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EVALUATION OF THE FUNDAMENTAL TRAFFIC RELATIONSHIPS AND THE SERVICE MEASURES OF URBAN ROADWAY SECTION IN FALLUJA CITY

A THESIS
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By
Rafal Ahmed Abbas Ali Al-Ani
(B.Sc. 2002)

Supervised by
Dr. Mehdi Ibraheem Thamer Al-kubaisy

Jamad Al-Ola 1431 A.H. **May 2010 A.D.**

بسم الله الرحمن الرحيم
{اللَّـهُ نُـورُ السَّـمَاوَاتِ وَالْـأَرْضِ مَثَـلُ نُـورِهِ كَمِشْكَاةٍ فِيهَا مِصْبَاحٌ الْمِصْبَاحُ فِـي زُجَاجَـةٍ الزُّجَاجَةُ كَأَنَّهَـا كَوْكَـبٌ دُرِّيٌّ يُوقَـدُ مِـن شَـجَرَةٍ مُّبَارَكَـةٍ زَيْتُونِـةٍ لَّـا شَـرْقِيَّةٍ وَلَـا غَرْبِيَّـةٍ يَكَـادُ زَيْتُهَا يُضِيءُ وَلَـوْ لَـمْ تَمْسَسْـهُ نَـارٌ نُّـورٌ عَلَـى نُورٍ يَهْدِي اللَّهُ لِنُورِهِ مَن يَشَـاءُ وَيَضْـرِبُ اللَّـهُ الْأَمْثَالَ لِلنَّاسِ وَاللَّهُ بِكُلِّ شَيْءٍ عَلِيمٌ }
صدق الله العظيم

Dedication

To....My parents
...My husband

Rafal

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My thanks are due to God Who is my creator and responsible for all my accomplishments.

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ABSTRACT

This research represents an evaluation of roadway section. The objective of this study was to evaluate the fundamental relationships and service quality of traffic flow for an urban street section in Falluja city by developing a statistical models to predict speed-flow-density relationships.

Based on surveys, Al-Tharthar street section, which starts from Baghdad main road and ends at the roundabout facility which leads to Al-Saqlawia direction, was selected to satisfy the objectives and specifications of this study. Data were collected by using video-recording technique. The required data were abstracted by using the EVENT program. The abstracted data were analyzed, grouped, and processed using computer programs which were written using Visual Basic programming language, developed for this purpose. Standard statistical analysis techniques were used to examine and analyze the observed data.

The produced data indicate that the vehicles arrival follow Negative exponential distribution and Erlang distribution for the north bound and south bound for the two segments of the highway section respectively, while the speed frequency distributions show that they follow normal and log normal distributions for segment one and segment two, respectively.

Field surveys were done to collect new data that were abstracted and processed to build statistical regression model which was used to predict speed, flow, and density relationships. Collected data also used to evaluate the level of service of selected highway section.

The FWASIM simulation traffic software program was used to verify the statistical developed traffic stream model. The obtained results are presented in this research. To test the validity and reliability of the model, the output results of predicated model were compared with output data obtained from FWASIM model using similar input data and segment geometry. The comparison shows a good agreement between the statistical and simulated model results.

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CHAPTER ONE
INTRODUCTION

1.1 General

During the last years, highways played an important part in transport and there had been enormous increase in the number, speed, and length of trips. This increase had brought into focus many problems of the highway system such as congestion, casualty, deterioration, design deficiencies, inadequate surface, etc, which require various studies to be carried out to arrive to a suitable solutions for them.

People in most societies recognized the importance of transportation as man has always the desire to move from one place to another. This human movement, whatever its purpose was, resulted in improved transportation modes.

In the recent years, transportation problems have been attracting a lot of attention. Road transportation is one of the most critical means of transportation today, due to its direct impacts on its surroundings. While the traffic volume on highway transportation system is increasing at a tremendous rate, the growth in highway infrastructure is far slower .

Recent growth in urban areas has resulted in increasing demand for travel on the urban street. Falluja city has seen a general deterioration of level of service (LOS) on its urban streets. In many areas in Falluja city, urban streets are operating in congested conditions throughout much of the work day.

Under free-flow or (conditions under capacity), traffic flow is generally smooth with near constant speeds and uniform headway. Once demand exceeds capacity, traffic flow becomes unstable resulting in congested or stop-and-go conditions. Under these conditions, traffic flow occurs with variable headway and speeds that may increase or decrease significantly over a short section of highway.

Improvement of the quality service of highway traffic movement would contribute to reduce the congestion. Evaluating quality of service and fundamental traffic flow relationships through development regression model are one of the most important issues that are discussed in transportation analysis.

Speed, flow, and density are macroscopic parameters characterizing the traffic stream as a whole, while headway, gap, and occupancy are microscopic measures for describing the space between individual vehicles.

A comprehensive statistical analysis of macroscopic parameters at highway segments are essential requirements in planning, design, and operation of transportation systems. This study obtained values for speed, flow, density and predicted regression model to explain the traffic fundamental relationship between them.

Statistical technique is conducted to traffic flow represented by the headway distribution models, which may simply consist of a single statistical distribution that possesses properties of observed headway. For example Negative, Shifted negative exponential, Gamma and Log-normal distributions, or they may contain a mixture of two distributions. Four types of statistical distribution that possosses properties of arrival headway and speed were statistically examied in this study .

1.2 Objectives of Study Project

The objectives of this study project are to evaluate the operation condition of Al-Tharthar street section through:

1. Predicting local statistical models to represent the fundamental traffic flow relationships between speed, flow and density.
2. Evaluating the level of service of the selected highway section, depending on parameter values obtained from predicated models.

1.3 Thesis Layout

To achieve the objectives of the study, six chapters were conducted which represent the structure of this study. A brief description of the contents of each chapter is presented as in following:

Chapter 1: contains the introduction, objectives of the study project and the layout of thesis.

Chapter 2: is a review of literature for previous studies similar to this study.

Chapter 3: discusses the objectives, specifications, description of survey site which satisfy the specification sets, methods and procedures for data collection, in addition the abstraction and processing of the required data.

Chapter 4: discusses the presentation and statistical analysis of observed data to determine the parameters necessary for the prediction of statistical development models.

Chapter 5: describes developing a statistical model and evaluating quality service, prediction for speed-flow-density relationships and comparison the developed model output with **FWASIM** program.

Chapter 6: presents the conclusions, recommendations and recommendations for further studies.

CHAPTER TWO
REVIEW OF LITERATURE

2.1 Highway Classification

There are three primary federal highway functional classifications: arterial, collector, and local roads. All streets and highways are grouped into one of these classes, depending on the character of the traffic (i.e., local or long distance) and the degree of land access that they allow. These classifications are described in table (2-1). Typically, travelers will use a combination of arterial, collector, and local roads for their trips. Each type of road has a specific purpose or function. Some provide land access to serve each end of the trip. Others provide travel mobility at varying levels, which is needed en route. The functional classification system can be further broken down into “rural’ and “urban” classifications, and there are sub-classifications within these groupings as well, figure (2-1) shows the functional classification system (The Federal Highway Administration’s, 1989).

Table 2-1: Functional Classification Systems

Functional System	Services Provided
Arterial	Provides the highest level of service at the greatest speed for the longest uninterrupted distance, with some degree of access control.
Collector	Provides a less highly developed level of service at a lower speed for shorter distances by collecting traffic from local roads and connecting them with arterials.
Local	Consists of all roads not defined as arterials or collectors; primarily provides access to land with little or no through movement.

Source: (The Federal Highway Administration’s, 1989).

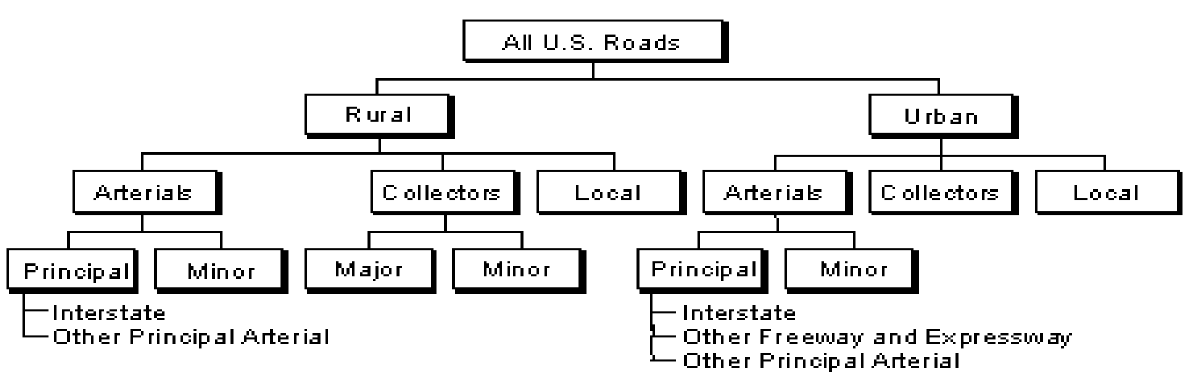

Figure 2-1: Functional Classification Systems

Interstate Highways are a formalized system of principal arterial in which there are no access points other than high speed ramps and are used for the longest trips. Freeway/expressway is a subclass of principal arterials that includes all other principal arterials with little or no direct access and very few at grade crossings. Other principal arterials have comparatively more access points and traffic impediments than freeway/expressway routes (Eric Foster, 2008).

