vehicles. Vehicles will tend to line up end to end, the speed is zero and the corresponding density is called jam density (K_j).

As density increases from zero, the flow also increases progressively from zero to a maximum value. After the flow reaches that value, a further increase in density will then result in decrease in flow which will eventually reach zero when the density is equal to jam density. That maximum value of flow under prevailing conditions corresponds to the capacity of a given highway.

Furthermore, the flow conditions corresponding to low density (density less than optimum density) and high speed are referred to as no congested regime, and flow condition with higher density (density greater than density at which flow is at the maximum) and low speed indicates congested regime. Moreover, it is worthy to note that the slope of any line drawn from origin of the speed-flow curve to any point on the curve indicates density, and in the same way, the slope of any line drawn from the origin of the flow-density curve gives speed (Roux, 2001).

2.5.2 Microscopic traffic flow models (Car-following theories)

Understanding car following theories contributes significantly to a deep understanding of traffic flow, since these theories provide a better description of a one-by-one vehicle following process on the same lane in traffic flow. Moreover, the basic element of this theory resides on knowledge of average distance headway between vehicles in a platoon. Chandler et al. (1958) proposed a linear model based on stimulus-response concept, where they hypothesized that the response of a driver is proportional to the stimulus he received. Further on, in the study which resulted into the development of car following model, the researchers associated with General Motors group have proposed five generations of car-following models (May, 1990). All these models were established working under the above stated stimulus-response concept, presented as shown in the following statement (May, 1990). Response = function [Sensitivity, Stimuli]

The response and stimuli are the common components of those models. The response is indicated by the acceleration or deceleration of the following vehicle depending on the fluctuation of velocity of the lead vehicle, while the difference in velocity between the lead and following vehicle corresponds to stimuli, the sensitivity term is the only variable term of those models which differentiates one generation to another (May, 1990).

At the early stage of development of these models, the sensitivity term was considered constant. The equation was given a following form:

$$\ddot{X}_{n+1}(t+\Delta t) = \propto [\dot{X}_n(t) - \dot{X}_{n+1}(t)] \qquad (2.30)$$

Where:

$\ddot{X}_{n+1}$= Acceleration/deceleration of the following vehicle (ft/sec^2)

$\dot{X}_n$= Velocity of the lead vehicle (ft/sec)

$\dot{X}_{n+1}$= Velocity of the following vehicle (ft/sec)

α = Sensitivity term (sec^{-1})

t = at time t

t+Δt = Variation of time after t time

In developing the second generation model, the research group incorporated the distance headway in the sensitivity term. The model was developed with two different sensitivity terms, one used for traffic situation associated with low spacing and other used for the situation associated with a high spacing. Due to difficulties to determine which sensitivity value to be use in the model, the research group proposed to measure the sensitivity term as an inverse function of the distance headway. After multiple transformations, they established a third generation model shown as follows:

$$\ddot{X}_{n+1}(t+\Delta t) = \frac{\propto_o}{x_n(t)-x_{n+1}(t)}[\dot{X}_n(t) - \dot{X}_{n+1}(t)] \qquad (2.31)$$

$\propto_o$ = Sensitivity parameter (ft/sec)

And other terms are as previously defined.

As the speed of the vehicle platoon increases, the drivers of individual vehicles within the platoon are likely to be sensitive to the relative distance between them and the vehicle in front of them, an attempt was made therefore, to insert the speed in the car-following model. Thus, in the light of incorporating the speed of the following vehicle to the sensitivity term of the model, the fourth model was established as shown in the following equation:

$$\ddot{X}_{n+1}(t+\Delta t) = \frac{\alpha'[\dot{X}_{n+1}(t+\Delta t)]}{x_n(t)-x_{n+1}(t)}[\dot{X}_n(t) - \dot{X}_{n+1}(t)] \qquad (2.32)$$

Where:

α' = Constant (dimensionless)

x_n = Position of the lead vehicle

x_{n+1} = Position of the following vehicle

And other terms are as previously defined.

The final generalized car-following model was developed by raising the speed and distance headway components of the sensitivity term to certain exponents. The resulting model known as the general model of car-following theories is shown as follows:

$$\ddot{X}_{n+1}(t+\Delta t) = \frac{\alpha_{l,m}[\dot{X}_{n+1}(t+\Delta t)]^m}{[x_n(t)-x_{n+1}(t)]^l}[\dot{X}_n(t) - \dot{X}_{n+1}(t)] \tag{2.33}$$

Where:

l = the distance headway exponent

m = the speed exponent

And other terms are as previously defined.

When the traffic stream is moving in a steady state, it was found that the speed, flow and density of this traffic stream can be determined using equation 2.31 (Garber and Hoel, 2010). Apart from the General motor research team, some other researchers have undertaken several works regarding this important traffic issue. Pipes (1953) had previously postulated that vehicles are separated by a legal distance and time as shown in the following equation:

$$x_n - x_{n+1} = L + T\,(\dot{X}_{n+1}) \tag{2.34}$$

Where:

L = Legal distance headway between two vehicles

T = Legal time headway between two vehicles

The transformation of Equation 2.32 leads to the basic response-stimuli concept of the car-following theories, from which Pipes (1953) developed a model similar to Equation 2.29, shown as follows:

$$\ddot{X}_{n+1}(t+T) = \lambda[\dot{X}_n(t) - \dot{X}_{n+1}(t)] \tag{2.35}$$

Where:

λ = Sensitivity factor

According to May (1990) and Baerwald *et al*. (1976), the general expression of λ proposed by Gazis *et al*. (1961) is given in the following form

$$\lambda = a\frac{\dot{X}_{n+1}{}^{m}(t+T)}{[X_n(t)-X_{n+1}(t)]^l} \tag{2.36}$$

Inserting the expression of λ from Equation 2.34 into Equation 2.33, results into the general equation of car-following theory (Equation 2.31) developed by General Motors research group.

2.6 Method of Collecting Traffic Data

2.6.1 Method of collecting traffic volume data

Most studies of traffic related problems begin with the collection of a good foundation of the roadway and traffic conditions. Any study can only be as accurate as the data it is based on. For this reason it is important that all traffic studies make special efforts to be thorough and accurate in the collection of all traffic data (Gresham *et al*., 2002).

The most common types of data collected for the purposes of traffic engineering are vehicle volume and speed data. The method of measurement should be suitable for the various activities of traffic movements through providing data about a number of vehicles (traffic volume), types of vehicles (traffic composition), speed, density, direction and the distance that vehicle travels.

The two basic methods of collecting data are manual observation and automatic recording. Each has its use and effectiveness depending on the type of information needed for analysis. The traffic data is the basis of all analysis in a traffic impact study and careful consideration needs to be given to the locations, types of counts and duration of counts. Manual counts are typically used to gather data for determination of vehicle classification, turning movements, direction of travel, pedestrian movements, or vehicle occupancy. Automatic counts are typically used to gather data for determination of vehicle hourly patterns, daily or seasonal variations and growth trends, or annual traffic estimates. Hence, since Nigerian traffic is heterogeneous, a manual count is recommended for a classified counting as is applied in this study.

The count period should be representative of the time of day, day of month, and month of year for the study area. For example, counts at a summer resort would not be taken in January. The count period should avoid special event or compromising weather conditions (Sharma, 1994). Count periods may range from 5 minutes to 1 year. Typical count periods are 15 minutes or 2 hours for peak periods, 4 hours for morning and afternoon peaks, 6 hours for morning, midday, and afternoon peaks, and 12 hours for daytime periods (Robertson, 1994). For example, if a 2-hour peak period count is conducted; eight 15-minute counts would be required.

A manual count study includes three key steps:

(1.) Perform necessary office preparations.

(2.) Select proper observer location.

(3.) Label data sheets and record observations.

Office preparations start with a review of the purpose of the manual count. This type of information will help determine the type of equipment to use, the field procedures to follow, and the number of observers required. For example, an intersection with multiple approach lanes may require electronic counting boards and multiple observers.

Observers must be positioned where they have a clear view of the traffic. Observers should be positioned away from the edge of the roadway. If observers are positioned above ground level and clear of obstructions they usually have the best vantage point. Visual contact must be maintained if there are multiple observers at a site.

Table 2.5 is a typical example of a general guideline for determining the appropriate period to collect peak hour information. Many nearby land uses may influence peak times of a particular intersection. Care should be given to understand the surrounding land uses before scheduling peak hour counts over a limited time period. It is suggested that an investigation of the daily counts be conducted prior to collecting the peak-hour counts to allow a determination of a typical range of peak hour traffic movements on a roadway facility.

Table 2.5: Typical Peak Flow Traffic Hour

Land Use	Typical Peak Hour
Residential	7:00-9:00 am weekday
	4:00-6:00 pm weekday
	5:00-6:00 pm weekday
Regional Shopping center	2:30-3:30 pm Saturday
	12:30-1:30 pm Saturday
Office	7:00-9:00 am weekday
	4:00-6:00 pm weekday
Industrial	Varies
Recreational	Varies
Hospital	Varies based on shift changes
School	Varies based on school release times

Source: Gresham *et al*. (2002).

For most studies an AM and PM count will be sufficient. Saturday counts are sometimes needed near high shopping areas. Count periods should avoid special events and adverse weather conditions. A count interval of 15-minutes should be used for most studies (Gresham et al., 2002).

2.6.2 Method of collecting speed data

Speed measurements are most often taken at a point (or a short section) of road way under conditions of free flow. The intent is to determine the speeds that drivers select, unaffected by the existence of congestion. This information is used to determine general speed trends, to help determine reasonable speed limits, and to assess safety.

Methods of conducting speed studies are divided into two main categories: Manual and Automatic. Spot speeds may be estimated by manually measuring the time it takes a vehicle to travel between two defined points on the roadway a known distance apart (short distance), usually less than 90 m. Distance between two points is generally depending upon the average speed of traffic stream. Table 2.6 gives recommended study length (in meters) for various average stream speed ranges (in kmph) (Mathew, 2014). Following are the some methods to measure spot speed of vehicles in a traffic stream, in which the first two are manual methods and the others are automatic:

(i) Pavement markings: In this method, markings of pavement are placed across the road at each end of trap. Observer start and stops the watch as vehicle passes lines. In this method, minimum of two observers are required to collect the data, of which one is to stand at the starting point of the road section to start and stop the stop watch and other one stands at end point to give indication to stop the watch when vehicle passes the end line. Advantages of this method are that after the initial installation no set-up time is required, markings are easily renewed, and disadvantage of this is that substantial error can be introduced, and magnitude of

Table 2.6: Required Length of Road Section for Speed Study

Stream Speed(km/hr)	Length (m)

Below 15	30
15 – 25	60
Above 25	90

Source: Mathew (2014)

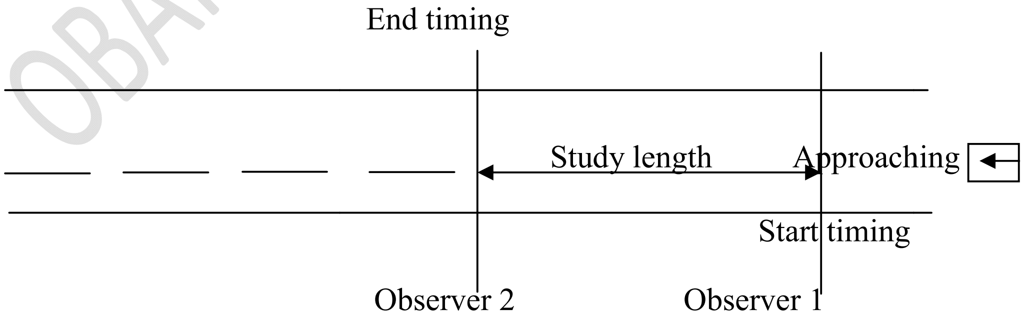


Figure 2.5: Pavement marking (Mathew, 2014)

error may change for substitute studies and this method is only applicable for low traffic conditions (Mathew, 2014). This is illustrated in Figure 2.5.

However, Roger *et al*. (2003) recommended that speed study can be manually carried out by usage of stop watches to time vehicles as they traverse an easily-recognized trap. While the trap can be marked with wide tape, natural or existing boundaries are often used. In such studies, an observer stands at the location of one of the trap boundaries (usually the entry boundary). This allows the observer to view the vehicles crossing the boundary without distortion. It guarantees however, that the observer views the other (usually the exit) boundary at an angle. This creates a systematic error called "parallax", which is illustrated in Figure 2.6. The observer sees the vehicle crossing the exit boundary that is d (in meters) from the entry boundary when the vehicle is actually only d_{eff} meter from the entry boundary.

While the exhibit shows parallax error occurring from the horizontal angle at which the observer sees the vehicle crossing the exit line, it can also occur in vertical dimension when measurements are taking from an overpass or other elevated position. The parallax error is easily corrected, as long as the angle of observation, φ, is known.

If φ is known, the distance d_{eff} may then be computed as:

$$d_{eff} = d_1 Tan\varphi \qquad (2.37)$$

The speed of an individual vehicle is then computed as:

$$S_i = \frac{d_{eff}}{t_i} \tag{2.38}$$

Where: S_i = speed of vehicle i, m/s

d_{eff} = actual distance over which travel time of vehicle i is measured, m

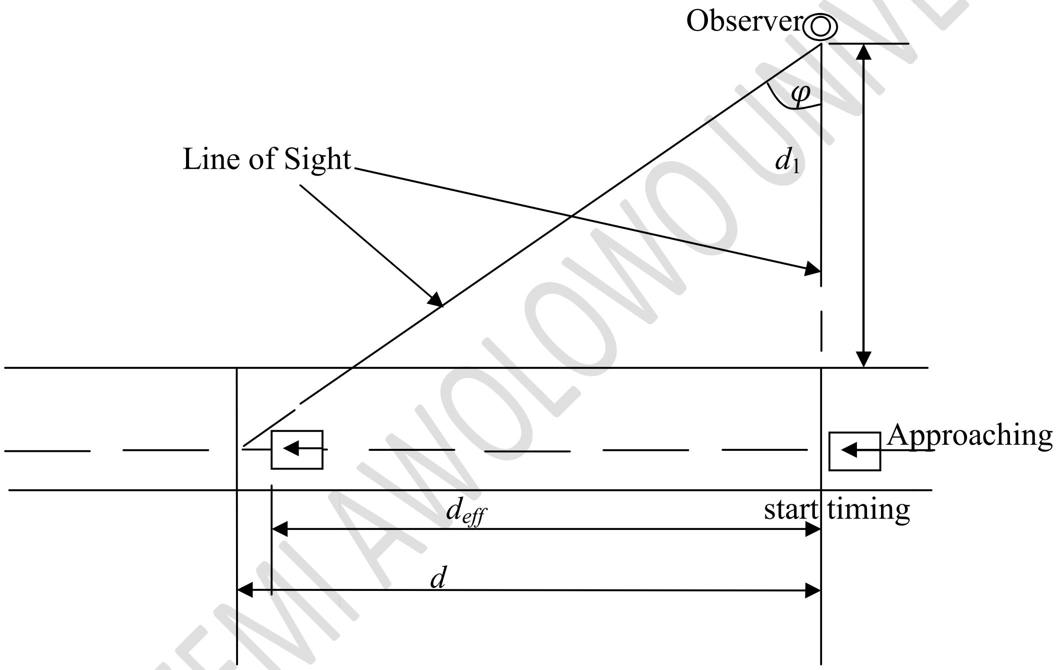


Figure 2.6: Parallax error illustrated (Roger *et al*., 2003)

t_i = times for vehicle i to traverse the distance d_{eff}, s

d_l= perpendicular distance of observer from line of travel, m

φ = angle between lines of sight, degrees

(ii) Enoscope or Mirror box: Enoscope consists of a simple open housing containing a mirror mounted on a tripod at the side of the road in such a way that an observer's line of sight turned through 90˚. The observer stands at one end of section and on the other end enoscope is placed to measure the time taken by the vehicle to cross the section (Figure 2.7). Advantages of this method are that it is simple and eliminates the errors due to parallax. However, the disadvantages of this method is that; considerable time is required to time each vehicle, which lengthen the study period and under heavy traffic condition it may be difficult to relate ostentatious to proper vehicle.