Arterial streets are an important resource in urban areas .They represent significant expenditure of funds and they carry large volumes of traffic (Kuner, R., 1988).

The following are definitions for the arterial types of highways (AASHTO, 2001):

Expressway is divided arterial highway of at least two through traffic lanes for each direction of travel with full or partial control of access and generally with grade separations at intersection.

Freeway: Freeways are arterial highways with full control of access. They are intended to provide for high levels of safety and efficiency in the movement of large volumes of traffic at high speeds.

Through street or through highway is every highway or portion thereof on which vehicular traffic is given preferential right of way, and at the entrances to which vehicular traffic from intersecting highways is required by law to yield right of way to vehicles on such through highway in obedience to either a stop sign or yield sign, when such signs are erected.

2.2 Traffic Demand

A demand is the principal measure of the amount of traffic using a given facility. Thus, the term demand relates to vehicles arriving, while the term volume relates to vehicles discharging. If there is no queue, a demand is equivalent to the traffic volume at a given point on the roadway. Traffic demand varies by month of the year, day of the week, hour of the day, and sub hourly interval within the hour. These variations are important if highways are to effectively serve peak demands without breakdown. The effects of a breakdown may extend far beyond the time during which demand exceeds capacity and may take up to several hours to dissipate. Thus, highways minimally adequate to handle a peak-hour demand may be subject to breakdown if flow rates within the peak hour exceed capacity (HCM2000).

2.2.1 Traffic Volume

Usually traffic volume and flow are associated with traffic demand. Traffic volume is expressed by the number of vehicles using the facility at a particular interval of time, while traffic flow, is the hourly rate of traffic that is using that facility (Victor Muchuruza**,** 2003).

(Ling Qin and Dr. Brian Smith, 2001) defined the traffic volume as the number of vehicles on all the lanes of the freeway counted during the collection length. The collection lengths are between 100 and 255 seconds with the average of slightly more than 120 seconds. Traffic flow rate in vphpl is calculated by the formula below

Traffic flow rate (vphpl) = (volume /collect length in sec /no. of lanes) * 3600 sec/hr

The term volume generally is used for operating conditions below the threshold of capacity and it relates to vehicles discharging (HCM2000).

Traffic volumes studies are conducted to collect data on the number of vehicles and/or pedestrians that pass a point on a highway facility during a specified time period. This time period varies from as little as 15 min to as much as a year, depending on the anticipated use of the data, the traffic volume studies are usually conducted when certain volume characteristics are needed, some of are the following (Nicholas J. Garber, 2003, et al.):

1- **Average Annual Daily Traffic (AADT)** is the average of 24–hour counts collected every day in the year.
2- Average Daily Traffic (ADT) is the average of 24–hour counts collected over a number of days greater than (1) but less than a year.
3- **Peak Hour Volume (PHV)** is the maximum number of vehicles that pass a point o a highway during period of (60) consecutive minutes.
4- Vehicle Mile of Travel (VMT) is a measure of travel along a section of road .It is the product of the traffic volume (that is, average weekday volume or ADT) and the length of roadway in miles to which the volume is applicable.

(Gordon Wells, 1979) showed that there are three primary types of count in which we are interested when dealing with traffic flow:

- Non–directional count is a count of traffic passing a point, irrespective of which way it is traveling. This is the basic count, from which a road–use map of an area can be produce.

- Directional count is similar to non–directional count, but more detailed. This is the normal type of detailed count for an urban area, intended to provide particulars of traffic flow by direction; this would be particularly important if, for instance, we are watching the introduction of a one–way system.
- Classification count is essential to all form of traffic survey. Also it is necessary to know the composition of traffic flow for many design purposes**.**

While (Nicholas J. Garber, 2003, et al.) mentioned that there are different types of traffic volume counts carried out, depending on the anticipated use of the data to be collected:

- Cordon Counts.
- Screen line Counts.
- Intersection Counts.
- Pedestrian Volume Counts.
- Periodic Volume counts , which divided in to :
    - Continuous Counts.
    - Control Counts.
    - Coverage Counts.

2.2.2 Traffic Composition

Variability in traffic demand also occurs in the proportions of the different types of vehicles comprising the traffic stream .The relative proportions of vehicle types have an effect on traffic speeds and other operating characteristics.

Vehicles of different sizes and weights have different operating characteristics that should be considered in highway design. Besides being heavier, trucks are generally slower and occupy more roadway space. Consequently, trucks have a greater effect on highway traffic operation than do passenger vehicles. The overall effect on traffic operation of one truck is often equivalent to several passenger cars. The number of equivalent passenger cars is dependent on the roadway gradient and, for two-lane highways, on the available passing sight distance. Thus, the larger the proportion of trucks in a traffic stream, the greater the traffic demand and the greater the highway capacity needed. For uninterrupted traffic flow, as typically found in rural areas, the various sizes and weights of vehicles, as they affect traffic operation, can be grouped into two general classes (AASHTO, 2001):

• Passenger cars—all passenger cars, including mini-vans, vans, pick-up trucks, and sport/utility vehicles.
• Trucks—all buses, single-unit trucks, and combination trucks.

The entry of heavy vehicles other than passenger cars into the traffic stream affects the number of vehicles that can be served. Trucks, buses, and recreational vehicles (RVs) are the three groups of heavy vehicles. Heavy vehicles adversely affect traffic in two ways (HCM 2000):

• They are larger than passenger cars and occupy more roadway space; and
• They have poorer operating capabilities than passenger cars, particularly with respect to acceleration, deceleration, and the ability to maintain speed on upgrades.

(Madaniyo Mutabazi, 1999, et al.) mentioned in his study that the proportion of larger vehicles (mostly trucks) would be expected to influence the levels of traffic conflicts resulting from merging maneuvers. If two streams have the same flow but different proportions of larger vehicles, the average gap in the two streams will be larger for the stream with shorter vehicles, and the size of the gap the driver chooses to accept for merging determines the probability of the conflict.

(Victor Muchuruza, 2003) stated in his study that the mix of traffic is one of the vital inputs to the design of any traffic operational control strategy. Trucks create large gaps that cannot be effectively occupied by cars under normal passing maneuvers. On both six- and four-lane freeway sections, truck percentages are higher on the shoulder lanes. Likewise, on 4-lane freeway sections, truck percentages are higher in the shoulder lanes. The percentages of trucks in the median lane on six-lane freeway sections were below five percent while on four-lane sites trucks using the median lanes were between 5 and 19 percent of the total volume.

(Sijong Jo, 2003) stated that Highways are generally designed to serve a mixed traffic flow that consists of passenger cars, trucks, buses, recreational vehicles, etc. The impacts of these different vehicle types are not uniform; thus, create special problems in highway operations and safety.

2.2.3 Traffic Arrival

Vehicle arrivals at any point are regulated by the sequence of time intervals or headway between successive vehicles in a stream. The distribution of time headways is one of the most fundamental quantities available to traffic flow theory, and many attempts have been made to develop models of headway distributions (Al-kubaisy, 2004).

2.2.3.1 *Random Arrival*

Much has been written concerning the apparent randomness of vehicle arrivals at a point on a road. As early as Adams (Adams, 1936) collected observations on vehicle arrivals, showing that under normal conditions, freely flowing traffic corresponds very closely to a random series of events.

(Matthew J. Huber, 1976) stated that an event is said to occur randomly when each small increment of time (or space) is equally likely to contain an event.

A series of events is defined as random when each event is completely independent of any other event and when equal intervals of time are equally likely to contain equal number of events (Adams, 1936).

2.2.3.2 *Non Random Arrival*

The assumption of random arrivals is less effective when traffic is beginning to show signs of congestion (Highway Research Board, 1964).

This lack of agreement is explained by two factors affecting characteristic of travel (Al-kubaisy, 2004):

1. Vehicles have finite lengths and each headway, therefore, contains a real minimum time for a vehicle to travel its length.
2. Lack of passing opportunities causes platooning or bunching of vehicles. Successive headways are no longer independent and the random hypothesis breaks down.

Under such conditions, vehicles may be classified into two types, the first is following vehicles (constrained) and the second is non-following vehicles (free). In this case, free vehicles are those, which are not impeded by preceding vehicles and following vehicles, are those prevented from reaching their desired speeds by vehicles ahead. The balance between these two types varies with the flow rate of traffic stream. As the flow rate increases, the proportion of following vehicles increases (Highway Research Board, 1964).

2.3 *Headway Distribution Models*

(Baher Abdulhai, 2004, et al.) defined headway as a measure of the temporal space between two vehicles, or, more specifically, the time that elapses between the arrival of the leading vehicle and the following vehicle at the designated test point along the lane. Headway between two vehicles is measured by starting a chronograph when the front bumper of the first vehicle crosses the selected point and subsequently recording the time that the second vehicle’s front bumper crosses over the designated point. Headway is usually reported in units of seconds.

Average value of headway is related to macroscopic parameters as follows:

$$h = \frac{1}{q} \quad \text{or} \quad q = \frac{3600}{h} \qquad \text{(2-1)}$$

where:

q = rate of flow in veh/hr.

h = average headway in sec.

(Al-kubaisy, 2004) mentioned that headway distribution models may simply consist of a single statistical distribution that possesses properties of observed headway, for example Shifted negative exponential, Gamma and Log-normal distributions, or they may contain a mixture of two distributions, representing headways of following and non following vehicles.

The mixed distribution models attempt to describe a distribution *f(t)* of all headways in terms of the distribution of following and non following headways, *g(t)* and *h(t)*, such that:

$$f(t) = \psi g(t) + (1-\psi)h(t) \qquad \text{(2-2)}$$

where:

ψ =the proportion of the following vehicles.

(R. J., Salter, 1990) stated that times headways are the time intervals between the passages of successive vehicles past a point on the highway. Because the inverse of the mean time headway is the rate of flow, headways have been described as the fundamental building blocks of traffic flow. When the arrival of vehicles at a particular point on the highway is described, the distribution may either describe the number of vehicles arriving in a time interval or the time interval between the arrival of successive vehicles, the first type is the counting distribution and the second type is the gap distribution.

(H., W., and Mosher, 1961) studied the relationship between the counting and the gap distribution but usually it is the gap distribution that is studied, it requires a short period of observation to collect data for the investigation of gap distributions than for an investigation into counting distribution; in the study of intersection capacity, gaps in the major roads flow are used by minor road vehicles to inter the major road and, once again, it is the gap distribution that is of importance.

One of the earliest headway distributions proposed for vehicular traffic flow was proposed by (R. Tapio Luttinen, 1996; Adams, 1936) who suggested that the Negative exponential would be a good fit to the cumulative gap distribution. Adams illustrated the validity of the Negative exponential distributions by observations of traffic flow in London, when this distribution represents the cumulative headway distribution then arrivals occur at random and the counting distribution may be represented by the Poisson distribution.

(Matthew J. Huber, 1976) mentioned that, statistical distributions can be classified in two general categories:

1. Counting or discrete distributions.
2. Interval or gap distributions.

2.3.1 Counting Distributions

Counting of the number of events that occur in a given time period is relatively easy and has been a useful tool of the transportation engineer, and consist of four types:

2.3.1.1 Poisson Distribution

The Poisson distribution is used to describe discrete events that are truly random, and historically was the first distribution to be applied to an analysis of vehicle flow .The distribution gives the probability of (x) events during a single trail using the single parameter m, where m is the average number of events in each trail.

$$p(x) = (m^x * e^{-m}) / x! \quad \text{.........................(2-3)}$$

where:

$x = 0,1,2,..........$

$p(x) =$ probability that x vehicles will arrive during a counting interval t

$m = \lambda * t =$ average number of vehicles during a period of duration t

λ = average rate of arrival, veh/s (flow rate).

t = duration of each counting interval, in sec

$e =$ natural base of logarithms.

Calculations may be simplified by noting that:

$$p(0) = e^{-m} = e^{-\lambda t} \quad \text{................................... (2-4)}$$

$$p(x) = \frac{m}{x} * p(x-1) \quad \text{...................................... (2- 5)}$$

For the Poisson distribution, the mean and variance are each equal to *m*, so that the ratio mean/variance=1.

2.3.1.2 Binomial Distribution

As traffic flow becomes congested, the flow becomes more uniform, so that the variance of the number of vehicles per interval is decreased. Consequently, the ratio mean/variance is greater than 1. The Binomial distribution gives the probability of x events in *n* trail as following:

$$p(x) = \frac{n!}{x!(n-x)!} p^x q^{(n-x)} \quad \text{(2-6)}$$

where:

$x = 0, 1, 2\ldots$
$p(x)$ = probability of x events in n trails.
n = number of trail (each one second interval is a trail).
x = number of events in *n* trail.
p = probability of an event on any given trail = probability that any one second interval will contain a vehicle.
q = probability of a failure on any given trail =1- p = probability that any one second interval does not contain a vehicle.

The two parameters of the Binomial distribution are estimated as follows:

$$p = \frac{\bar{x} - s^2}{\bar{x}} \quad \text{(2-7)}$$

$$n = \frac{(\bar{x})^2}{\bar{x} - s^2} \quad \text{(2-8)}$$

where:

$\bar{x}$ = mean number of events per *n*- second interval.
s^2 = variance in the number of events per *n*–second interval.

Calculations may be simplified by noting that:

$$p(0) = q^n \quad \text{(2-9)}$$

$$p(x) = \frac{(n+1-x)}{x} * \frac{p}{q} * p(x-1) \quad \text{(2-10)}$$

The mean value of *x* is *np* and the variance of x is *npq*

2.3.1.3 Negative Binomial Distribution

The Negative binomial distribution follows from Binomial distribution and gives the probability that *x* failures occur in n trail before getting *k* events. For example, if traffic stream is made up of a mixture of cars and trucks, the passage of a passenger car will be considered a failure, while the passage of a truck will be considered a successful event. This distribution may be stated as:

$$p(x) = \frac{(x+k-1)!}{x!(k-1)!} * p^k * q^x \quad \text{……..…….. (2-11)}$$

where:

x = 0, 1, 2…

$p(x)$ = probability of *x* failures in *n* trails before getting *k* successful events.

p = probability of success on any given trail, and *q= 1- p*.

k = number of successes in n trail where the last trail is a success.

Calculations may be simplified by noting that:

$$p(0) = p^k \quad \text{……………………...…………………..…… (2-12)}$$

and

$$p(x) = \frac{(x+k-1)}{x} * q^{p(x-1)} \quad \text{…………………………… (2-13)}$$

The mean value of *x* is = *kq/p* and variance of *x* = *kq/p²*.

Where there is a cyclic variation in flow or where the mean flow is changing during the counting period, giving a mean/variance ratio substantially less than (1), use the Negative binomial distribution where parameters are estimated as following:

$$p = \frac{\bar{x}}{s^2} \quad \text{……………………..……………….…… (2-14)}$$

$$k = \frac{(\bar{x})^2}{s^2 - \bar{x}} \quad \text{……………………………………… (2-15)}$$

2.3.1.4 Generalized Poisson Distribution

It is a counting distribution for the case where flow is between uniform and purely random which has been suggested by (H., W., and Mosher, 1961) because of its association with the Erlang distribution, which in turn is a useful interval distribution. The Generalized Poisson distribution is given by:

$$P(x) = \sum_{j=xk}^{(x+1)k-1} (e^{-\lambda t} * \lambda t^{j}) / j\,! \quad \text{.................................} \quad (2\text{-}16)$$

where:

$P(x)$ = probability that x vehicles will arrive during a counting interval t.
k= number of terms of the Poisson series associated with x.
λ= average rate of arrival, vehicles/time (flow rate).
t= duration of counting interval.

The selection of the two parameter (λt) and (k) can be done by calculating the mean and variance and then using the nomograph contained in Height's work, if k is chosen, the other parameter, λt can be estimated from the relationship:

$$\lambda t = \bar{x}k + 0.5(k-1) \quad \text{..............................} \quad (2\text{-}17)$$

where:

$\bar{x}$ is the sample mean.

2.3.2 Interval Distributions

(Matthew J. Huber, 1976) stated that there is also a distribution of intervals or gaps between the arrivals of successive vehicles. These intervals will be in time units and are continuous variables as opposed to the discrete variable obtained from counting distribution.

Three statistical distribution of traffic are discussed herein the following sections:

2.3.2.1 Negative Exponential Distribution

(R. Tapio Luttinen, 1996) defined the Negative exponential distribution as the interval time distribution of a totally random arrival process i.e. the Poisson process. The distribution is widely used in the theory of point processes. In traffic flow theory, the Negative exponential distribution has been used since Adams (1936). The frequency of unrealistically short headways in the Negative exponential distribution is too large. In fact, extremely short headways have the highest probability density. The model gets more distorted as the flow rate increases, the Negative exponential distribution can be considered as a model for vehicle headways under very low flow conditions and for applications that are not very sensitive to the shape of the headway distribution. This distribution may be derived as follows:

$$P(n = 0) = e^{-qt} \quad \text{......................................} \quad (2\text{-}18)$$

where:

$p(n = 0) =$ the probability of no vehicle arrival during (t) seconds.

q = the vehicle flow measured in vehicle/second.

(Al-kubaisy, 2004) mentioned that from the above relationship, it may be seen that (under conditions of random flow) the number of headways greater than any given value will be distributed according to an exponential curve.

2.3.2.2 Shifted Negative Exponential Distribution

(Matthew J. Huber, 1976) referred to small time headways which are very unlikely to occur in vehicles observed in a single traffic lane, but the Negative exponential distribution predicts the highest probabilities for short time headways, a region in which headways are prohibited.