(iii) Road detector (Pressure contact strips): Pressure contact strips, either pneumatic or electric, can be used to avoid error due to parallax and due to manually starting and stopping the chronometer or stopwatch. This is the best method over short distance it gives quite relevant data and if it is connected through graphical recorder then it gives continuous data automatically.

(iv) Doppler-Principle metres (Radar): This is recently developed method, it automatically records speed, employs a radar transmitter- receiver unit. The apparatus transmits high frequency electromagnetic waves in a narrow beam towards the moving vehicle, and reflected waves changed their length depending up on the vehicles speed and returned to the receiving unit, through calibration gives directly spot speed of the vehicle.

(v) Electronic-Principle detectors (Photography): In this method a camera records the distance moved by a vehicle in a selected short time.

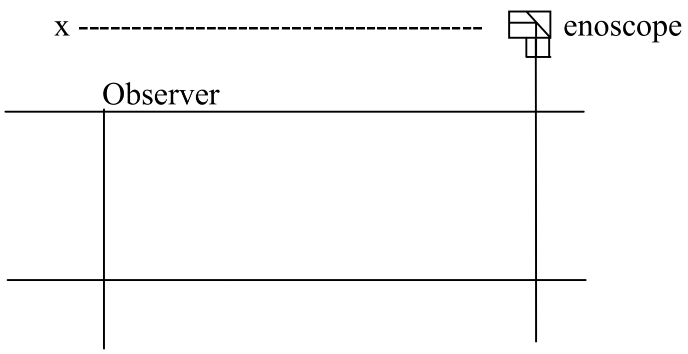


Figure 2.7: Enoscope method (Mathew, 2014)

The main advantage of this method is that, it gives a permanent record with 100% sample obtained. This method is quite expensive and generally used in developed cities.

Generally, sample sizes of 50 to 200 vehicles are taken. In that case, standard error of mean is usually under the acceptable limit. Since vehicle selection can be tricky, as observers have a natural tendency to record those vehicles that stand out in some way, such as fast cars, slow cars, trucks, platoon leaders, hence, a procedure may be used by selecting every 3rd, 4th or nth vehicle to record. The location at which speed measurements are taken must conform to the intentional purpose of the study.

2.7 Regression Analysis

Regression analysis is a statistical technique that is used to explore and define the relationships between two or more variables, where dependent variables also called output variables are related to independent variables called predictor variables (Montgomery and Runger, 2007). For the sake of this study, regression analysis will be used to determine some parameters of traffic flow models since their determination on site is difficult or even impracticable.

2.7.1 Procedure for developing regression model

The available values of variables under consideration are drawn in the graph called scatter plot in which the dependent variable is presented on the y-axis and independent variable on x-axis, in such a way that each pair of variable value is represented by a point plotted in a two-dimensional coordinate system. The reason for this step is to provide a way of visualizing the relationship between variables with the purpose to define a suitable model which provides the best fit to the observed data (Janvier, 2013).

Once the form of the model is obtained, the parameters needed must be determined. These parameters are estimated by the method of least squares which consists of minimizing the discrepancies between observed values and estimated values of the output variable.

Let y be a dependent variable and x an independent variable, it is further assumed that a linear relationship exists between y and x, as shown in Figure 2.8.

Since the mean of y is a linear function of x, and that actual observed values of y do not coincide to the straight line, an ad-hoc method to generalize this to a probabilistic linear model is to assume that the expected value of y is a linear function of x, and that the determination of the actual value of y for a fixed value of x can be given as a linear model plus a random error term as shown as follows:

$$y = \beta_0 + \beta_1 x + \varepsilon \qquad (2.39)$$

Where: β_0 and β_1= regression coefficients, and ε = random error

Using the above equation, each observation i can be expressed as:

$$y_i = \beta_0 + \beta_1 x_i + \varepsilon_i \qquad (2.40)$$

Where: y_i = the observed value of dependent variable for ith observation

The estimated or fitted value of y for each value of x from the line is therefore given by the following formula:

$$\hat{y} = \hat{\beta}_o + \hat{\beta}_1 x_1 \qquad (2.41)$$

Where: $\hat{y}$= the estimated value of the dependent variable for ith observation

The residual (e_i) which is the vertical difference between the observed value and estimated value of the output variable is therefore an estimate of vertical deviation obtained from the prediction of the actual value for the particular observation i, (Janvier, 2013).

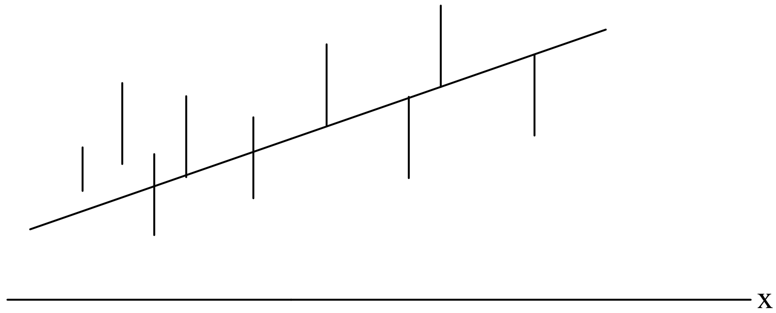


Figure 2.8: Deviations of the Data from the Estimated Regression Model (Montgomery and Runger, 2007)

Thus, it is given by the following formula:

$$e_i = y_i - \hat{y}_i \tag{2.42}$$

Box *et al*. (1978) and (Montgomery and Runger, 2007) stated that, the best fitting model is obtained when the sum of the square of errors between observed values and the estimated values of output variables from the model is minimized, that is, when sum of residuals is minimized. The sum of squares of residuals can be determined as follows:

$$SSE = \sum_{i=1}^{n}(e_i)^2 = \sum_{i=1}^{n}(y_i - \hat{y}_i)^2 \tag{2.43}$$

To compute $\hat{\beta}_o$ and $\hat{\beta}_1$, the following procedure is recommended**.**

$$\bar{x} = \frac{\sum x}{n} \tag{2.44}$$

$$\bar{y} = \frac{\sum y}{n} \quad (2.45)$$

$$SS_{xy} = \sum xy - \frac{(\sum x)(\sum y)}{n} \quad (2.46)$$

$$SS_{xx} = \sum x^2 - \frac{(\sum x)^2}{n} \quad (2.47)$$

$$SS_{yy} = \sum y^2 - \frac{(\sum y)^2}{n} \quad (2.48)$$

To find the regression line, β_0 and β_1 are computed as:

$$\hat{\beta}_1 = \frac{SS_{xy}}{SS_{xx}} \quad (2.49)$$

$$\hat{\beta}_0 = \bar{y} - \hat{\beta}_1\bar{x} \quad (2.50)$$

As seen in the Figure 2.8, the estimates of $\boldsymbol{\hat{\beta}_1}$ and $\boldsymbol{\hat{\beta}_0}$ give rise to a line which is considered at a certain extent as a best fit for the given data. The estimated or fitted value of y for each value of x from the line is therefore given by the following formula:

$$\hat{y}_i = \hat{\beta}_0 + \hat{\beta}_1 x_i \quad (2.51)$$

Where: $\hat{y}_i$ = the estimated value of the dependent variable for ith observation

The above equation represents the regression model with dependent variable $\hat{y}$ and independent variable $\hat{x}_i$.

2.7.2 Evaluation of regression model

A useful way of seeing how good the regression line is by seeing how big the sum of squares (SS) due to regression is or equivalently how small the sum of squares due to error in regression is. One way of measuring this can be done by calculating:

$$R^2 = \frac{\text{SS due to regression}}{\text{SS total variation}}$$

Usually R^2 is multiplied by 100 to represent the percentage variation explained by the variables. Once the least squares line has been obtained, it is natural to examine how effectively the line summarizes the relationship between x and y. The first question that has to be answered is, if the line is an appropriate way to summarize the relationship. In order to answer this question, the coefficient of determination R^2, will be calculated. The coefficient of determination, R^2, is determined thus:

$$R^2 = \frac{{SS_{xy}}^2}{SS_{xx}SS_{yy}} \quad (2.52)$$

It is noteworthy to indicate that the values of R^2 vary between 0 and 1. For models that fit well the available data, R^2 is near 1, otherwise, the values of R^2 are close to 0.For instance, if the computed value of R^2 for a set of data is x, this implies that the model explains x % of the total variation of data Janvier (2013). Table 2.7 illustrates the classification of goodness of fit by statistical parameters.

Table 2.7: Classification of Goodness of Fit by Statistical Parameters

Criteria	R^2	S_e/S_y
Excellent	>0.90	<0.35
Good	0.70-0.89	0.36-0.55
Fair	0.40-0.69	0.56-0.75
Poor	0.20-0.39	0.76-0.90
Very Poor	<0.19	>0.90

Source: Witczak et al. (2002)

However, while the R-squared provides an estimate of the strength of the relationship between the model and the response variable, it does not provide a formal hypothesis test for this relationship. The F-test of overall significance determines whether this relationship is statistically significant or not Witczak et al. (2002).

The hypotheses of F-test of overall significance are:

(1) Null hypothesis (H_o): There is no lack of linear fit

(2) Alternative hypothesis (H_1): There is lack of linear fit

It is based on these hypotheses that the adequacy of the model will be determined. Thus, important conclusion would be drawn by comparatively considering the P-value and the significance level ($\alpha \approx 0.05$).

If P-value > 0.05, H_o is rejected in favour of H_1. This implies that the model is not truly linear as proposed.

On the other hand, if P-value < 0.05, H_o is accepted. This implies that the model is truly linear Witczak et al. (2002).

CHAPTER THREE

MATERIALS AND METHODS

3.1 Overview

The first step in identifying traffic engineering measures appropriate in the development of traffic improvement programme is to collect and analyze information on traffic flow characteristics. This was done by first undertaking a reconnaissance survey of the road to enable decisions to be made as to the extent and the actual traffic surveys to be conducted. This study was carried out through a series of field data collection from a selected major urban road (divided into two segments) in Ile-Ife, a city in Osun State, South-West of Nigeria. The data were collected by means of manual techniques. During all data collection sessions, the observers were positioned at some distance away from the highway segments to obtain sufficient segment lengths, and prevent the drivers from changing their driving habits. The data collected for average travel speed (U_s) and traffic flow rate (Q) were used to formulate a regression model showing speed-flow-density relationships. The developed model was then evaluated using the coefficient of determination approach (R^2 approach) and F-test.

3.2 Site Selection

With the aim of obtaining accurate speed-flow-density curves and to meet the objectives of this study, it was necessary to obtain representative data that covered the full speed-flow-density domain. To this end, it was needed to find appropriate sections where bumper-to-bumper conditions were reached in the morning and evening, while it was also important to choose sufficient time-periods for study to ensure that all the degrees of congestion were covered. Therefore, the selected road was considered as two segments where the criteria were satisfied for both morning and afternoon peaks, and were therefore selected and used in this study. The appropriate sections where bumper-to-bumper conditions were observed for both morning and afternoon peaks were selected for each segment of the road.

3.3 Site Description

As previously mentioned, the data collection process was undertaken from the sections located on an urban two-lane two-way road located between Post Office and Teaching Hospital in Ile Ife. Figure 3.1 shows the map of the study site segments and surrounding areas.

Study Site 1: Post Office to Sabo Junction

This segment is located between Post Office and Sabo junction. This segment is 1.055km long with a lane width of 3.65m and a shoulder of approximately 1.85m width. A section (located between chainage 0+974 and 1+034) of this segment was selected for observation of vehicles and collection of traffic data. Plate 3.1 provides a screenshot of the study section.

Study Site 2: Sabo Junction to Teaching Hospital

This segment is located between Sabo junction and Teaching Hospital. This segment is 2.685km long with a lane width of 3.65m and an unpaved shoulder of approximately 5m width. A section (located between chainage 3+631 and 3+691) of this segment was also selected for observation of vehicles and collection of traffic data. Plate 3.2 provides a screenshot of the study section.

3.4 Method

3.4.1 Introduction

The method engaged in this study involved the collection of geometric data (road length and road width) of the road with the aid of the wheel counter (Plate 3.3), collection of speed data, reduction and processing of the speed data, collection of traffic volume data, reduction and

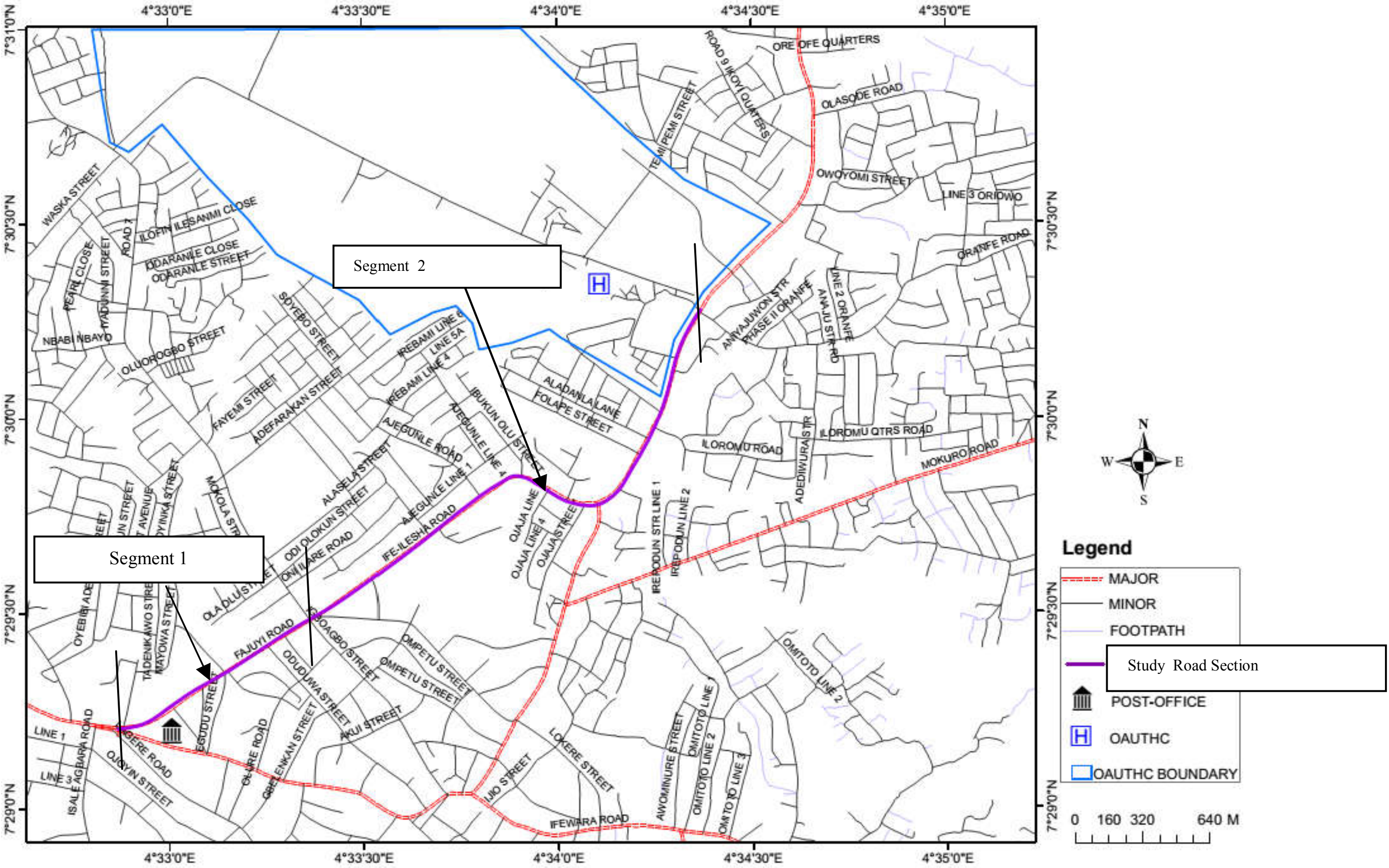


Figure 3.1: Locations of the study sites

Plate 3.1: A Screen Shot of Segment 1

Plate 3.2: A Screen Shot of Segment 2

processing of traffic volume data, computation of traffic density, model development, and evaluation of the model. An observation of the traffic flow on each study section of the road segments during morning peak conditions warranted a study period of about 3 hours (06:30-09:30), as well a study period of 2hours (04:00-06:00) for evening peak condition, for 7days (Monday-Sunday), consecutively. Distinction was made between passenger cars, cycles, trucks, and buses. During analysis for each section, adjustment factors (Nigerian Highway Design Manual, 2006) were used to convert heavy vehicles (trucks and buses) to passenger car equivalents (pcu's).

3.4.2 Collection of speed data

The aim of the speed study was to determine the average travel speed of vehicles traversing a section of roadway. This information was collected along with the traffic flow data with 15-minute increments, in order to establish the speed-flow relationship. The procedure for collecting the speed data is as follow:

(i.) A 60m section of roadway was marked for each segment of the road as an observation point, where vehicles were observed and data were collected.

(ii.) An observer was then posted at each end of the section. With the certainty that a person standing at one end of the section is visible to the one standing at the other end.

(iii.) When a vehicle passes an observer at one end of the section, the other observer signals the person at the other end of the section. The first person starts counting the time when the vehicle enters the entry point and stops the time when the vehicle reaches the exit point of the section where the second person is located. This time is recorded on the Traffic Speed Data Recording Sheet as shown in Appendix 2.

Plate 3.3: Observer taking measurement of the road

(iv.) In order to ensure accuracy and orderliness, data were collected for every 4th passenger car that traverses the section. This was been done repeatedly throughout the period of observation.

(v.) The procedure was carried out repeatedly for both morning (06:30-09:30) and evening (04:00-06:00) peaks for 7days (Monday-Sunday).

3.4.3 Reduction and processing of speed data

After collection of travel time of vehicles for every 15-minutes intervals of the period of observation, the average travel speed of vehicles for each of the 15-minutes interval were computed by:

a. Determining the average travel time(t_a) of vehicles for each of the 15-minutes;

$$t_a = \frac{\sum_1^n t_i}{n}, \text{ in seconds} \tag{3.1}$$

Where t_i = travel time for every i^{th} vehicle observed in a 15-minute,

n = total numbers of observed vehicles within the 15minutes,

i = 1,2,3,……………………, n

a. Dividing the average travel time, t_a (in seconds) by 3600 (the number of seconds in an hour). This give t_a (in hours).

b. Dividing the length of the section, L (60meters) by 1000 (number of meters in a kilometer). This gave L in kilometer.

c. Dividing the length of the section, L (in kilometer) by average travel time, t_a (in kilometer). This gave the space mean speed (U_s) in km/hr at each 15-minutes interval.

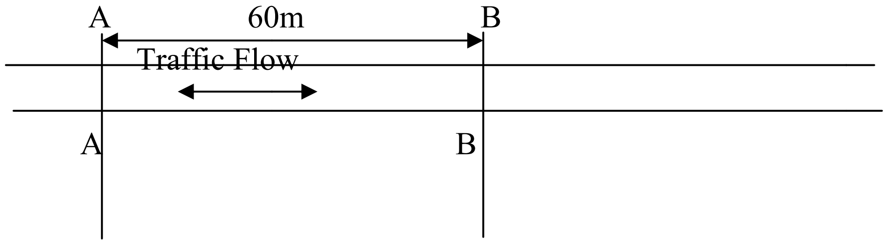


Figure 3.2: 60 m marking on each segment of the road

d. Computing the aggregate space mean speed (U_{agg}) for every 15-minutes interval in each day for the two segments. This was computed as follow:

$$U_{agg} = \frac{U_{s1} + U_{s2}}{2} \tag{3.2}$$

Where; U_{agg} = aggregate space mean speed for every 15-minutes interval

U_{s1} = space mean speed of segment – 1 for every 15-minutes interval

U_{s2} = space mean speed of segment – 2 for every 15-minutes interval

f. Computing the average space mean speed (Usa) for every corresponding 15-minutes intervals for the seven days of observation:

$$U_{sa} = \frac{\sum_1^7 U_{s(1,2)i}}{7} \tag{3.3}$$

Where;

U_{sa} = average space mean speed, in km/hr,

$U_{s(1,2)i}$ = aggregate space mean speed for the two segments for every 15- minutes interval in each day.

i = day-1, day-2, day-3, …………., day-7

3.4.4 Collection of traffic flow rate data

This section explains how the traffic volume count was conducted with sufficient accuracy to allow the data to serve the purpose of this study. As the name implies, the purpose of the count is to determine the flow rate of traffic (cars, buses, motorcycles and trucks) traveling a specific section of the road. Following are the steps involved in conducting the traffic count.

(i.) An observer was required per segment for the traffic volume study. Each observer stood at the entry point of the selected section of the segment.

(ii.) Data were collected simultaneously with the speed data on the same selected section of the road. The observer recorded every vehicle in both directions of the section, classifying them to either passenger car (pc), buses (bs), trucks (trk), or cycles (cy). Plate 3.5 shows various classes of vehicles as they were been observed and distinguished.

(iii.) A 15-minute increment was used for collection and recording of traffic flow as it was done for speed data. As the study began at 6:30 am, then the numbers of vehicles travelling the road were counted from 6:30 to 6:45, 6:46-7:00, 7:01-7:15, etc. This was also applied during the evening(4:00pm – 6:00pm) observations.

(iv.) Hatch marks were used to record each vehicle. At the end of each 15-minute period the total of the hatch marks was computed and noted. Appendix 1 shows a copy of the traffic volume data sheet.

(v.) Steps (1) to (4) were carried out repeatedly for seven days (Monday to Sunday).

3.4.5 Reduction and processing of traffic flow data

After the collection of the traffic flow data, the following steps were followed to reduce the field data and process them to achieve the expected output required towards achieving the objectives of this study:

(i.) The heterogeneous classes of vehicles were converted to passenger car equivalent (pce) and the total volume of traffic for every 15-minutes increment was computed in terms of passenger car equivalent (pce).

(ii.) The traffic flow rate in pc/hr/ln for every 15-minutes increment was then computed using:

$$Q_i = \frac{4 \times Q_{15}}{n_l} \tag{3.4}$$

Where Q_i = traffic flow rate for each road segment, in pc/hr/ln
i = segment number (1,2)
Q_{15} = flow of traffic for 15-minutes increment, in pc

(a) Motorcycle

(b) Passenger Car

(c) Bus

Plate 3.4: Classes of Vehicles

n_l = number of lane

(iii.) The average traffic flow rate for the two segments for every 15-minutes increment of each day was computed using:

$$Q_a = \frac{\sum_1^m Q_i}{m} \quad (3.5)$$

Where Q_a = average traffic flow rate for every 15-minutes increment

Q_i = traffic flow rate for each road segment

i = 1, 2, 3, …………………., m

m = numbers of segments

(iv.) The average traffic flow rate for seven days was computed using:

$$Q = \frac{\sum_1^n (Q_a)_i}{n} \quad (3.6)$$

Where Q = average traffic flow rate for a seven-day count

Q_a= corresponding average traffic flow rate for every 15-minutes increment for each day

i = 1, 2, 3, 4, ………, n

n = number of day spent for the counting = 7 days

3.4.6 Computation of traffic density data

The traffic density (k) was gotten using the average traffic flow rate (Q) and average travel speed (U_{sa}) computed in the previous sections. This was been done using the conventional traffic flow equation relating speed, flow and density. That is;

$$k = \frac{Q(\frac{pc}{hr}/ln)}{U_{sa}(\frac{km}{hr})}\text{, pc/km/ln} \qquad (3.7)$$

This value was computed for every 15-minute increment.

3.4.7 Model development

Having gotten the data for speed, flow and density, linear regression models showing speed-flow-density relationships were then formulated using appropriate statistical method. The speed-density model was first developed since it represents the simplest form of the three models, showing a linear relationship. The speed-flow model and flow-density models were then developed from the speed-density model.

Speed-Density model

The resulting ***Speed-Density*** relationship is shown in the following equation:

$$\boldsymbol{U_s = U_f\left(1 - \frac{K}{K_j}\right) = U_f - \frac{U_f}{K_j}K}$$

Where:

U_s = Space mean speed(km/h)

U_f = Free flow speed (km/h)

K = Density (pc/km/ln)

K_j = Jam density (pc/km/ln)

In this case, the free flow speed (U_f) and the jam density (K_j) were found from the developed speed-density model.

Flow-Speed model

Since $Q = K.U$, the speed-flow model was then developed by applying this equation to the developed speed-density model as shown below:

$K = Q/U_s$

Substituting this in the speed-density model;

$$\boldsymbol{Q = K_j\,(U_s - \frac{U_s^2}{U_f}) = K_j \cdot U_s - \frac{K_j}{U_f}U_s^2}$$

Where:

Q= Flow rate, pc/hr/ln

U, K_j *and* U_f are defined as previously.

To determine the speed at maximum flow, $\mathrm{U_m} = \frac{U_\mathrm{f}}{2}$

Flow-Density model

The flow-density model was developed by substituting U_s= Q/K in the developed speed-density model. Thus;

$$\boldsymbol{U_s = Q/K = U_f\,(1 - \frac{K}{K_j}) = U_f - \frac{U_f}{K_j}K}$$

This implies; $\boldsymbol{Q = U_f \cdot K - \frac{U_f}{K_j}K^2}$

To find the density at maximum flow, $\mathrm{K_m} = \frac{K_\mathrm{j}}{2}$ (from eqn. 2.29)

3.4.8 Model evaluation

The models were evaluated using the coefficient of determination (R^2) approach. The closeness or deviation of the R^2 value from 1 is a determinant of the accuracy of the line of best of best fit, representing the fitted curve and the models. However, the values of R^2 does not indicate the adequacy of the models, hence the models were further assessed using the F-tests.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Overview

The first part of any traffic flow analysis process is data collection and the ability to be able to present the results in a format that is easily understandable. Since macroscopic traffic flow models require data for speed, flow and density, and the density cannot be easily obtained on the field, the traffic volume and average travel time data were utilized in order to compute the traffic density by applying the fundamental traffic flow equation.

4.2 Traffic Flow Data

4.2.1 Traffic composition

The traffic composition obtained in this study is as shown in Table 4.1 and graphically presented in Figure 4.1.

The result shows that motorcycle is the predominant means of transportation along the selected road, followed by buses, cars and trucks. This is typical of an urban city like Ile-Ife.

4.2.2 Traffic flow rate, Q

Table 4.2 shows the result of average traffic flow rate (pcu/h/ln) obtained for segment 1, segment 2 and the aggregate flow rate for the selected road. A plot of aggregate traffic flow rate and time interval is shown in Figure 4.2.

The result illustrates relevant traffic information which is of significance to traffic engineers and transportation planners.

Table 4.1: Traffic Composition for the Selected Road

Segment 1					
Vehicle Type	Car	Buses	Trucks	Cycles	Total
Composition	18%	25%	1%	56%	100%
Segment 2					
Vehicle Type	Car	Buses	Trucks	Cycles	Total
Composition	29%	25%	2%	44%	100%
Average					
Vehicle Type	Car	Buses	Trucks	Cycles	Total
Composition	23%	25%	2%	50%	100%

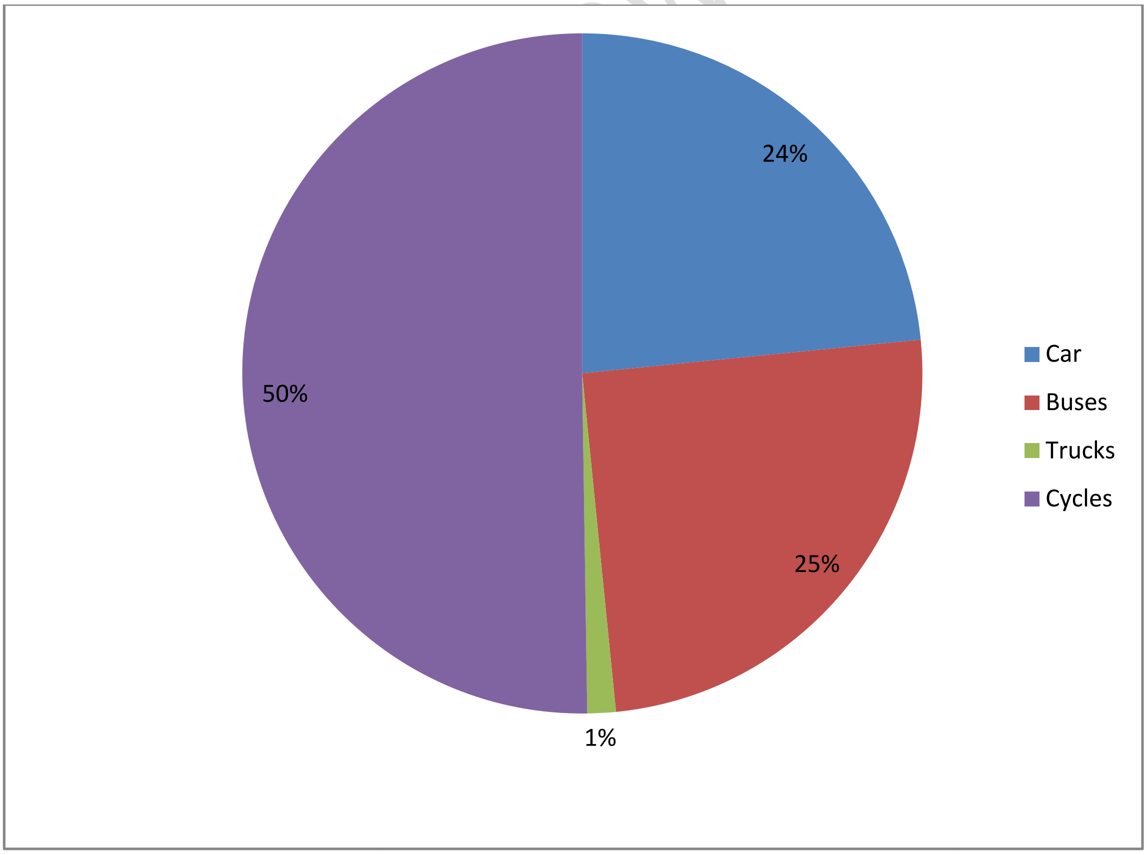

24%
25%
1%
50%
Car
Buses
Trucks
Cycles

Figure 4.1: Traffic Composition for the Selected Road

Table 4.2: Traffic Flow Rate for the Selected Road

Day	Time Interval	Traffic Flow Rate, Q (pcu/h/ln) Segment 1	Segment 2	Aggregate
	6:30-6:45	435	876	655
	6:46-7:00	610	602	606
	7:01-7:15	943	1258	1100
	7:16-7:30	734	2146	1440
	7:31-7:45	1024	2942	1983
	7:46-8:00	1008	4145	2576
	8:01-8:15	1126	2685	1905
Day 1	8:16-8:30	1152	2992	2072
	8:31-8:45	1158	3025	2091
	8:46-9:00	1096	2996	2046
	9:01-9:15	944	1093	1019
	9:16-9:30	938	1130	1034
	4:00-4:15	554	1334	944
	4:16-4:30	999	1865	1432
	4:31-4:45	1003	621	812
	4:46-5:00	1044	851	947
	5:01-5:15	1105	882	994
	5:16-5:30	1019	710	864
	5:31-5:45	1194	722	958
	5:46-6:00	1230	819	1024
	6:30-6:45	589	579	584

	6:46-7:00	840	964	902
	7:01-7:15	1610	1201	1405
	7:16-7:30	446	1645	1046
	7:31-7:45	1335	1841	1588
	7:46-8:00	939	1633	1286
	8:01-8:15	1110	1599	1354
Day 2	8:16-8:30	1192	1964	1578
	8:31-8:45	1303	1490	1396
	8:46-9:00	1113	1430	1271
	9:01-9:15	1286	1411	1348
	9:16-9:30	1418	1159	1289
	4:00-4:15	600	1112	856
	4:16-4:30	727	974	851
	4:31-4:45	790	974	882
	4:46-5:00	595	1030	812
	5:01-5:15	596	953	774
	5:16-5:30	680	1276	978
	5:31-5:45	753	1174	964
	5:46-6:00	649	1193	921

Table 4.2: Traffic Flow Rate for the Selected Road (Contd.)