The Shifted exponential distribution avoids the problem of extremely short headways predicted by the Negative exponential distribution by setting a threshold for short headways. The Shifted exponential distribution can be considered for low flow conditions, or for applications that are sensitive to extremely short headways but not to other properties of the headway distribution**.** The Shifted exponential distribution is not a reasonable model for headways, but its properties are best under low flow rates (R. Tapio Luttinen, 1996).

The Shifted exponential distribution has been widely used in simulation studies (Lin F.1985*b*; Lin F.1985*a*; Shawaly, 1988).

(R.J., Salter, 1990) mentioned that it is important to note a difficulty of the use of the Negative exponential distribution even under free-flow conditions is that the probability of observing headway increase as the size of the headway decrease. But vehicles have a finite length and a minimum following headway, this presents a problem when only a limited number of over takings are observed, for this reason traffic flow has been described by the use of displaced (Shifted) Negative exponential distribution to describe the observed data.

The equation, which describes this distribution, is given below.

$$P(h \geq t) = e^{-q(t-\tau)/(1-q\tau)} \quad \text{.............................(2-19)}$$

where:

q and t as described herein before, and

τ = the minimum following headway.

2.3.2.3 Erlang Distribution

The Shifted negative exponential distribution makes the probability of headway less than the shifted distance to the right of the distribution which is equal to zero. A more desirable distribution, one that would have a very low, but not zero, probability of a small headway, is the Erlang distribution. The Erlang distribution is the interval distribution associated with the Generalized Poisson distribution in the same manner that the Negative exponential interval distribution is associated with the Poisson counting distribution. The probability density function of the Erlang distribution can be stated as (Matthew J Huber, 1976):

$$f(t) = \lambda e^{-\lambda t} * \frac{(\lambda t)^{-1}}{(k-1)!} \quad \text{.................................... (2-20)}$$

where:

λ = the rate of flow, the reciprocal of the mean time headway.

t = the time interval.

k = an integer constant

The mean time headway ($\bar{u}$) and the variance of headway (s^2) of Erlang distribution are calculated as follows:

$$\bar{u} = \frac{k}{\lambda} \qquad \text{and} \qquad s^2 = \frac{k}{\lambda^2}$$

(R. Tapio Luttinen, 1996) defined the Gamma distribution as a generalization of the exponential distributions. A special case of the Gamma distribution, namely the Erlangian distribution, has been widely used in the traffic flow theory.

(R.J., Salter, 1990) described the probability density function of the Erlang distribution by equation:

$$f(t) = \frac{(qa)^a}{(a-1)!} * t^{a-1} * e^{-aqt} \quad \text{.................................. (2-21)}$$

where:

$a = 1, 2, 3 \ldots$

q is the rate of flow (veh/sec), the reciprocal of the mean time headway

The parameter a is an integer.

Mean time headway ($\bar{u}$) = a/b.

Variance (s^2) = a/b^2.

The value of the parameter (a) in this distribution reflects the distribution of headways for a rang of traffic flow conditions, when a=1 the distribution becomes Negative exponential and when a is infinite complete uniformity of headways results, the value of a thus reflecting flow conditions between free flowing and congested states, ($a \geq 2 \rightarrow$ Erlang distribution) .

2.4 Normal Distribution

(Nicholas J. Garber, 2003, et al.) stated that Normal distribution as a distribution that describes the speed distribution over a given section of highway. The Normal distribution is given as:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} * e^{-(x-\mu)^2/2\sigma^2} \quad \text{for } -\infty < x < \infty \text{ .... (2-22)}$$

where:

μ = true mean of population.

σ = true standard deviation.

σ^2 = true variance.

The properties of Normal distribution are used to determine the minimum sample size for an acceptable error of the estimated speed, the basic properties of this distribution are:

1. The Normal distribution is symmetrical about the mean.
2. The total area under the Normal distribution curve is equal to (1) or 100 percent
3. The area under the curve between $\mu + \sigma$ and $\mu - \sigma$ is 0.6827.
4. The area under the curve between $\mu + 1.96\sigma$ and $\mu - 1.96\sigma$ is 0.95.
5. The area under the curve between $\mu + 2\sigma$ and $\mu - 2\sigma$ is 0.9545.
6. The area under the curve between $\mu + 3\sigma$ and $\mu - 3\sigma$ is 0.9971.
7. The area under the curve between $\mu + \infty$ and $\mu - \infty$ is 1.0000.

2.5 Log Normal Distribution

The Log normal distribution is the distribution of a variant whose logarithm obeys the normal law of probability (Van As, 1980). The form of this distribution, with mean (μ) and standard deviation (σ), for a random variable ln t (ln being the natural logarithm) is as follows:

$$f(t) = (\frac{1}{t\sigma\sqrt{2\pi}})e^{-(\ln t-\mu)^2/2\sigma^2} \quad \text{………………… (2-23)}$$

where:

$$\mu = e^{u+1/2\sigma^2} \qquad \text{and} \qquad \sigma = e^{2\mu+\sigma^2}(e^{\sigma^2}-1)$$

2.6 Speed Studies

(AASHTO, 2001) mentioned that speed is one of the most important factors considered by a traveller in selecting alternative routes or transportation modes. Travellers assess the value of a transportation facility in moving people and goods by its convenience and economy, which are directly related to its speed .The speed of vehicles on a road or highway depends ,in addition to capabilities of the drivers and their vehicles, upon four general conditions: the physical characteristics of the highway and the amount of roadside interference, the weather, the presence of other vehicles, and the speed limitations (established either by law or by traffic control devices).

Usually speed studies are done in free flowing conditions when vehicles interactions are minimal. According to the Highway Capacity Manual (HCM 2000), free flow condition occurs when there is a minimum of 4-second headway between vehicles.

(Mc Shane, 1998, et al.) mentioned that a minimum of 500 feet is required from a flow obstruction to the location where the speeds are measured. Also, to avoid bias in the sample data, the speed of traffic should be free from any concentrated law enforcement, just before or while taking speed measurements. Generally, the minimum sample size required is greater than 30 measured spot speeds in order to reduce variations and increase precision. On higher volume roads, the minimum sample size for analysis has to be at least 100 measure spot speeds, The following formula is used to determine the minimum sample size, n required for any statistical analysis.

$$n = (\frac{Z*S}{E})^2 \quad \text{……….……………………… (2–24)}$$

where:

Z = Confidence level.

S = Estimate of the standard deviation.

E = Range of error.

(Nicholas J. Garber, 2003, et al.) defined certain significant values that are needed to describe speed characteristics, they are:

- ***Time mean speed (Average speed)*** is the arithmetic mean of speeds of vehicles passing a point on a highway during an interval of time, as in equation below :

$$\bar{u}_t = \frac{\sum fi * ui}{\sum fi} \quad \text{.............................. (2-25)}$$

where:

$\bar{u}_t$ = Time mean speed or arithmetic mean.

fi = Number of observations in each speed group.

ui = Midvalue for the ith speed group.

- ***Space mean speed*** is the harmonic mean of speeds of vehicles passing a point on a highway during an interval of time , it is obtained by dividing the total distance traveled by two or more vehicles on a section of highway by the total time required by these vehicles to travel that distance .This speed is found by :

$$\bar{u}_s = \frac{N * L}{\sum t_i} = \frac{L}{\frac{1}{N}\sum \frac{L}{u_i}} = \frac{1}{\frac{1}{N}\sum \frac{1}{u_i}} \quad \text{........................ (2-26)}$$

where:

$\bar{u}_s$ = Space mean speed.

N = Number of observations.

t_i =The time for vehicle ***i*** to cross distance *L*.

- ***Median speed*** is the speed at the middle value in a series of spot speeds that are arranged in ascending order.
- ***Model speed*** is the speed value that occurs most frequently in a sample of spot speeds.
- ***The ith–percentile spot speed*** is the spot speed value below which ***i*** percent of the vehicles travel; for example 85th–percentile spot speeds the speed below which 85 percent of the vehicles travel and above which 15 percent of vehicles travel.

- ***Pace*** is the range of speed–usually taken at 10 mi/h intervals that have the greatest number of observations.
- ***Standard deviation of speed*** is a measure of the spread of the individual speeds, it estimates as:

$$S = \sqrt{\frac{\sum fi(ui - \bar{u})^2}{N - 1}} \qquad \text{.............................. (2-27)}$$

where:

S = Standard deviation.
ui = Midvalue of speed class i.
fi = Frequency of speed class i.
N = Number of observations.

2.7 Traffic Stream Models (Speed-Flow-Density relationships)

When considering the flow of traffic along a highway, three descriptors are of considerable significance (R.J., Salter, 1990). They are the speed and the density, which describe the quality of service experienced by the stream and the flow or volume, which measures the quantity of the stream and the demand on the facility. Numerous 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 (R.J., Salter, 1990). Usually the experimenters have been interested in the relationship between speed and flow because of a desire to estimate the optimum speed for maximum flow.

(Obeid, 2001) studied the development of mathematical models that deal with the three main characteristics of traffic: speed, flow, and density, in order to describe the intersection between these key variables for different types of facilities on selected arterial streets in Baghdad city. Linear regression analysis was applied to the speed, flow, and density data using Grapher (for windows) software .Also level of service for each section was evaluated by using the HCS-1994 software package.

(Nicholas J. Garber, 2003, et al.) noted that the traffic flow theory involves the development of mathematical relationships among the primary elements of a traffic stream: flow, density and speed; these relationships help the traffic engineer in planning, designing, and evaluating the effectiveness of implementing traffic engineering measures on a highway system.

(C. Jotin Khisty, 1998, et al.) mentioned as field measurements of speed, flow and density became available, several researchers evolved traffic flow models based on actual curve fitting and statistical testing, the evaluation of models proceeded a long two lines:

1- Relationships of speed-flow-density were tested in terms of goodness of fit to actual field data.
2- Relationships were supposed to satisfy certain boundary conditions:
- Flow is zero at zero density.
- Flow is zero at maximum density.
- Mean free speed occurs at zero density.
- Flow–density curves are convex (there is a point of maximum flow).

The following figures illustrate the typical speed–flow–density relationship:

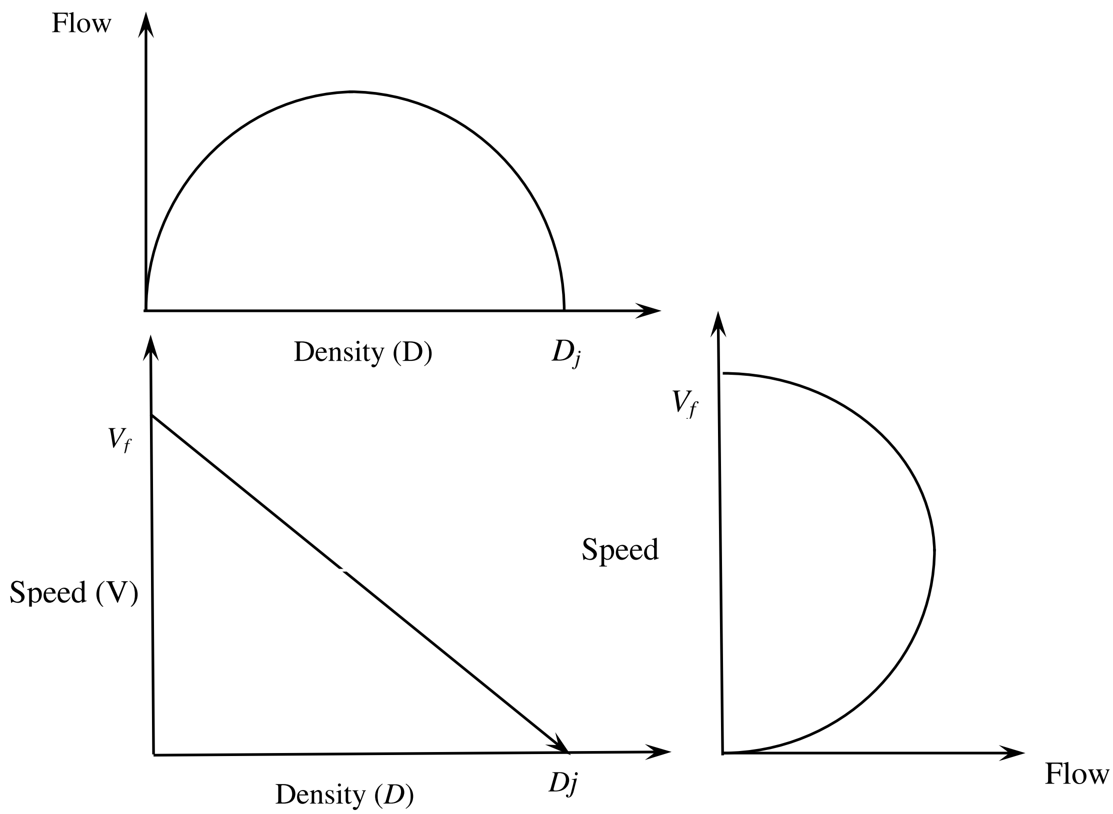

Figure 2-2: Speed–Flow–Density relationships (C. Jotin Khisty, 1998, et al.)

(Matthew J Huber, 1976) mentioned that the relationship among the three variables V, D and Q is called a traffic stream model; a dimensional analysis of the three variables gives the following relationship:

$$Q\ (veh/h) = V\ (mi/h) * D\ (veh/mi)$$

or ………………….(2-28)

$$Q\ (veh/h) = V\ (km/h) * D\ (veh/km)$$

where:

Q = mean rate of flow.
V = space mean speed.
D = mean density.

2.7.1 Speed-Density Models

It is an observable fact that drivers reduce their speeds as the number of vehicles increases (Highway Research Board 1974). Because of the close interaction between density and speed, and knowing density and speed, from which flow can be computed, early investigators explored relationships between speed and density.

Speed-density models consist of the following models (Matthew J. Huber, 1976):

1- Linear model (Greenshields).
2- Logarithmic model (Greenberg).
3- Generalized single–regime models
- Pipes–Munjal.
- Drew.
- Bell–shaped curve.

4- Multiregime models.

(C. Jotin Khisty, 1998, et al.) discussed so far the general model connecting speed, flow, and density which is a linear model proposed by Greenshields (1935); Greenshields’ model provides the slop and intercept by hand-fitting a straight line to plotted data or by using linear regression. This model satisfies all the four boundary conditions in the previous section, although the statistical quality may be poor (e.g., low coefficients of determination and high standard errors). The form of the model is:

$$V_s = V_f - (\frac{V_f}{D_j})D \quad \text{............................} \quad (2\text{-}29)$$

where:

V_s = the space mean speed.
V_f = the space mean speed for free flow conditions.
D = the density.
D_j = the jam density.

This model is simple and several investigators have found good correlation between the model and field data (Matthew J Huber, 1976). Figure (2-3) illustrates this linear relationship.

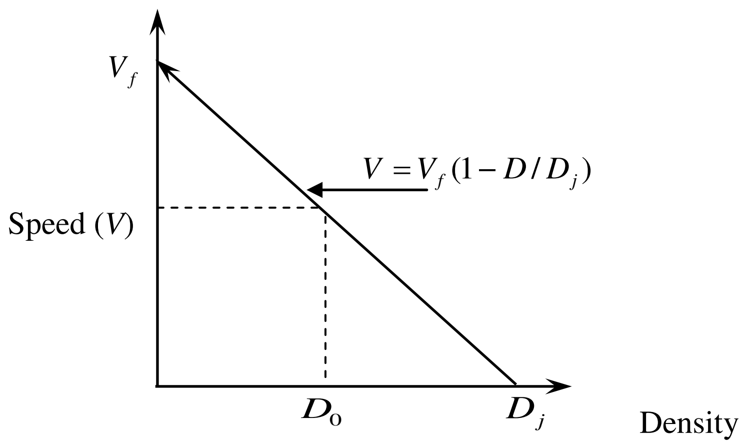

Figure 2-3: Greenshield's linear speed-density relationship

However a linear relationship does not always exist over the entire range of observations (Highway Research Board 1974). Some observers have suggested models that take the form shown in figure (2-4).

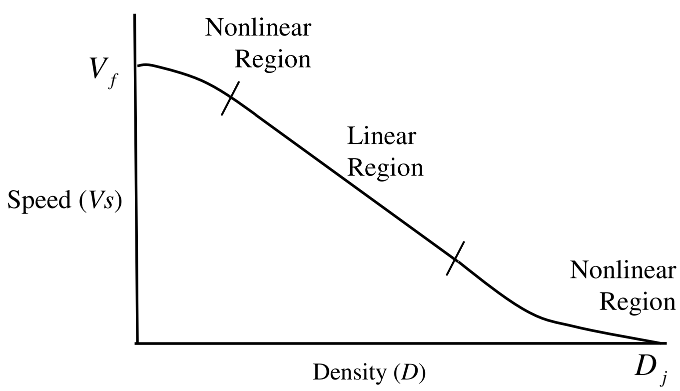

Figure 2-4: General type of speed-density curve obtained by field data

Greenberg (R.J., Salter, 1990) observed traffic flow in the north tube of the Lincoln Tunnel at New York City. He assumed that high density traffic behaved in a similar manner to the movement of continuous fluid. He found the developed speed–density model was of the form:

$$V_s = C \ln\left(\frac{D_j}{D}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \text{(2-30)}$$

where: C is constant and the remaining symbols are as previously defined.

This model shows good agreement with field data for congested flow condition, but is less satisfactory at low values of density, as may be seen by setting D near to zero in equation (2-30).

Greenberg's model shows better goodness -of-fit as compared to Greenshield's model; although it violates the boundary conditions in that zero density can only be attained at an infinitely high speed (C. Jotin Khisty, 1998, et al.).

(Nicholas J. Garber, 2003, et al.) referred to that Greenshields model satisfies the boundary conditions when the density D is approaching zero as well as when density is approaching the jam density, the Greenshields model therefore can be used for light or dense traffic .The Greenberg model satisfies the boundary conditions when the density is approaching the jam density, but it does not satisfy the boundary conditions when density approaching zero, the Greenberg model is therefore useful only for dense traffic condition.