Day	Time Interval	Traffic Flow Rate, Q (pcu/h/ln)		
		Segment 1	Segment 2	Aggregate
	6:30-6:45	446	659	552
	6:46-7:00	407	1030	718
	7:01-7:15	619	1021	820
	7:16-7:30	623	1125	874
	7:31-7:45	757	1117	937
	7:46-8:00	722	963	842
	8:01-8:15	689	1060	875
Day 3	8:16-8:30	807	989	898
	8:31-8:45	647	1448	1048
	8:46-9:00	721	1195	958
	9:01-9:15	754	1132	943
	9:16-9:30	628	1190	909
	4:00-4:15	491	1011	751
	4:16-4:30	528	774	651
	4:31-4:45	659	672	666
	4:46-5:00	677	1291	984
	5:01-5:15	583	838	711

	5:16-5:30	441	1149	795
	5:31-5:45	525	955	740
	5:46-6:00	662	1305	984
	6:30-6:45	513	567	540
	6:46-7:00	772	822	797
	7:01-7:15	901	705	803
	7:16-7:30	831	919	875
	7:31-7:45	1194	1115	1154
	7:46-8:00	1045	980	1012
	8:01-8:15	1262	787	1025
Day 4	8:16-8:30	1336	933	1134
	8:31-8:45	1290	943	1117
	8:46-9:00	1018	791	904
	9:01-9:15	778	1049	913
	9:16-9:30	779	1154	967
	4:00-4:15	580	731	655
	4:16-4:30	684	986	835
	4:31-4:45	880	938	909
	4:46-5:00	1087	751	919
	5:01-5:15	496	923	709
	5:16-5:30	814	812	813
	5:31-5:45	669	804	736
	5:46-6:00	1475	781	1128

Table 4.2: Traffic Flow Rate for the Selected Road (Contd.)

Day	Time Interval	Traffic Flow Rate, Q (pcu/h/ln) Segment 1	Segment 2	Aggregate
	6:30-6:45	437	723	580
	6:46-7:00	773	765	769
	7:01-7:15	881	716	798
	7:16-7:30	1137	879	1008
	7:31-7:45	1284	596	940
	7:46-8:00	1273	858	1065
	8:01-8:15	1219	887	1053
Day 5	8:16-8:30	1234	990	1112
	8:31-8:45	765	1043	904
	8:46-9:00	700	962	831
	9:01-9:15	685	909	797
	9:16-9:30	754	880	817
	4:00-4:15	653	566	609

	4:16-4:30	758	589	673
	4:31-4:45	599	602	600
	4:46-5:00	743	646	695
	5:01-5:15	878	497	688
	5:16-5:30	701	454	577
	5:31-5:45	847	457	652
	5:46-6:00	526	532	529
	6:30-6:45	482	219	350
	6:46-7:00	637	212	424
	7:01-7:15	858	162	510
	7:16-7:30	847	239	543
	7:31-7:45	955	367	661
	7:46-8:00	634	245	440
	8:01-8:15	679	177	428
Day 6	8:16-8:30	659	244	452
	8:31-8:45	645	217	431
	8:46-9:00	496	320	408
	9:01-9:15	702	247	475
	9:16-9:30	392	296	344
	4:00-4:15	434	338	386
	4:16-4:30	574	351	462
	4:31-4:45	584	309	447
	4:46-5:00	513	465	489
	5:01-5:15	364	363	363
	5:16-5:30	404	231	317
	5:31-5:45	322	359	341
	5:46-6:00	344	521	432

Table 4.2: Traffic Flow Rate for the Selected Road (Contd.)

	Time	Traffic Flow Rate, Q (pcu/h/ln)		
Day	Interval	Segment 1	Segment 2	Aggregate
	6:30-6:45	570	589	580
	6:46-7:00	758	632	695
	7:01-7:15	854	710	782
	7:16-7:30	958	756	857
	7:31-7:45	826	824	825
	7:46-8:00	992	853	923
	8:01-8:15	713	786	749
Day 7	8:16-8:30	781	731	756
	8:31-8:45	749	736	743

8:46-9:00	805	576	691
9:01-9:15	699	680	689
9:16-9:30	636	754	695
4:00-4:15	496	848	672
4:16-4:30	579	912	745
4:31-4:45	570	890	730
4:46-5:00	694	952	823
5:01-5:15	699	916	807
5:16-5:30	718	957	838
5:31-5:45	546	954	750
5:46-6:00	669	1066	868

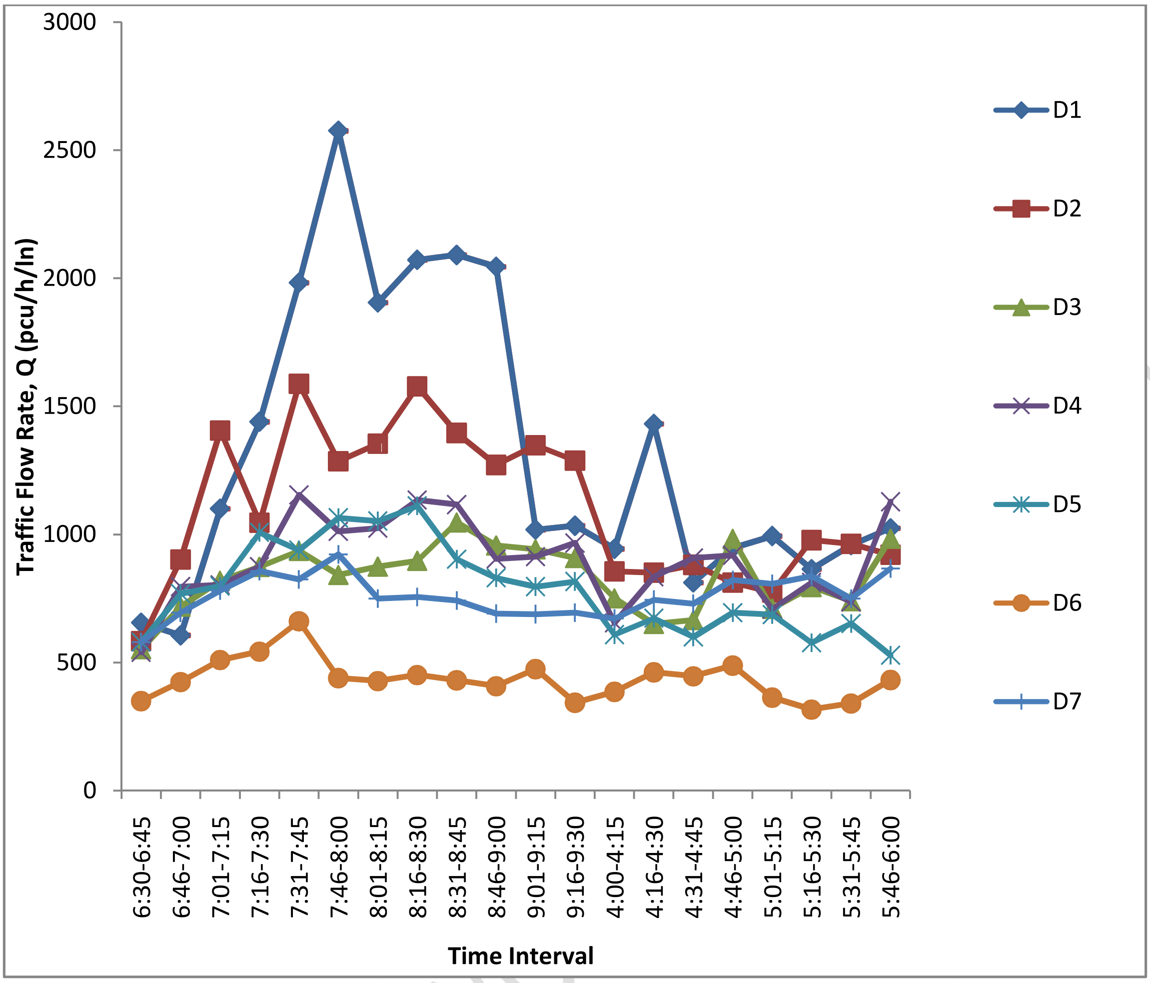


Figure 4.2: Aggregate traffic flow rate for the selected road

The traffic information were classified as: the **'early morning rise traffic, from 6:30 am to 7:00 am'**, when few vehicles were using the road at the early hour of the morning, the **'morning peak traffic, from 7:00 am to 9:30 am'**, when the travel demand was at its peak in the morning, and the **'evening peak traffic, from 4:00 pm to 6:00 pm'** when the travel demand was at its peak in the evening. The information is as follows:

(i.) 'Early morning rise traffic'

'Day 1' shows a maximum 'early morning rise traffic' of 655 pcu/h/ln (between 6:30 am and 6:45 am) and a minimum 'early morning rise traffic' of 606 pcu/h/ln (between 6:46 am and 7:00 am). 'Day 2' shows a maximum 'early morning rise traffic' of 902 pcu/h/ln (between 6:46 am and 7:00 am) and a minimum 'early morning rise traffic' of 584 pcu/h/ln (between 6:30 am and 6:45 am). 'Day 3' shows a maximum 'early morning rise traffic' of 718 pcu/h/ln (between 6:46 am and 7:00 am) and a minimum 'early morning rise traffic' of 552 pcu/h/ln (between 6:30 am and 6:45 am). 'Day 4' shows a maximum 'early morning rise traffic' of 797 pcu/h/ln (between 6:46 am and 7:00 am) and a minimum 'early morning rise traffic' of 540 pcu/h/ln (between 6:30 am and 6:45 am). 'Day 5' shows a maximum 'early morning rise traffic' of 769 pcu/h/ln (between 6:46 am and 7:00 am) and a minimum 'early morning rise traffic' of 580 pcu/h/ln (between 6:30 am and 6:45 am). 'Day 6' shows a maximum 'early morning rise traffic' of 424 pcu/h/ln (between 6:46 am and 7:00 am) and a minimum 'early morning rise traffic' of 350 pcu/h/ln (between 6:30 am and 6:45 am). 'Day 7' shows a maximum 'early morning rise traffic' of 695 pcu/h/ln (between 6:46 am and 7:00 am) and a minimum 'early morning rise traffic' of 580 pcu/h/ln (between 6:30 am and 6:45 am).

In summary, the highest and the lowest early morning rise traffic throughout the week were 902 pcu/h/ln on 'Day 2' and the 350 pcu/h/ln on 'Day 6', respectively. This information is essential to traffic analyst and designers when considering the condition of the road at the early hour of the day when fewer vehicles are just using the road.

(ii.) 'Morning peak traffic'

The morning peak traffic condition shows that 'Day 1' has a maximum morning peak traffic of 2576 pcu/h/ln (between 7:46 am and 8:00 am) and a minimum traffic of 1019 pcu/h/ln (between 9:01am and 9:15 am). 'Day 2' has maximum morning peak traffic of 1588 pcu/h/ln (between 7:31 am and 7:45 am) and a minimum traffic of 1046 pcu/h/ln (between 7:16 am and 7:30 am). 'Day 3' has maximum morning peak traffic of 1048 pcu/h/ln (between 8:31 am and 8:45 am) and a minimum traffic of 820 pcu/h/ln (between 7:01 am and 7:15 am). 'Day 4' has maximum morning peak traffic of 1154 pcu/h/ln (between 7:31 am and 7:45 am) and a minimum traffic of 803 pcu/h/ln (between 7:01 am and 7:15 am). 'Day 5' has maximum morning peak traffic of 1112 pcu/h/ln (between 8:31 am and 8:45 am) and a minimum traffic of 797 pcu/h/ln (between 9:16 am and 9:30 am). 'Day 6' has maximum morning peak traffic of 661 pcu/h/ln (between 7:31 am and 7:45 am) and a minimum traffic of 408 pcu/h/ln (between 8:46 am and 9:00 am). 'Day 7' has maximum morning peak traffic of 923 pcu/h/ln (between 7:46 am and 8:00 am) and a minimum traffic of 691 pcu/h/ln (between 8:46 am and 9:00 am).

In summary, the highest and the lowest morning peak traffic throughout the week was found to be 2576 pcu/h/ln (between 7:46 am and 8:00 am) on 'Day 1' and 408 pcu/h/ln (between 8:46 am and 9:00 am) on 'Day 6', respectively. The earlier period depicts the time when travel demand was at its peak in the morning and the later period depicts the time when travel demand was at its lowest in the morning.

(iii.) 'Evening peak traffic'

The evening peak traffic condition shows that 'Day 1' has a maximum evening peak traffic of 1432 pcu/h/ln (between 4:16 pm and 4:30 pm) and a minimum evening peak traffic of 812 pcu/h/ln (between 4:31 pm and 4:45 pm). 'Day 2' has maximum evening peak traffic of 978 pcu/h/ln (between 5:16 pm and 5:30 pm) and minimum evening peak traffic of 812 pcu/h/ln (between 4:46 pm and 5:00 pm). 'Day 3' has maximum evening peak traffic of 984 pcu/h/ln (between 4:46 pm and 5:00 pm) and minimum evening peak traffic of 651 pcu/h/ln (between 4:16 pm and 4:30 pm). 'Day 4' has maximum evening peak traffic of 1128 pcu/h/ln (between 5:46 pm and 6:00 pm) and a minimum evening peak traffic of 655 pcu/h/ln (between 4:00 pm and 4:15 pm). 'Day 5' has maximum evening peak traffic of 695 pcu/h/ln (between 4:46 pm and 5:00 pm) and minimum evening peak traffic of 529 pcu/h/ln (between 5:46 pm and 6:00 pm). 'Day 6' has maximum evening peak traffic of 489 pcu/h/ln (between 4:46 pm and 5:00 pm) and minimum evening peak traffic of 317 pcu/h/ln (between 5:16 pm and 5:30 pm). 'Day 7' has maximum evening peak traffic of 868 pcu/h/ln (between 5:46 pm and 6:00 pm) and minimum evening peak traffic of 730 pcu/h/ln (between 4:31 pm and 4:45 pm).

In summary, the highest and the lowest evening peak traffic throughout the week was found to be 1432 pcu/h/ln (between 4:16 pm and 4:30 pm) on 'Day 1' and 317 pcu/h/ln (between 5:16 pm and 5:30 pm) on 'Day 6', respectively. The earlier period depicts the time when travel demand was at its peak in the evening and the later period depicts the time when travel demand was at its lowest in the evening.