(Underwood, R.T., 1961) proposed another form of Greenberg model:

$$V_s = Vf * e^{-D/D_m} \quad \text{.............................. (2-31)}$$

where:

D = the density.

D_j = the jam density or density at which all vehicles are stopped.

D_m = density at which the flow rate is maximum.

The shortcoming of this model is that it does not represent zero speed at high concentrations.

(L.A. Pipes, 1967) and (P.K. Munjal, 1971, et al.) have described a general family of speed density models of which the linear model is a special case. The proposed model is of the form:

$$V_s = V_f (1 - \frac{D}{D_j})^n \quad \text{.................................... (2-32)}$$

where:

n is a real number greater than zero.

It will be seen that for n=1 the relationship reduces to Greenshields'model.

(D.R. Drew, 1965) has described a family of models of which Greenshields'model is a special case. The other families result from car following analysis. He proposed a family of models of the form:

$$V_s = V_f \{1-(\frac{D}{D_j})^{(n+1)/2}\} \text{ …… for } n > -1 \quad \text{…………. (2-33)}$$

where: n is a real number.
when n=1 equation (2-33) can be solved to yield Greenshield's model.

(J.S. Drake, 1967) proposed the use of the bell-shaped or normal curve as a model of speed–density using the form:

$$V_s = Vf * e^{-1/2(D/D_m)^2} \text{ ………………………………… (2-34)}$$

(Tom V. Mathew, 2008) mentioned that all the above models are based on the assumption that the same speed-density relation is valid for the entire range of densities seen in traffic streams. Therefore, these models are called single-regime models. However, human behaviour will be different at different densities. This is corroborated with field observations which show different relations at different range of densities. Therefore, the speed-density relation will also be different in different zones of densities. Based on this concept, many models were proposed generally called multi-regime models. The simplest one is called a two-regime model, where separate equations are used to represent the speed-density relation at congested and uncongested traffic.

(Mark Burris and Sunil Patil, 2008) stated that Some popular two-regime models are listed in table (2-2), two-regime models have smaller mean deviations than those of the single-regime models. The Edie model slightly overestimates the maximum flow while the other two slightly underestimate the maximum flow. The linear two-regime model slightly overestimates free-flow speed and the modified Greenberg model slightly underestimates free flow speed. The optimum speed is slightly overestimated by the Edie model. All models underestimate jam density significantly.

Table 2-2: Equations and breakpoints for two-regime models

Two-Regime Model	Free-Flow Regime	Congested-Flow Regime
Modified Greenberg Model	Constant speed 48 ($k \leq 35$)	Greenberg model $U = 32Ln\left[\frac{145.5}{K}\right]$ ($k \geq 35$)
Two-regime Linear Model	Greenshield Model $U=60.9-0.515k$ ($k \leq 65$)	Greenshield Model $U=40-0.265k$ ($k \geq 65$)
Edie Model	Underwood Model $U = 54.9 * e^{\frac{-K}{163.9}}$ ($k \leq 50$)	Greenberg model $U = 26.8Ln\left[\frac{162.5}{K}\right]$ ($k \geq 50$)

Source: (Mark Burri and Sunil Patil, 2008)

2.7.2 Flow-Density Models

(Hight F.A., 1963) mentioned that the relationship between flow and density illustrated in figure (2-5) has been referred to as the fundamental diagram of traffic.

From the flow-density diagram which illustrated in figure (2-5), it can be clarified that as density increases, the flow increases to a maximum flow. This corresponds to the optimum density, D_0. Beyond the optimum density, the flow decreases with increasing density. The optimum density is the cut-off point between free-flowing and congested traffic conditions. The relationship between flow and density can be used to study the need for control to ensure that the optimum density is not reached so that the highway facility demand can be satisfied most of the time with a high level of service (Victor Muchuruza, 2006).

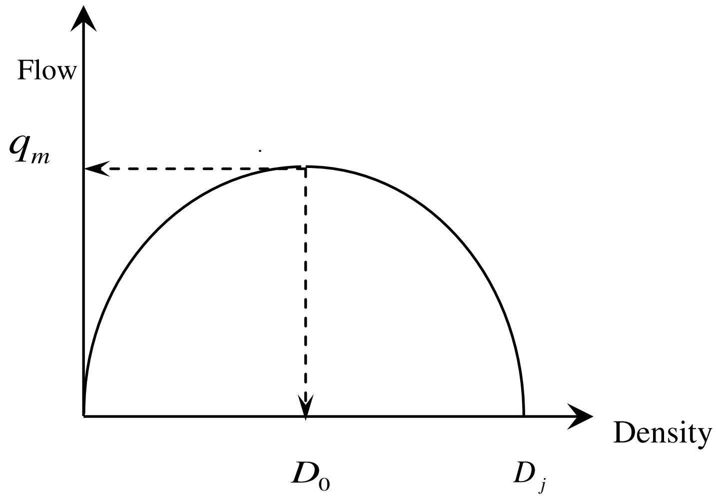

Figure 2-5: Typical flow-density diagram

The section bellow will deal with two models of flow-density relationship (Matthew J Huber, 1976):

1. The Parabolic Model

This model follows directly from Greenshield's model of speed-density. The model is concluded as follows:

$$Q = VD \quad \text{..............................} \quad (2\text{-}28)$$

By substituting equation (2-29) for V in the above relationship, it will result:

$$Q = V_f D - \frac{V_f}{D_j} D^2 \quad \text{..........................} \quad (2\text{-}35)$$

at maximum value (q_m) → $D_0 = \dfrac{D_j}{2}$ .... (2- 36)

To determine the maximum flow, substitute D_0 from equation (2- 36) in equation (2- 35), which will result in:

$$q_m = V_f * (D_j / 2) - (V_f / Dj) * (Dj / 2)^2 = V_f * Dj / 4 \quad \text{.........} \quad (2\text{-}37)$$

Some important features of the flow-density diagram may be summarized as follows (R.J., Salter, 1990):

1- The flow is obviously zero when the density is zero and at the jam density the flow may also be assumed to be zero.
2- Between these limits the flow must rise to at least one maximum, often referred to as maximum capacity, to give a shape of the approximate form shown in figure (2-5).
3- At any point on this curve, the slope of the line joining that point to the origin is the space mean speed, the slope is obviously greatest at origin and decreases to zero at the jam density.

2. The Logarithmic Model

The logarithmic model of flow-density follows directly from Greenberg's model of speed-density. Substituting equation (2-30) for V in equation (2-28) it will result:

$$Q = D^{*}C\ln\left(\frac{D_j}{D}\right) \qquad \text{(2-38)}$$

2.7.3 Speed-Flow Relationship

(Al-kubaisy, 2004) stated that speed-flow model can be determined, as the speed-density model has been determined. The free flow speed V_f at zero density is the maximum obtainable speed. There will be a second point of zero flow, corresponding to zero speed at maximum density D_j. Between zero and maximum speeds, the diagram will form some type of loop toward maximum flow. The speed curve associated with Greenshield's model, equation (2-29) is developed as below:

From equation (2-29): $V_s - V_f = V_f\left(\frac{-D}{D_j}\right)$

therefore: $D = D_j\left(1-\frac{V}{V_f}\right)$

substituting this expression for D in equation (2-28) gives

$$Q = D_j\left(V - \frac{V^2}{V_f}\right) \qquad \text{(2-39)}$$

The speed at maximum flow (V_0) will be (from eq. 2-28 and eq.2-37):

$$V_0 = \frac{q_m}{D_0} \quad \rightarrow$$

$$V_0 = \frac{V_f}{2} \qquad \text{(2-40)}$$

This model results in a parabolic speed-flow curve as shown in figure (2-6). This figure represents the general form of speed-flow relationship.

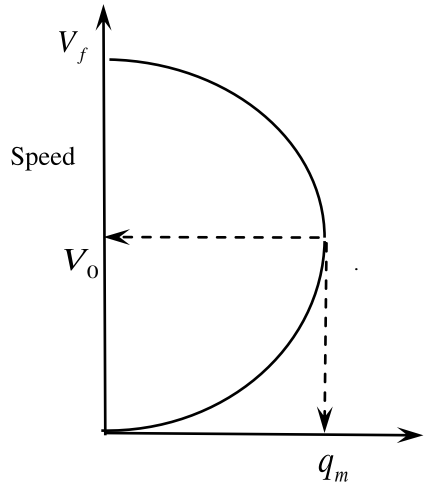

Figure 2-6: General form of speed-flow relationship

It is clear from figure (2-6) that in low flow (free-flow) conditions, speed decreases as the flow increases up to a maximum flow point (q_m). Furthermore, speed decreases with flow reductions because of the increase of traffic density beyond the maximum flow. This occurs because maximum flow can be reached in congested traffic conditions, and in these conditions the traffic flow is not stable, allowing for a decrease in speed should any perturbation occur in the traffic stream, The speed-flow relationship is used in the planning and design stages of transportation facilities to examine the capacity and quality of operation (level of service) that the designed facility is intended to provide (Victor Muchuruza**,** 2006).

2.8 Capacity and Level of Service

The definition of capacity published by (HCM, 1965) is the maximum number of vehicles that have a reasonable expectation of passing over a given roadway in a given time period under the prevailing roadway and traffic conditions.

(Carter .A, 1976) said that under ideal roadway and environmental conditions and with the most homogeneous group of drivers and vehicles ever likely to in normal highway usage, the capacity of a single traffic lane is approximately (2400) passenger vehicle per hour. This volume would be produced by average headways of 1.5 sec, such average headways have been observed for short time periods and on rare occasions for an entire hour in the central lanes of a freeway built to high design standers.

(Vuchic. V. R., 1981) stated that the capacity is independent of demand in the sense that it does not depend on the total number of vehicles (or whatever) demanding service. It is expressed in terms of units of some specific thing, however, so that it does depend on traffic composition (for instance, for highways, the percentage of trucks or other heavy vehicles), it is dependent on physical and environmental conditions, such as the geometric design of facilities or the weather.

(Horonjeff. R, 1983, et al.) stated that capacities of airports and rail systems are largely functions of their control systems. Highway systems, by contrast, involve very little positive control; as a result, their capacities and other flow characteristics depend heavily on driver behavior. The analysis of highway capacity is based primarily on empirical relationships, such as the speed-flow and flow-density relationships.

(HCM, 2000) defines Vehicle capacity as the maximum number of vehicles that can pass a given point in one or both directions during a specified period under prevailing (roadway, traffic, and control) conditions. This assumes that there is no influence from downstream traffic operation, such as the backing up of traffic into the analysis point.

The capacity that a highway can accommodate is limited by:

1. The physical features of a highway, which do not change unless the geometric design of the highway changes.
2. The traffic conditions, which are determined by traffic composition.
3. The ambient conditions which include visibility, road surface conditions, temperature and wind.

(HCM, 2000) defines Level of service (LOS) is a quality 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.

The highway capacity manual gives six levels of service and defines six corresponding volumes for numbers of highway types. These volumes are referred to service volumes (hourly volume) which may be defined as the maximum number of vehicles that can pass over a given section of a lane during a specified time period while operating conditions are maintained corresponding to the selected level of service.

The following are the various levels of services:

(A) Free flow and drivers are able to maintain their desired speeds with little or no delay.

(B) Stable flow, but operating speed beginning to be restricted by traffic conditions; still has reasonable freedom to select their speed and lane of operation.

(C) Stable flow, but operating speeds maneuverability are more closely by the higher volumes, lane changing, desired speed and overtaking are restricted and queue may begin to form .

(D) Approaching unstable flow with tolerable operating speeds being maintained though affected considerably by changes in operating conditions and queue may begin to form.

(E) Unstable flow with stop–start flow, speeds seldom exceed 50 km/hr and demand is at or near capacity.

(F) Forced flow takes place, and speeds are low and volumes are above capacity.

(HCM, 2000) explained the prevailing conditions that effecting on capacity and (LOC):

1. Roadway conditions, which consist of the following :
   - Geometric elements.
   - May influence capacity or performance measures such as speed.
   - Number of lanes.
   - Type of facility.
   - Lane widths.
   - Shoulder widths.
   - Lateral clearance.
   - Design speed.
   - Horizontal and vertical alignment which depend on design speed and topography.
   - Availability of exclusive lanes at intersections.
   - Severity of terrain reduces capacity and service flow rates for individual vehicles and passing vehicles.

2. Traffic Conditions, which consist of the following:
   - Vehicle type; the entry of heavy vehicles like (trucks, buses, recreational Vehicles) into the traffic stream affects the number of vehicles that can be served in two way:

     a- Heavy vehicles adversely affect traffic which they are larger than passenger cars and occupy more roadway space; also they have poorer operating capabilities than passenger cars, particularly with respect to acceleration, deceleration and ability to maintain speed on upgrades.

b- The second impact is more critical. The inability of heavy vehicles to keep pace with passenger cars in many situations creates large gaps in the traffic stream, which are difficult to fill by passing maneuvers.

- Vehicle type hindrances.
- Directional distribution (the optimal conditions when the amount of traffic is the same in each direction.
- Lane distribution (shoulder lane carries less traffic than other lanes).

3. Control Conditions, which consist of the following:
   - The most critical condition is traffic signal (signal phasing, allocation of green time, cycle length and relationship with adjacent controls).
   - Stop-Yield (capacity on minor street depends on traffic conditions on major street).
   - Curb parking (restriction increases number of lanes available).
   - Turn restrictions (eliminate conflicts increasing capacity).

2.9 Methods of Data Collection

(Pignataro L.J., 1973) mentioned that in any traffic survey, the limitations in the amount of money and time which can be devoted to the task and the questions of public inconvenience must be seriously considered.

(Baerwald, J. E., 1975) explained that it should be remembered that all the data collected must be analyzed for any use, more extensive and complicated survey will increase not only the costs of conducting the survey but also the subsequent analysis. The investigators must therefore have a clear understanding of the purpose of the survey before commencing the work, and the survey should be planned accordingly to give no more than the relevant data.

Generally counting may be carried out either manually or by automatic means:

2.9.1 Manual Methods

Those are the simplest, most direct and in certain circumstances, the only satisfactory means of carrying out a volume survey (Davies, E, 1963).

(Pignataro L.J., 1973) stated that the manual counts make use of field observers to obtain volume data and therefore they are subject to the limitations of human factors generally precluding 24-hours' continuous counts. In fact, they are important for periodic checking of the accuracy of automatic devices.

In addition, unusual conditions occurring during the time of the count cannot be recorded by the manual methods.

In other words, (Palanjian, H .S., 1986) mentioned in his study that the manual methods that can give vehicle classification very easily are suitable for short terms and non–continuous count.

2.9.2 Automatic Methods

(Davies. E., 1963) referred to the passing of vehicles can be recorded automatically using traffic counters, a traffic counter consists of two principle parts, the detector to sense the passage of a vehicle and the counter to record the pulses to it from the detector.

Since simplicity and reliability are two of the main considerations in the day to day operation of portable traffic counters, the type which is most commonly used the pneumatic detectors (Glanville.W.H, 1965).

Detectors fall into several categories such as pneumatic detectors, electrical detectors, photoelectric detectors, radar detectors …etc (Taylor. I. G., 1987, et al.) .

Automatic methods of counting have the following advantages:

- They have a relatively low cost per hour of counting and little attendant labor is required, for installation and maintenance, and therefore measurements can be made for long periods (Ashworth. R., 1972).
- They are accurate if regularly inspected and maintained (Gordon Wells, 1979).
- Cheap for long periods of counting which cover installation costs (Gordon Wells, 1979).

The disadvantages of automatic methods are:

- Difficulties occur in finding suitable sites for the equipment in localities where willful damage is likely to be encountered (Ashworth, R., 1972).
- Installation expensive for short sample counts (Gordon Wells, 1979).
- Requires frequent and skilled attention (Gordon Wells, 1979).
- Cannot make classified counts (Gordon Wells, 1979).
- Cannot compensate for e.g. three–axled Lorries (Gordon Wells, 1979).
- Equipment is relatively expensive, particularly as several units are usually required at the same time and may then not be used for some months (Gordon Wells, 1979).

2.10 Video Recording Technique

(Lee D. Han, 1989, et al) mentioned that the CCTV (Closed Circuit Television) surveillance systems, installed at numerous locations in the highways, use television cameras to detect and verify incidents.

Advantages:

- CCTV systems can cover a large area, depending on installations.
- They have effective verification method.
- They provide reliable information.
- Current technology of cameras has made it possible to receive good video images in severe weather conditions.
- They can use video images in disseminating information to motorist.

Disadvantages:

- They have high capital cost to cover a large area.
- Incident identification is a function of coverage.
- They can be used for incident identification, but this is a labor-intensive process.

(Baculinao.N., 2001) mentioned that some agencies are cautious about the use of VIVDS (Video Image vehicle detection system) because of difficulties associated with the accurate detection of vehicles that are distant from the camera, experience with VIVDS in Texas indicates that acceptable advance detection can be achieved at distances up to 500 ft (as measured from the camera to the most distant point of advance detection).

(James Bonneson, 2002, et al.) mentioned in his study that video imaging vehicle detection systems (VIVDS) become an increasingly common means of detecting traffic at intersections and interchanges in Texas. This interest stems from the recognition that video detection is often cheaper to install and maintain than inductive loop detectors at multi-lane intersections; it is also recognized that video detection is more readily adaptable to changing conditions at the intersection. The benefits of (VIVDS) have become more substantial as the technology matures, its initial cost drop, and experience with it grows.

(James Bonneson, 2002, et al.) mentioned that (VIVDS) are used at intersection or interchange in the following conditions:

- When more than 12 stop-line detectors are needed at the intersection or interchange.
- When inductive loop life is short due to poor pavement or poor soil conditions.

- When extensive intersection reconstruction will last for one or more years.
- When the loop installation is physically impractical due to the presence of a bridge deck, rail road trucks, or underground utilities.
- When the pavement in which the loop is placed will be reconstructed in less than three years or during overlay projects at large intersections where the cost of replacing all loops exceeds the cost of installing the VIVDS.