4.2.3 Travel speed

The average travel speed (km/hr) of vehicles on segment 1 and 2 were computed and the result is as shown in Table 4.3 and illustrated in Figure 4.3.

Similarly, as stated in section 4.2.2 for traffic flow rate, the result for average travel speed illustrates relevant traffic information which is also of significance to traffic analysis and transportation planning.

Table 4.3: Average Travel Speed, Us (km/h) along the Selected Road

Day	Interval	Segment 1	Segment 2	Aggregate
	6:30-6:45	24.00	42.86	33.43
	6:46-7:00	15.79	37.50	26.64
	7:01-7:15	11.76	40.00	25.88

Day 1	7:16-7:30	23.08	37.50	30.29
	7:31-7:45	19.35	35.29	27.32
	7:46-8:00	18.18	37.50	27.84
	8:01-8:15	15.79	30.00	22.89
	8:16-8:30	9.09	33.33	21.21
	8:31-8:45	14.29	35.29	24.79
	8:46-9:00	21.43	37.50	29.46
	9:01-9:15	18.75	37.50	28.13
	9:16-9:30	20.00	35.29	27.65
	4:00-4:15	16.67	40.00	28.33
	4:16-4:30	8.70	33.33	21.01
	4:31-4:45	17.14	35.29	26.22
	4:46-5:00	16.67	37.50	27.08
	5:01-5:15	17.14	37.50	27.32
	5:16-5:30	16.67	40.00	28.33
	5:31-5:45	17.14	40.00	33.52
	5:46-6:00	16.67	40.00	28.33
Day 2	6:30-6:45	16.67	50.00	33.33
	6:46-7:00	16.67	50.00	33.33
	7:01-7:15	18.75	50.00	34.38
	7:16-7:30	14.63	46.15	30.39
	7:31-7:45	15.79	46.15	30.97
	7:46-8:00	15.00	40.00	27.50
	8:01-8:15	15.38	40.00	27.69
	8:16-8:30	8.96	50.00	29.48
	8:31-8:45	9.09	50.00	29.55
	8:46-9:00	13.04	50.00	31.52
	9:01-9:15	24.00	46.15	35.08
	9:16-9:30	26.09	50.00	38.04
	4:00-4:15	23.08	50.00	36.54
	4:16-4:30	23.08	46.15	34.62
	4:31-4:45	25.00	42.86	33.93
	4:46-5:00	23.08	50.00	36.54
	5:01-5:15	23.08	46.15	34.62
	5:16-5:30	24.00	50.00	37.00
	5:31-5:45	25.00	46.15	35.58
	5:46-6:00	25.00	46.15	35.58

Table 4.3: Average Travel Speed, Us (km/h) along the Selected Road (contd.)

Day	Interval	Segment 1	Segment 2	Aggregate
	6:30-6:45	30.00	46.15	38.08
	6:46-7:00	28.57	50.00	39.29
	7:01-7:15	30.00	46.15	38.08
	7:16-7:30	30.00	46.15	38.08
	7:31-7:45	28.57	50.00	39.29
	7:46-8:00	28.57	46.15	37.36
	8:01-8:15	27.27	42.86	35.06
	8:16-8:30	31.58	46.15	38.87
	8:31-8:45	27.27	46.15	36.71
Day 3	8:46-9:00	27.27	46.15	36.71
	9:01-9:15	25.86	42.86	34.36
	9:16-9:30	30.00	46.15	38.08
	4:00-4:15	27.27	42.86	35.06
	4:16-4:30	26.09	42.86	34.47
	4:31-4:45	27.27	42.86	35.06
	4:46-5:00	31.58	37.50	34.54
	5:01-5:15	27.27	42.86	35.06
	5:16-5:30	24.00	40.00	32.00
	5:31-5:45	24.00	42.86	33.43
	5:46-6:00	28.57	42.86	35.71
	6:30-6:45	27.27	46.15	36.71
	6:46-7:00	27.27	42.86	35.06
	7:01-7:15	22.22	50.00	36.11
	7:16-7:30	25.00	46.15	35.58
	7:31-7:45	16.22	46.15	31.19
	7:46-8:00	26.09	46.15	36.12
	8:01-8:15	25.00	46.15	35.58
	8:16-8:30	20.69	46.15	33.42
	8:31-8:45	23.08	46.15	34.62
	8:46-9:00	27.27	50.00	38.64
	9:01-9:15	25.00	42.86	33.93
Day 4	9:16-9:30	28.57	50.00	39.29
	4:00-4:15	25.00	42.86	33.93
	4:16-4:30	28.57	42.86	35.71
	4:31-4:45	31.58	42.86	37.22
	4:46-5:00	25.00	46.15	35.58
	5:01-5:15	26.09	46.15	36.12
	5:16-5:30	19.35	46.15	32.75
	5:31-5:45	25.00	50.00	37.50

	5:46-6:00	28.57	46.15	37.36

Table 4.3: Average Travel Speed, Us (km/h) along the Selected Road (contd.)

Day	Interval	Segment 1	Segment 2	Aggregate
	6:30-6:45	25.00	50.00	37.50
	6:46-7:00	30.00	50.00	40.00
	7:01-7:15	28.57	54.55	41.56
	7:16-7:30	28.57	46.15	37.36
	7:31-7:45	27.27	46.15	36.71
	7:46-8:00	22.22	42.86	32.54
	8:01-8:15	18.75	46.15	32.45
	8:16-8:30	23.08	50.00	36.54
	8:31-8:45	22.22	46.15	34.19
	8:46-9:00	27.27	54.55	40.91
Day 5	9:01-9:15	28.57	50.00	39.29
	9:16-9:30	26.09	54.55	40.32
	4:00-4:15	26.09	50.00	38.04
	4:16-4:30	31.58	46.15	38.87
	4:31-4:45	35.29	50.00	42.65
	4:46-5:00	30.00	46.15	38.08
	5:01-5:15	27.27	46.15	36.71
	5:16-5:30	26.91	50.00	38.45
	5:31-5:45	31.58	54.55	43.06
	5:46-6:00	31.58	46.15	38.87
	6:30-6:45	30.00	46.15	38.08
	6:46-7:00	31.58	54.55	43.06
	7:01-7:15	28.57	54.55	41.56
	7:16-7:30	20.69	54.55	37.62
	7:31-7:45	28.57	50.00	39.29
	7:46-8:00	26.09	54.55	40.32
	8:01-8:15	28.57	54.55	41.56
	8:16-8:30	27.27	54.55	40.91
	8:31-8:45	28.57	54.55	41.56
Day 6	8:46-9:00	28.57	54.55	41.56
	9:01-9:15	28.57	46.15	37.36
	9:16-9:30	27.27	54.55	40.91
	4:00-4:15	26.09	46.15	36.12
	4:16-4:30	30.00	46.15	38.08

	4:31-4:45	26.09	46.15	36.12
	4:46-5:00	26.09	46.15	36.12
	5:01-5:15	30.00	42.86	36.43
	5:16-5:30	27.27	46.15	36.71
	5:31-5:45	30.00	46.15	38.08
	5:46-6:00	27.27	42.86	35.06

Table 4.3: Average Travel Speed, Us (km/h) along the Selected Road (contd.)

Day	Interval	Segment 1	Segment 2	Aggregate
	6:30-6:45	28.57	60.00	44.29
	6:46-7:00	30.00	40.00	35.00
	7:01-7:15	30.00	46.15	38.08
	7:16-7:30	28.57	46.15	37.36
	7:31-7:45	28.57	46.15	37.36
	7:46-8:00	25.00	42.86	33.93
	8:01-8:15	30.00	46.15	38.08
	8:16-8:30	27.27	40.00	33.64
	8:31-8:45	28.57	40.00	34.29
	8:46-9:00	30.00	46.15	38.08
Day 7	9:01-9:15	30.00	42.86	36.43
	9:16-9:30	33.33	42.86	38.10
	4:00-4:15	25.00	46.15	35.58
	4:16-4:30	27.27	46.15	36.71
	4:31-4:45	27.27	46.15	36.71
	4:46-5:00	27.27	46.15	36.71
	5:01-5:15	28.57	46.15	37.36
	5:16-5:30	28.57	42.86	35.71
	5:31-5:45	27.27	50.00	38.64
	5:46-6:00	28.57	52.63	40.60

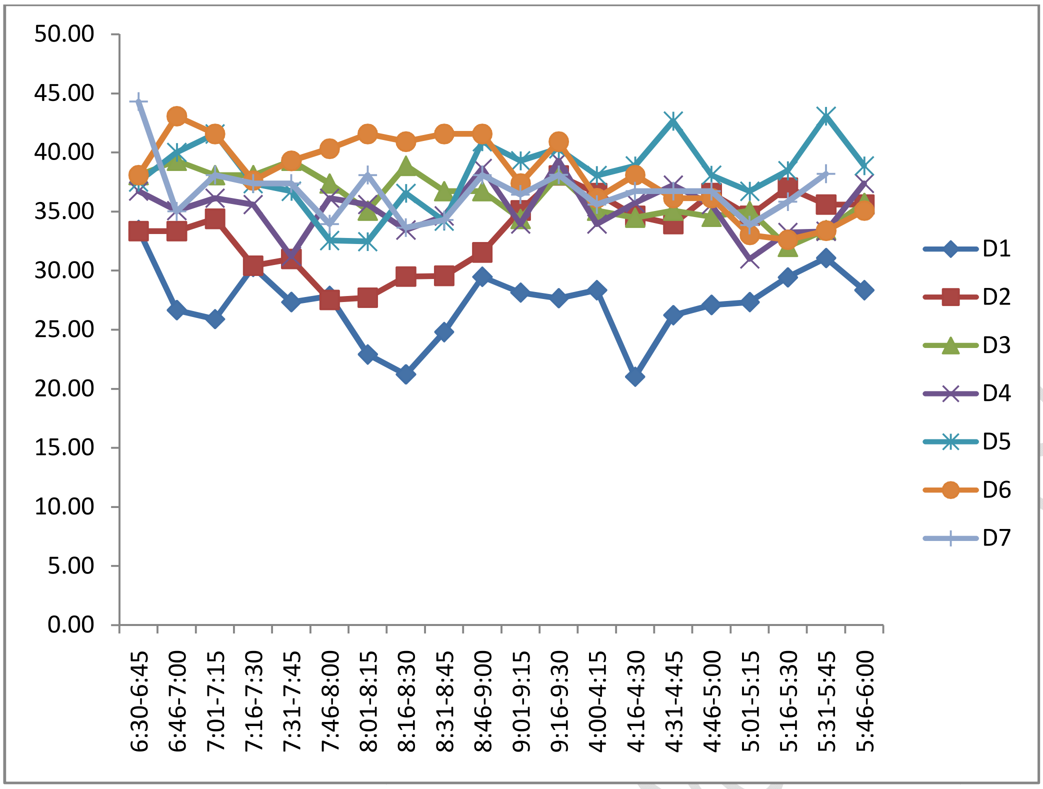


Figure 4.3: Aggregate average travel speed by time

The traffic information were also classified as: the **'early morning rise traffic, from 6:30 am to 7:00 am'**, when few vehicles were using the road at the early hour of the morning, the **'morning peak traffic, from 7:00 am to 9:30 am'**, when the travel demand was at its peak in the morning, and the **'evening peak traffic, from 4:00 pm to 6:00 pm'** when the travel demand was at its peak in the evening. The information is as follows:

(i.) The 'early morning rise traffic, from 6:30 am to 7:00 am'

'Day 1' shows a maximum average travel speed of 33.43 km/h (between 6:30 am and 6:45 am) and a minimum average travel speed of 26.64 km/h (between 6:45 am and 7:00 am). 'Day 2' shows an average travel speed of 33.33 km/h between 6:30 am and 6:45 am and 33.33 km/h between 6:45 am and 7:00 am, that is, travel speed is constant within these periods. 'Day 3' shows a maximum average travel speed of 39.29 km/h (between 6:46 am and 7:00 am) and 38.58 km/h (between 6:30 am and 6:45 am). 'Day 4' shows a maximum average travel speed of 36.71 km/h (between 6:30 am and 6:45 am) and 35.06 km/h (between 6:46 am and 7:00 am). 'Day 5' shows a maximum average travel speed of 40.00 km/h (between 6:46 am and 7:00 am) and 37.50 km/h (between 6:30 am and 6:45 am). 'Day 6' shows a maximum average travel speed of 43.06 km/h (between 6:46 am and 7:00 am) and 38.08 km/h (between 6:30 am and 6:45 am). 'Day 7' shows a maximum average travel speed of 44.29 km/h (between 6:30 am and 6:45 am) and 38.08 km/h (between 6:30 am and 6:45 am).

These periods corroborate with the same periods when maximum and minimum traffic flow rates were experienced at the same periods of the day.

(ii.) The 'morning peak traffic, from 7:00 am to 9:30 am'

'Day 1' shows maximum average travel speed of 30.29 km/h (between 7:15 am and 7:30 am) and a minimum average travel speed of 21.21 km/h (between 8:16 am and 8:30 am). 'Day 2' shows maximum average travel speed of 38.04 km/h (between 7:01 am and 7:15 am) and a minimum average travel speed of 27.50 km/h (between 9:16 am and 9:30 am). 'Day 3' shows maximum average travel speed of 39.29 km/h (between 7:31 am and 7:45 am) and a minimum average travel speed of 34.36 km/h (between 9:01 am and 9:15 am). 'Day 4' shows maximum average travel speed of 39.29 km/h (between 9:16 am and 9:30 am) and a minimum average travel speed of 31.19 km/h (between 7:31 am and 7:45 am). 'Day 5' shows maximum average travel speed of 40.32km/h (between 9:16 am and 9:30 am) and a minimum average travel speed of 32.45 km/h (between 8:01 am and 8:15 am). 'Day 6' shows maximum average travel speed of 41.56 km/h (between 7:00 am and 9:00 am) and a minimum average travel speed of 37.36 km/h (between 9:01 am and 9:15 am). 'Day 7' shows maximum average travel speed of 38.08 km/h (between 7:01 am and 7:15 am) and a minimum average travel speed of 33.64 km/h (between 8:16 am and 8:30 am).

These periods also corroborate with the same periods when maximum and minimum traffic flow rates were experienced at the same periods of the day.

(iii.) The 'evening peak traffic, from 4:00 pm to 6:00 pm'

'Day 1' shows maximum average travel speed of 33.52 km/h (between 5:30 pm and 5:45 pm) and a minimum average travel speed of 21.01 km/h (between 4:16 pm and 4:30 pm). 'Day 2' shows maximum average travel speed of 37.00 km/h (between 5:16 pm and 5:30 pm) and a minimum average travel speed of 33.93 km/h (between 4:31 pm and 4:45 pm). 'Day 3' shows maximum average travel speed of 35.71 km/h (between 5:46 pm and 6:00 pm) and a minimum average travel speed of 32.00 km/h (between 5:16 pm and 5:30 pm). 'Day 4' shows maximum average travel speed of 37.50 km/h (between 5:31 pm and 5:45 pm) and a minimum average travel speed of 33.93 km/h (between 4:01 pm and 4:15 pm). 'Day 5' shows maximum average travel speed of 43.06 km/h (between 5:31 pm and 5:45 pm) and a minimum average travel speed of 38.08 km/h (between 4:46 pm and 5:00 pm). 'Day 6' shows maximum average travel speed of 38.08 km/h (between 4:16 pm and 5:45 pm) and a minimum average travel speed of 35.06 km/h (between 5:46 pm and 6:00 pm). 'Day 7' shows maximum average travel speed of 40.00 km/h (between 5:46 pm and 6:00 pm) and a minimum average travel speed of 35.58 km/h (between 4:00 pm and 4:15 pm).

These periods also corroborate with the same periods when maximum and minimum traffic flow rates were experienced at the same periods of the day.

4.2.4 Traffic density

Having gotten the results for average traffic flow rates and average travel speeds, the traffic densities, K (pc/km/ln) are computed as shown on Table 4.4.

Traffic density, being the number of vehicles per kilometer per lane of a highway depicts rate at which vehicles are utilizing the available road space. It determines the space between consecutive vehicles on the roadway, and thus relates with the level of service and permissible average travel along a selected roadway.

Table 4.4 illustrates the following relevant traffic information:

(i.) The 'early morning rise traffic, from 6:30 am to 7:00 am'

At this period, 'Day 1' shows maximum traffic density of 23 pcu/km/ln (between 6:46 am and 7:00 am) and a minimum traffic density of 20 pcu/km/ln (between 6:30 am and 6:45 am), this implies the corresponding space between consecutive vehicles during these periods are 43.48 m and 50.00 m, respectively. 'Day 2' shows maximum traffic density of 27 pcu/km/ln (between 6:46 am and 7:00 am) and a minimum traffic density of 18 pcu/km/ln (between 6:30 am and 6:45 am), this implies the corresponding space between consecutive vehicles during these periods are 37.04 m and 55.56 m, respectively.

Table 4.4: Computation of average traffic density, K (pcu/km/ln)

Day	Interval	Qagg (pcu/h/ln)	Us (km/h)	K = Qagg/Us
	6:30-6:45	655	33.43	19.60
	6:46-7:00	606	26.64	22.73
	7:01-7:15	1100	25.88	42.51

	7:16-7:30	1440	30.29	47.53
	7:31-7:45	1983	27.32	72.57
	7:46-8:00	2576	27.84	92.53
	8:01-8:15	1905	22.89	83.22
Day 1	8:16-8:30	2072	21.21	97.68
	8:31-8:45	2091	24.79	84.35
	8:46-9:00	2046	29.46	69.42
	9:01-9:15	1019	28.13	36.21
	9:16-9:30	1034	27.65	37.39
	4:00-4:15	944	28.33	33.31
	4:16-4:30	1432	21.01	68.13
	4:31-4:45	812	26.22	30.96
	4:46-5:00	947	27.08	34.98
	5:01-5:15	994	27.32	36.36
	5:16-5:30	864	28.33	30.49
	5:31-5:45	958	28.57	33.52
	5:46-6:00	1024	28.33	36.15
	6:30-6:45	584	33.33	17.51
	6:46-7:00	902	33.33	27.05
	7:01-7:15	1405	34.38	40.88
	7:16-7:30	1046	30.39	34.40
	7:31-7:45	1588	30.97	51.26
	7:46-8:00	1286	27.50	46.75
	8:01-8:15	1354	27.69	48.89
	8:16-8:30	1578	29.48	53.53
	8:31-8:45	1396	29.55	47.25
Day 2	8:46-9:00	1271	31.52	40.33
	9:01-9:15	1348	35.08	38.43
	9:16-9:30	1289	38.04	33.87
	4:00-4:15	856	36.54	23.41
	4:16-4:30	851	34.62	24.57
	4:31-4:45	882	33.93	25.98
	4:46-5:00	812	36.54	22.22
	5:01-5:15	774	34.62	22.37
	5:16-5:30	978	37.00	26.43
	5:31-5:45	964	35.58	27.08
	5:46-6:00	921	35.58	25.87

Table 4.4: Computation of average traffic density, K (pcu/km/ln) (contd.)

Day	Interval	Qagg (pcu/h/ln)	Us (km/h)	K = Qagg/Us
	6:30-6:45	552	38.08	14.5
	6:46-7:00	718	39.29	18.28
	7:01-7:15	820	38.08	21.52
	7:16-7:30	874	38.08	22.95
	7:31-7:45	937	39.29	23.84
	7:46-8:00	842	37.36	22.54
	8:01-8:15	875	35.06	24.94
	8:16-8:30	898	38.87	23.1
	8:31-8:45	1048	36.71	28.53
Day 3	8:46-9:00	958	36.71	26.08
	9:01-9:15	943	34.36	27.43
	9:16-9:30	909	38.08	23.87
	4:00-4:15	751	35.06	21.42
	4:16-4:30	651	34.47	18.88
	4:31-4:45	666	35.06	18.98
	4:46-5:00	984	34.54	28.47
	5:01-5:15	711	35.06	20.26
	5:16-5:30	795	32.00	24.84
	5:31-5:45	740	33.43	22.12
	5:46-6:00	984	35.71	27.54
	6:30-6:45	540	36.71	14.71
	6:46-7:00	797	35.06	22.72
	7:01-7:15	803	36.11	22.24
	7:16-7:30	875	35.58	24.59
	7:31-7:45	1154	31.19	37.01
	7:46-8:00	1012	36.12	28.02
	8:01-8:15	1025	35.58	28.80
	8:16-8:30	1134	33.42	33.94
	8:31-8:45	1117	34.62	32.25
	8:46-9:00	904	38.64	23.4
	9:01-9:15	913	33.93	26.91
Day 4	9:16-9:30	967	39.29	24.6
	4:00-4:15	655	33.93	19.31
	4:16-4:30	835	35.71	23.37
	4:31-4:45	909	37.22	24.42
	4:46-5:00	919	35.58	25.82
	5:01-5:15	709	36.12	19.63
	5:16-5:30	813	32.75	24.81
	5:31-5:45	736	37.50	19.63

	5:46-6:00	1128	37.36	30.18

Table 4.4: Computation of average traffic density, K (pcu/km/ln). (contd.)

Day	Interval	Q_{agg} (pcu/h/ln)	U_s (km/h)	K = Qagg/U_s
	6:30-6:45	580	37.50	15.46
	6:46-7:00	769	40.00	19.23
	7:01-7:15	798	41.56	19.21
	7:16-7:30	1008	37.36	26.98
	7:31-7:45	940	36.71	25.59
	7:46-8:00	1065	32.54	32.73
	8:01-8:15	1053	32.45	32.43
	8:16-8:30	1112	36.54	30.42
	8:31-8:45	904	34.19	26.43
	8:46-9:00	831	40.91	20.31
Day 5	9:01-9:15	797	39.29	20.27
	9:16-9:30	817	40.32	20.25
	4:00-4:15	609	38.04	16.01
	4:16-4:30	673	38.87	17.32
	4:31-4:45	600	42.65	14.07
	4:46-5:00	695	38.08	18.24
	5:01-5:15	688	36.71	18.73
	5:16-5:30	577	38.45	15.01
	5:31-5:45	652	43.06	15.14
	5:46-6:00	529	38.87	13.60
	6:30-6:45	350	38.08	9.19
	6:46-7:00	424	43.06	9.85
	7:01-7:15	510	41.56	12.27
	7:16-7:30	543	37.62	14.43
	7:31-7:45	661	39.29	16.82
	7:46-8:00	440	40.32	10.9
	8:01-8:15	428	41.56	10.29
	8:16-8:30	452	40.91	11.04
	8:31-8:45	431	41.56	10.37
Day 6	8:46-9:00	408	41.56	9.81
	9:01-9:15	475	37.36	12.7
	9:16-9:30	344	40.91	8.40
	4:00-4:15	386	36.12	10.68
	4:16-4:30	462	38.08	12.14
	4:31-4:45	447	36.12	12.36

	4:46-5:00	489	36.12	13.52
	5:01-5:15	363	36.43	9.97
	5:16-5:30	317	36.71	8.63
	5:31-5:45	341	38.08	8.94
	5:46-6:00	432	35.06	12.33

Table 4.4: Computation of average traffic density, K (pcu/km/ln). (contd.)

Day	Interval	Qagg (pcu/h/ln)	Us (km/h)	K = Qagg/Us
	6:30-6:45	580	44.29	13.09
	6:46-7:00	695	35.00	19.85
	7:01-7:15	782	38.08	20.52
	7:16-7:30	857	37.36	22.94
	7:31-7:45	825	37.36	22.07
	7:46-8:00	923	33.93	27.19
	8:01-8:15	749	38.08	19.68
	8:16-8:30	756	33.64	22.47
	8:31-8:45	743	34.29	21.66
	8:46-9:00	691	38.08	18.13
Day 7	9:01-9:15	689	36.43	18.91
	9:16-9:30	695	38.10	18.24
	4:00-4:15	672	35.58	18.87
	4:16-4:30	745	36.71	20.30
	4:31-4:45	730	36.71	19.88
	4:46-5:00	823	36.71	22.40
	5:01-5:15	807	37.36	21.60
	5:16-5:30	838	35.71	23.45
	5:31-5:45	750	38.64	19.40
	5:46-6:00	868	40.60	21.37

'Day 3' shows maximum traffic density of 18 pcu/km/ln (between 6:46 am and 7:00 am) and a minimum traffic density of 15 pcu/km/ln (between 6:30 am and 6:45 am), this implies the corresponding space between consecutive vehicles during these periods are 55.56 m and 66.67 m, respectively.

'Day 4' shows maximum traffic density of 23 pcu/km/ln (between 6:46 am and 7:00 am) and a minimum traffic density of 15 pcu/km/ln (between 6:30 am and 6:45 am); this implies the corresponding space between consecutive vehicles during these periods are 43.48 m and 66.67 m, respectively. 'Day 5' shows maximum traffic density of 19 pcu/km/ln (between 6:46 am and 7:00 am) and a minimum traffic density of 16 pcu/km/ln (between 6:30 am and 6:45 am); this implies the corresponding space between consecutive vehicles during these periods are 52.63 m and 62.50 m, respectively. 'Day 6' shows maximum traffic density of 10 pcu/km/ln (between 6:46 am and 7:00 am) and a minimum traffic density of 9 pcu/km/ln (between 6:30 am and 6:45 am), this implies the corresponding space between consecutive vehicles during these periods are 100.00 m and 111.11 m, respectively. 'Day 7' shows maximum traffic density of 20 pcu/km/ln (between 6:46 am and 7:00 am) and a minimum traffic density of 13 pcu/km/ln (between 6:30 am and 6:45 am), this implies the corresponding space between consecutive vehicles during these periods are 50.00 m and 76.92 m, respectively.

(ii.) The 'morning peak traffic, from 7:00 am to 9:30 am'

At this period, 'Day 1' shows maximum traffic density of 98 pcu/km/ln (between 8:31 am and 8:45 am) and a minimum traffic density of 36 pcu/km/ln (between 9:01 am and 9:15 am), this implies the corresponding space between consecutive vehicles during these periods are 37.78 m and 27.78 m, respectively. 'Day 2' shows maximum traffic density of 54 pcu/km/ln (between 8:16 am and 8:30 am) and a minimum traffic density of 34 pcu/km/ln (between 9:16 am and 9:30 am), this implies that the corresponding space between consecutive vehicles during these periods are 18.52 m and 29.41 m, respectively. 'Day 3' shows maximum traffic density of 29 pcu/km/ln (between 8:31 am and 8:45 am) and a minimum traffic density of 22 pcu/km/ln (between 7:01 am and 7:15 am), this implies that the corresponding space between consecutive vehicles during these periods are 34.48 m and 45.45 m, respectively. 'Day 4' shows maximum traffic density of 37 pcu/km/ln (between 8:31 am and 8:45 am) and a minimum traffic density of 22 pcu/km/ln (between 7:01 am and 7:15 am), this implies that the corresponding space between consecutive vehicles during these periods are 27.03 m and 45.45 m, respectively. 'Day 5' shows maximum traffic density of 33 pcu/km/ln (between 7:46 am and 8:00 am) and a minimum traffic density of 19 pcu/km/ln (between 7:01 am and 7:15 am), this implies that the corresponding space between consecutive vehicles during these periods are 30.30 m and 52.63 m, respectively.

'Day 6' shows maximum traffic density of 17 pcu/km/ln (between 7:31 am and 7:45 am) and a minimum traffic density of 10 pcu/km/ln (between 8:46 am and 9:00 am), this implies that the corresponding space between consecutive vehicles during these periods are 58.82 m and 100.00 m, respectively. 'Day 7' shows maximum traffic density of 27 pcu/km/ln (between 7:46 am and 8:00 am) and a minimum traffic density of 18 pcu/km/ln (between 8:46 am and 9:00 am), this implies that the corresponding space between consecutive vehicles during these periods are 37.03 m and 55.56 m, respectively.

(iii.) The 'evening peak traffic, from 4:00 pm to 6:00 pm'

'Day 1' shows maximum traffic density of 68 pcu/km/ln (between 4:16 pm and 4:30 pm) and a minimum traffic density of 31 pcu/km/ln (between 5:16 pm and 5:30 pm); this implies that the corresponding space between consecutive vehicles during these periods are 14.70 m and 32.26 m, respectively. 'Day 2' shows maximum traffic density of 27 pcu/km/ln (between 5:31 pm and 5:45 pm) and a minimum traffic density of 22 pcu/km/ln (between 5:01 pm and 5:15 pm); this implies that the corresponding space between consecutive vehicles during these periods are 37.03 m and 45.45 m, respectively. 'Day 3' shows maximum traffic density of 29 pcu/km/ln (between 4:46 pm and 5:00 pm) and a minimum traffic density of 19 pcu/km/ln (between 4:16 pm and 4:30 pm); this implies that the corresponding space between consecutive vehicles during these periods are 34.48 m and 52.63 m, respectively. 'Day 4' shows maximum traffic density of 30 pcu/km/ln (between 4:46 pm and 5:00 pm) and a minimum traffic density of 19 pcu/km/ln (between 4:00 pm and 4:15 pm); this implies that the corresponding space between consecutive vehicles during these periods are 33.33 m and 52.63 m, respectively. 'Day 5' shows maximum traffic density of 20 pcu/km/ln (between 9:16 pm and 9:30 pm) and a minimum traffic density of 14 pcu/km/ln (between 5:46 pm and 6:00 pm); this implies that the corresponding space between consecutive vehicles during these periods are 50.00 m and 71.43 m, respectively. 'Day 6' shows maximum traffic density of 14 pcu/km/ln (between 4:46 pm and 5:00 pm) and a minimum traffic density of 9 pcu/km/ln (between 5:16 pm and 5:30 pm); this implies that the corresponding space between consecutive vehicles during these periods are 71.43 m and 111.11 m, respectively. 'Day 7' shows maximum traffic density of 24 pcu/km/ln (between 5:16 pm and 5:30 pm) and a minimum traffic density of 19 pcu/km/ln (between 5:31 pm and 5:45 pm); this implies that the corresponding space between consecutive vehicles during these periods are 41.67 m and 52.63 m, respectively.

4.3 Model Development and Evaluation

In this study the speed-density model was considered as the primary model due to its simplicity and the ability to develop other traffic flow relationships from it. It was assumed that there is a linear relationship between travel speed and traffic density as postulated by Greenshields (1935). The model was developed using linear regression analysis, and evaluated using the coefficient of determination (R-squared) technique. Due to the limitation of the R-squared technique in showing the adequacy of the developed model, an F-test analysis was then carried out to determine the significance level of the model.

The speed-density model was developed using the corresponding average travel speed and traffic density data as shown in Table 4.4.

From the analysis of the data, the following regression parameters were obtained:

$\sum K = 3805, \sum U = 5014, \sum K.U = 125594, \sum K^2 = 135914,$

$\sum U^2 = 178779$, $\bar{K} = 26.61$, $\bar{U} = 35.07$, n = 140, $SS_{kk} = 32475$, SS_{uu} =-43683,

$SS_{k.u} = -10704$

Thus, the linear regression equation representing the speed-density model for the selected road is;

$$U_s = -0.324K + 41.13 \quad (4.1)$$

This is illustrated as shown in Figure 4.4.

The evaluation of this model gives an R-squared value of 0.611, to verify the adequacy of this value, the F-test analysis is shown in Table 4.5.

Using the following hypotheses:

H_o (Null hypothesis): There is no lack of linear relationship, and

H_1 (Alternative hypothesis): There is lack of linear relationship

The F-test analysis shows that the model has a P-value less than 0.05 significance level and that F (0.08) < $F_{critical}$ (0.76), it implies that the null hypothesis is correct and the model is significant at 0.05. In order to justify the low value of $F_{critical}$, the F-statistics was carried out using the density as the dependent variable and the average travel speed as the independent variable. The result is illustrated in Table 4.6 and it shows that F(12.17) > $F_{critical}$ (1.32);

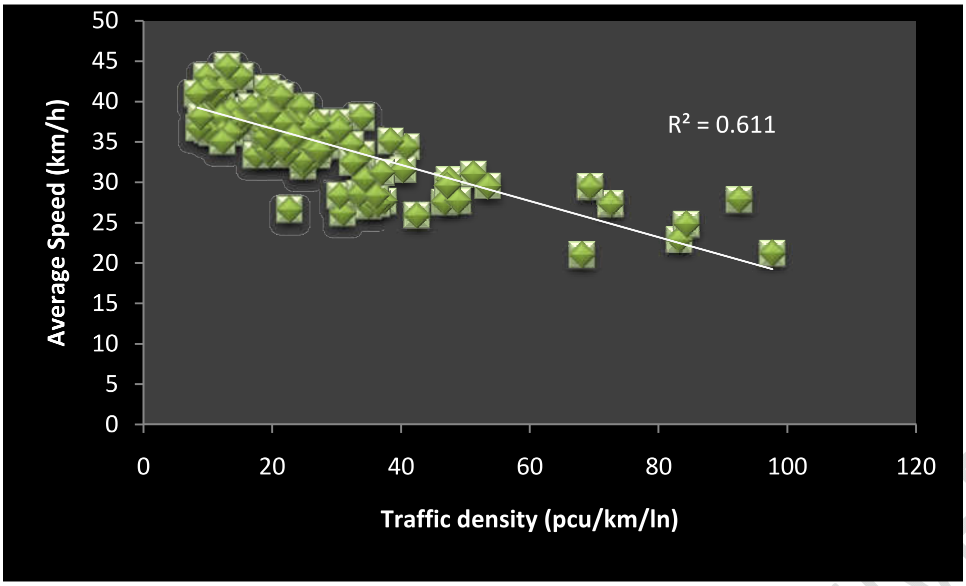


Figure 4.4: Speed-density model for the selected road

Table 4.5: F-Test for Speed-Density Relationship

	Speed (km/h)	*Density (pcu/km/ln)*
Mean	35.20	26.49
Variance	20.41	248.44
Observations	140.00	140.00
Df	139.00	139.00
F	0.08	
P(F<=f) one-tail	0.00	
F Critical one-tail	0.76	

Table 4.6: F-Test for Density-Speed Relationship

	Density, pcu/km/ln	*Speed, km/h*
Mean	26.49	35.20
Variance	248.44	20.41
Observations	140.00	140.00
df	139.00	139.00
F	12.17	
P(F<=f) one-tail	1.73541E-40	
F Critical one-tail	1.32	

this implies that the model is non-linear as proposed, hence, the initial F-Test is considered as the most suitable, and this justified the developed speed-density model.

Thus, the resulting **speed-density model** is: $U_s = -0.324K + 41.13$

By relating this equation with the equation postulated by Greenshields (1935);

$U_s = (-0.324)K + 41.13$

$$\boldsymbol{U_s} = \left(-\frac{U_f}{K_j}\right)\mathbf{K} + \boldsymbol{U_f}$$ Greenshields (1935)

This implies that when traffic density, K = 0, the free flow speed, $U_f = 41.13$ km/h.

Also, $\frac{U_f}{K_j} = 0.324, thus,\ Jam\ density, K_j = 127$ pcu/km/ln

These are essential traffic parameters needed for evaluating the prevailing traffic performance of the road.

Having got the speed-density model for morning and evening peaks, the flow-density model was developed thus:

The flow-density model was determined by applying $Q = K \times U$

Since $U_s = -0.324K + 41.13$

Thus, the **flow-density model** was got as;

$$Q = -0.324K^2 + 41.13K \tag{4.2}$$

This model is illustrated as shown in Figure 4.5. The model showed that flow-density relationship for the selected road is quadratic in nature, with R-squared value of 0.942 and p-value of 0.000. The F-test shows that F(6728.12) > $F_{critical}$ (1.32); this implies

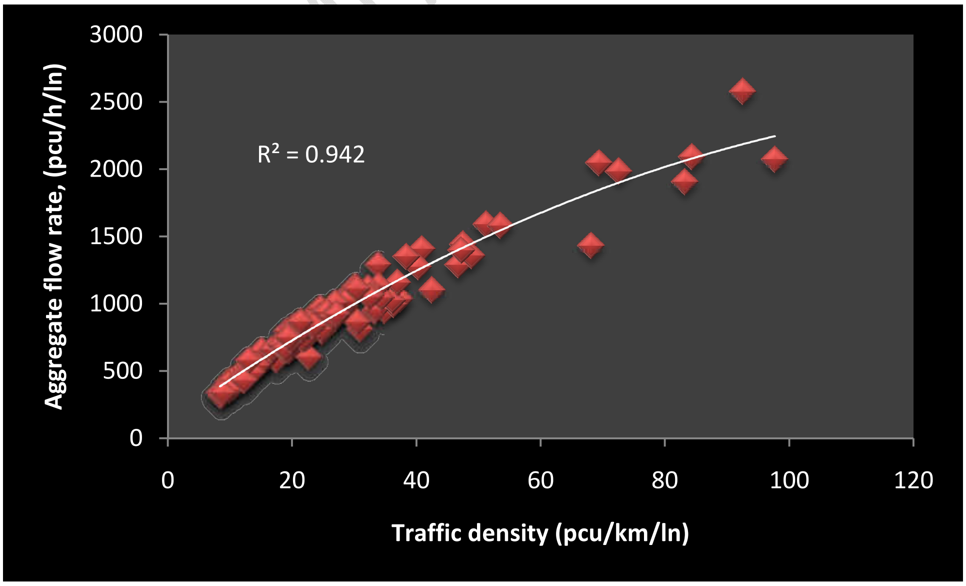


Figure 4.5: Flow-density model for the selected road

Table 4.7: F-Test for Flow-Density Relationship

	Flow, pcu/h/ln	Density,pcu/km/ln
Mean	877.19	35.20
Variance	137306.09	20.41
Observations	140.00	140.00
Df	139.00	139.00
F	6728.12	
P(F<=f) one-tail	2.1101E-226	
F Critical one-tail	1.32	

that model is not linear and thus, the developed flow-density model is justified as proposed and the R-squared value is significant at 0.05.

Also, the flow-speed model was developed using the developed speed-density model.

Since, $U_s = -0.324K + 41.13$

By applying $K = Q/U$, the **flow-speed model** was got as;

$$Q = -3.09{U_s}^2 + 126.94U_s \qquad (4.3)$$

This model is illustrated as shown in Figure 4.6. The model also showed that flow-speed relationship for the selected road is quadratic in nature, with an R-squared value of 0.431. The F-test for flow-speed relationship shows a p-value of 0.000 and the F (6728.12)>F_{cr}(1.32); this showed that the model is not linear as proposed. It also shows that the flow-speed model is significant at 0.05.

4.3.1 Capacity determination

Capacity is reached when the product of density and speed results in the maximum flow rate. This condition is shown as optimum speed U_o (often called critical speed), and was obtained as 20.57 km/h for this study.

Also, the optimum density K_o (sometimes referred to as critical density) was obtained as,

$K_o = 63.5$ pcu/km/ln.

Therefore, the capacity which represents the maximum flow $Q_m = K_o \times U_o$ (4.4)

$Q_m \approx 1306$ pcu/h/ln

Table 4.8 shows a comparison of the determined capacity and operating speed with that recommended for a two-way two-lane road by the Federal Ministry of Works, Highway Design Manual (2007).

Table 4.8: F-Test for Flow-Speed Relationship

	Flow, pcu/h/ln	Speed, km/h
Mean	877.19	35.20
Variance	137306.09	20.41
Observations	140.00	140.00
Df	139.00	139.00
F	6728.12	
P(F<=f) one-tail	2.1101E-226	
F Critical one-tail	1.32	

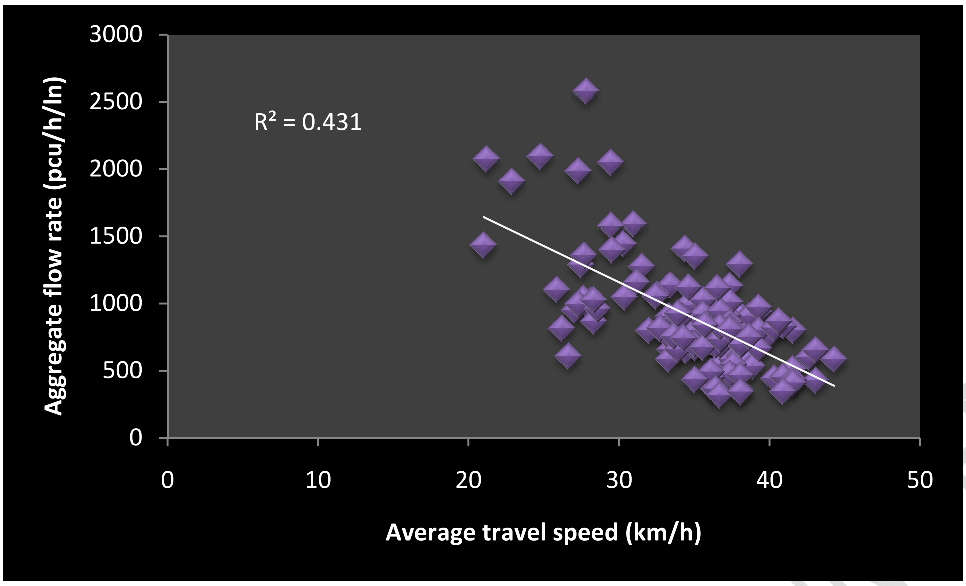


Figure 4.6: Flow-speed model for the selected road

Table 4.9: Capacity (pcu/h/ln) and operating speed (km/h) compared with the desired values

	Capacity (pcu/h/ln)	Operating Speed (km/h)
Federal ministry of works	1500	50 – 60
This study	1306	41.13

Since the obtained capacities and operating speeds are less than the recommended values, it implies that the road is operating below the required capacity.

4.3.2 Determination of percentage free flow speed (PFFS)

Recall; ATS = FFS – 0.0155(Q_o) – $f_{np,ATS}$

Where ATS = average travel speed, km/h

FFS = free flow speed = 41.13 km/h

Q_o = optimum traffic flow rate per lane = 1306 pcu/h/ln

$f_{np,ATS}$ = ATS Adjustment Factor for No-Passing Zones = 0.3 (HCM, 2010)

Therefore, ATS = 41.13 – 0.0155 (1306) – 0.3 = 20.59 km/h

Thus, $\text{PFFS} = \frac{\text{ATS}}{\text{FFS}} = \frac{20.59}{41.13} = 0.501 = 50.10\ \%$

This implies that the level of service (LOS) of the road is E (see Table 2.3). This denotes that the capacity of the highway has been reached. And that traffic flow conditions are best described as unstable with any traffic incident causing extensive queuing and even breakdown. Levels of comfort and convenience are very poor and travel speeds are low.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The developed models illustrate several significant points. First, a zero flow rate occurs under two conditions. One is when there are no vehicle on the road; density is zero, and flow rate is zero. Speed is theoretical for this condition and would be selected by the first driver (presumably at a high value). This speed is called the free flow speed and was computed to be 41.13 km/h. The second is when density becomes so high that all vehicles must stop, the speed is zero, and the flow rate is zero, because there is no movement and vehicles cannot pass the point on the roadway. The density at which all movement stops is called jam density denoted by K_j. In this study K_j was determined to be 127 pcu/km/ln.

Between these two extreme points, the dynamics of traffic flow produce a maximizing effect. As flow increases from zero, density also increases, since more vehicles are on the roadway. When this happens, speed declines because of the interaction of vehicles. This decline is negligible at low and medium densities and flow rates. As the density increases, these generalized models suggest that speed decreases significantly before capacity is achieved.

Following an extensive literature review on the existing traffic flow models, the need for the development of a macroscopic traffic model capable of modelling an integrated network of motorway and urban roads was identified. The majority of the traffic models used for the prediction of traffic movement is only sufficient for signalized road networks, where the traffic interactions experienced at the priority junctions cannot be captured. For instance, the traffic on the minor road at a T-junction is not only characterized by the corresponding flow capacity but also limited by the incoming flow on the major road. The techniques presented for priority junctions mainly focused on investigating the traffic on single intersection and are not accurate for modelling directional flow, therefore they are not capable of identifying traffic characteristic at the network level. Considering the traffic interaction at both signalized and priority junctions, this thesis describes a network level traffic flow model (NTFM) the purpose of which is to provide a method for predicting traffic flows and performance measure through the road network. In summary, this study provided a new means of determining the level of service and performance rating of two-lane highways compared to the method provided in the Highway Capacity Manual (HCM). Traffic parameters obtained from this study showed that the road is operating below the required capacity and at the level of service E. This denotes that the capacity of the highway has been reached. And that traffic flow conditions are best described as unstable with any traffic incident causing extensive queuing and even breakdown. Levels of comfort and convenience are very poor and travel speeds are low. This calls for timely effective traffic management plan for the road in order to withstand the increasing travel demand.

5.2 Recommendations

It is recommended that:

(i.) The developed models could be used to predict the speed-flow-density relationships for urban roads in Ile-Ife and other similar areas.

(ii.) Similar study should be carried out for other major roads in various cities of Nigeria.

(iii) To obtain more accurate and enough traffic data on the road segment, loop detectors should be installed on various roads in the cities of Nigeria.

(iii.) Furthermore, to prevent congestion and to improve efficiency, off-road parking systems at interval along the road should be incorporated to the roadway system, commercial activities should be controlled and kerb parking prohibition strategy should be enforced.

REFERENCES

Aderamo, A.J. (2002). 'Transport and the Nigerian Urban Environment'. Paper delivered at 45th Annual Conference of the Nigerian Geographical Association held at University of Ilorin. 30th June – 3rd July 2002.

Al-Kaisy, A. and Karjala, S. (2008). 'Indicators of Performance on Two-Lane Rural Highways: Empirical Investigation'. Transportation Research Record: *Journal of the Transportation Research Board, 2071(1): 87-97.*

Alkubaisi,M.I. and Abbas, R.A. (2011). 'Speed Flow Density Models Prediction for Urban Roadway in Falluja City'. *Anbar Journal for Engineering Sciences, 2011, 4(1): 3-10.*

Amadou, G., Lyudmila, M. and Rene, B. (2010). 'Inter Macroscopic Models for Traffic Networks'. *IEEE Transactions on Intelligent Transportation System.11:3-7.*

Baerwald, J.E., Huber, M.J. and Keefer, L.E. (1976). 'Transportation and Traffic Engineering Handbook'. Prentice-Hall Inc, New Jersey.

Banks, J. (1989). 'Freeway Speed-Flow-Concentration Relationship: More Evidence and Interpretations'. *Transport Research Record. 1225, 53-60.*

Bellemans T., De Schutter, B. and De Moor, B. (2002). 'Models for traffic control'. *Applied Science Journal, 2002, 43 (3–4): 13–22.*

Bellomo, N., Coscia, V. and Delitala, M. (2002). 'On the Mathematical Theory of Vehicular Traffic Flow I. Fluid Dynamic and Kinetic Modelling'. *Applied Science Journal. 12 (12): 1801–1843.*

Berge, G. and Vaa, T. (2003). 'Feeling of speed: A qualitative study of reasons for speed choices'. Transportökonomisk Institutt, Oslo, Norway.

Brilon, W. and Ponzlet , M. (1994). 'Variability of speed-flow relationships on german auto-bahns'. *Transportation Research Record. 1555, 91-98.*

Chandler, R., Herman, R. and Montroll, E. (1958). 'Traffic dynamics: studies in car following'. *Operations Research Journal. 6, 165–184.*

Cohen, B. (2004). 'Urban growth in developing countries: a review of current trends and a caution regarding existing forecasts'. *World Development Journal. 32, 23-51.*

Cohen, B. (2006). 'Urbanization in developing countries: current trends, future projections, and key challenges for sustainability'. *Technology in Society Journal. 28, 63-80.*

Daganzo, C.F. (2005). 'Improving City Mobility through Gridlock Control: An Approach and Some Ideas'. Berkeley Center of Excellence on Future Urban Transport Working Paper. California, USA.

Drake, J.S., Schofer, J.L. and May, A.D. (1967). 'A statistical analysis of speed–density hypotheses'. *Highway Research Record. 154, 53–87.*

Durning, A. T. (1992). 'How much is enough? The consumer society and the future of the earth'. Norton, New York.

Edie, L.C. (1960). 'Car Following and Steady-State Theory for Non-Congested Traffic'. *Operations Research Journal. 9, pp. 66-76.*

Filarski, R. (2004). 'The rise and decline of transport systems: changes in a historical context'. Rotterdam, Rijkswaterstaat.

Gakenheimer, R. (1999). 'Urban mobility in the developing world'. *Transportation Research Part A: Policy and Practice. 33, 671-689.*

Garber, N. and Sankar, S. (2003). 'Feasibility of Incorporating Crash Risk in Developing Congestion Mitigation Measures for Interstate Highways: A Case Study of the Hampton Roads Area'. Virginia Transportation Research Council, Final Report VTRC 02-R17. Charlottesville, Virginia.

Garber, N.J and Hoel, L.A. (2010). 'Traffic and Highway Engineering'. Cengage Learning. Stamford.

Geroliminis, N. and Daganzo, C. F. (2008). 'Existence of urban-scale macroscopic fundamental diagrams: Some experimental findings'. *Transportation Research Part B-Methodological. 42, 759–770.*

Google. "Ile-Ife Google Maps". Available on http://maps.google.com. Assesed on December 2014.

Greenberg, H. (1959). 'A Mathematical Analysis of Traffic Flow, Tunnel Traffic Capacity Study'. The Port of New York Authority, New York.

Greenshields, B.D. (1934). 'A Study of Highway Capacity'. *Highway Research Board Proceedings. 14, p. 468.*

Greenshields, B.D. (1935). 'A study of traffic capacity'. *Highway Research Board Proceedings. 14, 448-477.*

Gunnar, F. and Jannis, R. (2010). 'Operational Macroscopic Modelling of Complex Urban Road Intersections'. *Transport and Mobility Laboratory, Ecole Polytechnique Fédérale de Lausan. 1, p10-5.*

Gwilliam, K. (2003). 'Urban transport in developing countries'. Transport Reviews: *A Transnational Transdisciplinary Journal. 23, 197-216.*

Haberman, R. (1977). 'Mathematical Models in Mechanical Vibrations, Population Dynamics, and Traffic Flow'. Prentice-Hall, 259-394.

Haight, F.A. (1963). 'Mathematical Theories of Traffic Flow'. Academic Press, New York.

Hall, F. and Persaud, B. (1989). 'Evaluation of speed estimates made with single detector data from freeway traffic management systems'. *Transportation Research Record. 1232, 9-16.*

Harwood, D.W., May, A.D., Anderson, I.B. and Archilla, A.R. (1999). 'Capacity and quality of service of two-lane highways'. *Midwest Research Institute Journal. NCHRP Final Rep. 55 (3), 145-148.*

Hashim, I.H. and Abdel-Wahed, T.A. (2011). 'Evaluation of performance measures for rural two-lane roads in Egypt'. *Alexandria Engineering Journal. 50, 245-255.*

Herman, R. and Prigogine, I. (1979). 'A two-fluid approach to traffic flow modelling'. *Town Science Journal. 204:148-151.*

Highway Capacity Manual (2000). Transportation Research Board, National Research Council, Washington D.C.

Hwasoo, Y. (2008). 'Asymmetric Microscopic Driving Behaviour Theory'. Unpublished Ph.D Thesis, Submitted to the Department of Civil and Environmental Engineering, University of California, Berkeley.

Janvier, T. (2013). 'Establishing and Applying Speed-Flow Relationships for Traffic on Rural Two-Lane Two- Way Highways in The Western Cape'. Unpublished M.Sc. Thesis, Submitted to the Department of Civil Engineering, Faculty of Engineering, Stellenbosch University.

Jiyang B. B., Winnie D. and Serge H. (2010). 'Macroscopic fundamental diagram: Investigating its shape using simulation data'. *Transportation Research Board, 40-48.*

Kim, J. and Elefteriadon, L. (2009). 'Estimation of capacity of two-way highways using simulation model'. *Journal of Transportation Engineering. 136, 61-66.*

Leutzbach, W. (1988). 'An Introduction to the Theory of Traffic Flow'. Springer-Verlag, Berlin, Germany.

Levinson, D., Lei, Z., Shantanu, D. and Atif, S. (2003). 'Ramp Meters on Trial: Evidence from the Twin Cities Ramp Meters Shut-Off'. Paper presented at the 81st Meeting of the Transportation Research Board. Washington, D.C, 2003.

Lighthill, M.J. and Whitman, G.B. (1955). 'On kinematic waves. II. A theory of traffic flow on long crowded roads'. Proceedings of the Royal Society of London. Series A. *Mathematical and Physical Sciences. 229, 317-345.*

Lum, K., Fan, H., Lam, S., Olszewski, P. (1998). 'Speed-Flow Modeling of Arterial Roads in Singapore'. *Journal of Transportation Engineering. 124, 213-222.*

Luttinen, T.R. (2001). 'Traffic Flow on Two-Lane Highways. An Overview'. TL Research Report, TL Consulting Engineers, Ltd.

Mannering, F.L., Kilareski, W. and Washburn, S. (2007). 'Principles of Highway Engineering and Traffic Analysis'. John Wiley and Sons, New York.

Mathew V. (2006-12-10). 'Traffic stream Models'. Lecture Notes on Transportation System Engineering, Department of Civil and Environmental Engineering, Institute of Information Technology, Bombay.

Mathew V. (February 19, 2014). 'Measurement Over a Short Section'. Lecture Notes on Transportation System Engineering, Department of Civil and Environmental Engineering, Institute of Information Technology, Bombay.

Mathew V. (February 19, 2014). 'Congestion Studies'. Lecture Notes on Transportation System Engineering, Department of Civil and Environmental Engineering, Institute of Information Technology, Bombay.

Markos, P. (1991). 'Concise Encyclopaedia of Traffic and Transportation Systems'. ISBN:0-08-036203-6.1991. Prentice, London.

May, A.D. (1990). 'Traffic Flow Fundamentals'. Prentice-Hall Inc, New Jersey.

McShane, W.R. and Roess, R.P. (1990). Traffic Engineering. Pearson Prentice-Hall, Inc, New Jersey.

Michalopoulos, P. G., Yi, P. and Lyrintzis, A. S. (1993). 'Continuum Modelling of Traffic Dynamics for Congested Freeways'. *Transportation Research Board. 27B, (4), pp. 315-332.*

Montgometry, D.C. and Runger, G.C. (2007). 'Applied Statistics and Probability for Engineers'.Wiley.com, United States.

Nigerian Highway Design Manual (2007). Part I. Directorate of Highway Planning and Design, Federal ministry of Works, Headquarter, Abuja.

Nielsen, O.A., Jorgensen, R.M. (2008). 'Estimation of speed-flow and flow-density relations on the motorway network in the greater Copenhagen region'. *Intelligent Transport Systems, IET. 2, 120-131.*

O'Flaherty, C.A. (1997). 'Transport Planning and Traffic Engineering'. Arnold member of Hodder Headline Group, London.

Obeid, M.A. (2001). 'Development of Traffic Flow Characteristic Models for Selected Sections with Two Arterial Street in Baghdad City', Unpublished M.Sc. thesis, Submitted to the Department of Civil Engineering, University of Technology, Iraq.

Ogunbodede E. F. (2008). 'Urban Road Transportation in Nigeria from 1960 to 2006: Problems, Prospects And Challenges'. *Ethiopian Journal of Environmental Studies and Management, 1 (1), pp. 3-8.*

Ogunsanya, A.A. (1988); "Moving the Urban Masses: Two Steps Forward one Backwards". *Nigerian Institute of Transport and Technology, NITT, Nigeria. 1(2), pp. 7-9.*

Olszewski, P., Fan, H.S.L., Tan, Y.W. (1995). 'Area-wide traffic speed-flow model for the Singapore CBD'. *Transportation Research Part A: Policy and Practice. 29, 273-281.*

Otuoze, H.S., Amartey, Y.D., Sada, B.H. and Ahmed, A.H. (2012). 'Macroscopic approach to modeling of road traffic data in Zaria, Nigeria'. *World Journal of Engineering and Pure and Applied Sciences; 2(3):125- 30.*

Papageorgiou, M. (1983). 'Dynamic modelling, assignment, and route guidance in traffic networks'. *Transportation Research B 23, 29-48.*

Pipes, L. A. (1953). 'An operational analysis of traffic dynamics'. *Journal of applied physics, 24, 274-281.*

Robertson, H. D., Hummer, J. E. and Nelson, D. C. (1994). 'Volume Studies. In Manual of Transportation Engineering Studies'. Prentice Hall, Inc., pp. 6– 31.

Roger, P.R., Elena, S.P. and William, R.M. (2003), 'Traffic Engineering'.Third Edition. Prentice Hall, New Jersey.

Rogers, M. (2003), 'Highway Engineering'. Blackwell Publishing Limited, Paris, France. ISBN 0-632-05993-1.

Roux, J. (2001). 'Establishing and applying speed-flow relationships for traffic on South African freeways'. Unpublished M.Sc. thesis, Submitted to the Department of Civil Engineering, University of Stellenbosch, Stellenbosch.

Rozic, P. (1992). 'Capacity of two-lane, two-way rural highways: The new approach'. Transportation Research Record: *Journal of the Transportation Research Board, 1365: 19-29.*

Rust, F. C., Van W.I., Ittmann, h. W. and Kistan, K. (2008). 'The role of R&D in transport infrastructure in South Africa'. 27th Annual Southern African Transport Conference, 2008, Pretoria.

Salter, Y. and Hounsell, N. B. (2012). 'Highway Traffic Analysis and Design'. 3rd edn. Palgrave Macmillan, Hampshire.

Sharma, S. C. (1994). 'Seasonal Traffic Counts for a Precise Estimation of AADT'. *ITE Journal. 64, pp. 34–41.*

Shawky, M. & Hashim, I. (2010). Impact of horizontal alignment on traffic performance at rural two-lane highways. In Proc. of 4th International Symposium on Highway Geometric Design. Valencia, Spain, 2010.

Taale, H. (2008). 'Integrated anticipatory control of road networks. A game- theoretical approach'. Trail Research School, Delft, Netherlands.

Tapani, A. (2008). 'Traffic Simulation modelling of rural roads and driver assistance systems'. Unpublished Ph.D dissertation, Submitted to Department of Civil Engineering, Linköping studies in sciences and technology, Linköping.

Thomson, J.M. (1967). 'Speeds and flows of traffic in central London: Speed-flow relations'. *Traffic Engineering and Control. 8, 455-458.*

The World Bank (2002). 'Urban mobility in three cities, Addis Ababa, Dar es Salaam and Nairobi'. Sub-Saharan Africa Transport Policy Program. The World Bank, Economic Commission for Africa.

The World Bank (2010). World databank, world development indicators and global development finance. The World Bank.

TRB (1985). Highway Capacity Manual. Transportation Research Board, National Research Council, Washington D.C.

TRB (2010). Highway Capacity Manual 2010. Transportation Research Board, National Research Council, Washington D.C.

Tsubota, T., Bhaskar, A. and Chung, E. (2014). 'Brisbane Macroscopic Fundamental Diagram :Empirical findings on network partitioning and incident detection'. In Transportation Research Board 93rd Annual Meeting, Washington, D.C, 2014.

Twagirimana, J. (2013). 'Establishing and Applying Speed-Flow Relationships for Traffic on Rural Two-Lane Two- Way Highways in The Western Cape'. Unpublished M.Sc. Thesis, Submitted to the Department of Civil Engineering, Faculty of Engineering, Stellenbosch University.

Underwood, R.T. (1961). 'Speed, volume, and density relationships: Quality and theory of traffic flow'. *Yale Bureau of Highway Traffic. 141-188.*

United Nations (2004). World urbanization prospects: the 2003 revision data tables and highlights. New York: UN.

Van As, C. (2003). 'The development of an analysis method for the determination of level of service of two-lane undivided highways in South Africa'. Project Summary. South African National Roads Agency, South Africa.

Van Lint, H. (2009). 'Innovations in dynamic traffic management'. Intelligent transport systems and services for road transport. Delft University of Technology.

Wardrop, J.G. (1968). 'Journey speed and flow in central urban areas'. *Traffic Engineering and Contro Journal. 9, 528-532.*

Witczak, M.W., Kalaush, K. and Pellinen, T. (2002). 'Simple Performance Test for Superpave Mix Design'. NCHRP Report no. 465.

Xie, H., Jiang, Y. and Zhang, L. (2011). 'Empirical Study on Relationship among the Lane-Changing, Speed and Traffic Flow'. *Systems Engineering Procedia. 2(0), 287-294. Yagar.*

Zahavi, Y. (1972). 'Traffic performance evaluation of road networks by the relationship'. Parts I and II. *Traffic Engineering and Control, 14(5, 6):228-231,292-293.*

Zwarteveen, J.W. (2011). 'Urban Traffic Flow Modeling in Addis Ababa'. Unpublished M.Sc. Thesis, Submitted to the Department of Marine and Transport Technology, Faculty of mechanical, maritime and materials engineering, Universityy of Technology. Delft, Netherlands.

Appendix 1: Traffic Volume data sheet

SITE LOCATION: **SEGMENT:**

DATE:

STARTING TIME: **ENDING TIME:**

FIELD OFFICER'S NAME:

		PC	**BUSES**	**TRUCKS**	**CYCLES**	**PC**	**BS**	**TRK**	**CY**	**TOTAL**
S/N	**INTERVAL**		**TOTAL TALLY**			**TOTAL PCE**				

PC = passenger cars, BS = buses, TRK = trucks, CY = motor cycles,

PCE= passenger car equivalent

Appendix 2: Speed recording data sheet

SITE LOCATION: **SEGMENT:**

DATE:

STARTING TIME: **ENDING TIME:**

SEGMENT LENGTH,L:

FIELD OFFICER'S NAME:

S/N	**INTERVAL**	**VEHICLE, *i***	**T.TIME(s)**	**VEHICLE, *i***	**T.TIME(s)**

		1		27	
		2		28	
		3		29	
		4		30	
		5		31	
		6		32	
		7		33	
		8		34	
		9		35	
		10		36	
		11		37	
		12		38	
		13		39	
		14		40	
		15		41	
		16		42	
		17		43	
		18		44	
		19		45	
		20		46	
		21		47	
		22		48	
		23		49	
		24			
		25		TOTAL TIME (Seconds) =	
		26		AV. Travel TIME (seconds) =	

Average Travel Speed, Us =