(Shourie Kondagari, 2006) stated that the Road-based traffic surveillance systems may also include other types of monitoring devices such as Closed Circuit Television (CCTV), Video Image Detection Systems (VIDS), and sensors such as Remote Traffic Microwave Sensors (RTMS). CCTV's and VIDS systems are more efficient and cost effective traffic monitoring equipment that provides real-time traffic information, but are sensitive to all weather conditions.

There are many issues associated with camera location and field–of-view calibration that must be taken in the consideration of video recording technique (James Bonneson, 2002, et al.):

1. ***Camera location***: An optimal camera location is one that maximizes detection accuracy, as such; an optimal location is one that provides a stable, unobstructed view of each traffic lane as following:

- *Occlusion*: Detection accuracy is adversely affected by vehicle occlusion, occlusion refers to a situation where one vehicle blocks or obstructs the view of camera of a second vehicle, three types of occlusion are presented with most camera locations: adjacent lane, same lane, and cross lane.

- *Camera stability*: Desirable camera heights and offset are often limited by the availability of structures that can provide a stable camera mount, most VIVDS products have an image stabilization feature that can compensate for some camera motions; however, excessive camera motion can adversely affect detection accuracy.

2. ***Field of view calibration:*** calibration of the camera field of view is based on a one–time adjustment to the camera pitch angle and the lens focal length. An optimal field of view is one that has the stop line parallel to the bottom edge of the view and in the bottom one-half of this view; the optimal view includes all approach traffic lanes. The focal length would be adjusted such that the approach width, as measured at the stop line equates to 90 to 100 percent of the horizontal width of the view; the factors that must be overcome to obtain an optimal field of view are:

- *Sun Glare and Reflection* :
  Detection accuracy is significantly degraded by glare from the sun and sometimes from strong reflections. Sun glare represents direct sunlight entering the camera (typically during dawn and dusk hours), reflections emanate as "star" of bright light coming from vehicle corners or edge.

- *Image size* :
  Detection accuracy is dependent on the size of the detected vehicle as measured in the field of view. Accuracy improves as the video image size of vehicle increases. Image size in turn can be increased by increasing the lens focal length.

- *Other considerations* :
  Several additional design factors can compromise detection accuracy like light sources, power lines, and headlight glare.

2.11 Traffic Models

The literature survey provides a summery of the evaluation, comparisons, and experimental studies of the computer programs that are related to the primary objective of this study.

2.11.1 Highway Capacity Software (HCS2000)

HCS is developed and maintained by Mc Trance center, University of Florida, as a part of its user-supported software maintenance as a faithful implementation of the highway capacity manual (HCM) procedures.

(Shamel Ahmed, 2003) defined (HCS) as a public domain package that automates the procedures of Highway Capacity Manual (HCM). It is one of the most widely used software packages that include Basic Freeway segments, Ramps and Ramp junction, signalized intersections, urban and sub urban Arterials, Two lanes Highway and Multilane Highway. Mc Trans has upgraded modules of the highway capacity software (HCS) to add new features and to incorporate the modified procedures in the new Highway capacity Manual (HCM2000), the Transportation Research Board (TRB) distributed this exciting new release (HCS2000TM) immediately following the publication of the HCM2000. HCS2000 comprises a complete new release of the HCS, upgrading modules to incorporate procedures in both metric units and US to faithfully implement the HCM2000.

2.11.2 FWASIM (Freeway Weaving Area SImulation Model)

(Al-kubaisy, 2004) developed simulation model during the course of his study. It was written using visual basic programming language. The model is of periodically scanning type and simulates the traffic behavior at weaving area for highways at Baghdad. The model permits measurement of a full range of traffic characteristics and allows many alternative designs to be tested. This type of modeling of traffic is classified as microscopic because of its nature of handling the movements of vehicles and resolution. Every vehicle makes a decision depending on parameters of the leading vehicle and its own characteristics. It should be noted that the macroscopic characteristics such as the flow, density etc., are all aggregated from these individual vehicle interactions The model enables traffic stream models (speed, flow, density relationships) to be tested within the freeway weaving area.

2.11.3 CORSIM

CORSIM is a comprehensive traffic simulation package developed to model surface streets, freeway systems, and combined networks having simple or complex control conditions. The strengths of the model lie in its ability to simulate a wide variety of traffic conditions from signalized arterial corridors and freeway corridors to stop controlled intersections. CORSIM is also one of the most well-documented simulation tools available, due in large part to the continued validation and updating that have occurred over nearly 30 years of use. Its simulation capabilities include (Steven L. Jones, 2004, et al.):

- Arterial networks.
- Freeway and surface street interchanges.
- Pre-timed and actuated signals, coordination, and pre-emption.
- Freeway weaving sections, lane adds and lane drops.
- Stop and yield controlled intersections.
- Simulation of queue length, queue blockage, and spillback.
- Origin-destination traffic flow patterns and traffic assignment.
- Network animation (CORSIM, 2003).

2.11.4 AIMSUN

AIMSUN (Advanced Interactive Micro-Simulation for Urban and Non-Urban Networks) is a full function microscopic simulation tool with a broad range of simulation capabilities. Like CORSIM, it can simulate surface street networks, freeways, interchanges, weaving sections, pre-timed and actuated signals, stop controlled intersections, and roundabouts. It also has features not contained in CORSIM, such as full trip distribution capabilities, dynamic traffic assignment, real-time vehicle guidance, and 3-D animations. AIMSUN is used in conjunction with the traffic network graphical editor (TEDI) and is part of the Generic Environment for Traffic Analysis and Modeling (GETRAM) open simulation environment (Steven L. Jones, 2004, et al.).

CHAPTER THREE
DATA COLLECTION, ABSTRACTION AND PROCESSING

This chapter includes sections which explain available methods of traffic data collection in addition to the objectives and specifications of data collection for the purpose of this research project.

The selected sites for data collection are described and the encountered problems during their stages of data collection are presented .Traffic data were collected during peak periods. Data abstraction and processing are also presented in this chapter

3.1 Objectives

The Objectives of this chapter are***:***

One of the important objectives of this research is to predict statistical models which represent the fundamental traffic flow relationship .To achieve this objective, it was necessary to collect data about the traffic behavior during various conditions. Sufficient data had to be collected to assist in the prediction of these models.

In this chapter, data were collected using video recording techniques for all selected sites in peak hours for both directions during morning and evening periods .These collected data were then abstracted by the aid of computer programs developed by researcher and others for those purposes.

The data required fell into the following three main categories:

1. Traffic flow characteristics which include traffic volume and traffic composition.
2. Traffic speed data.
3. Highway geometry which includes the segment length, width, cross section, number of lanes and their widths.

3.2 Specification of Suitable Site for Data Collection

To collect correct and sufficient data that satisfy the requirements of statistical calculations and representations, it was necessary to select sites which have level of traffic flow, speed and section geometry that give realistic results that can be analyzed statistically, accordingly,

1. Selected sites should satisfy the followings (Al-Neami and Al-kubaisi,2000):

    - Existence of an accessible vantage point allows for data collection to be made without effect on the observed traffic behavior.

    - Vehicle flow varies over the times of the day and the days of the week.

    - Range in the percentage of vehicle movement types and traffic compositions is to be considered.

2. A short inspection visits were made to a number of possible highways in Falluja city. A check was made to ensure that the selected highway will conform to the listed above requirements.

Based on surveys, Al-Tharthar highway section, that starts from Baghdad main road and ends to the roundabout facility which leads to Al-Saqlawia direction was selected since it was found to satisfy the objectives and specifications of data collection. The location of this highway is presented in Appendix "A". A brief description of the highway section that was selected to collect data is presented in the following section:

3.3 Description of Survey Site

Al-Tharthar urban street section was selected to collect the required data .It consists of two segments separated by Al-Hadra Mosque intersection. A brief description of each segment is presented below:

Segment one: is located at Al-Tharthar highway section. The beginning of this segment starts from the main highway that leads to Baghdad and the end is at Al-Hadra Mosque. This segment has two directions; North Bound (NB) and South Bound (SB), they are divided by a concrete median with high (50) cm and the above width is (36) cm .A plan for this segment is presented in figure (3-1) which shows the dimensions, lane widths, shoulders and other geometric main features.

Segment two: is located at Al-Tharthar highway section. The beginning of this segment starts from Al-Hadra Mosque and the end is at Al-Saqlawia direction roundabout. This segment also has two directions; North Bound (NB) and South Bound (SB), they are divided by a concrete median with high (50) cm and the above width is (36) cm.

Plan of this segment is shown in figure (3-2) which shows the dimensions, lanes width, shoulders and other geometric features.

Note: For both segments, there are many local roads which link with them; but most of these local roads are temporary closed by concrete barricades due to the current security conditions imposed by the US Occupation forces and local authorities.

3.4 Method of Data Collection

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.

According to the currently available methods of traffic data collection, the video recording is the most suitable method due to the following factors:

1. Shortening time and cost.
2. The large size of required activities and traffic behaviors which may be achieved.
3. The manual and automatic methods allow only for partial information about traffic movements to be collected during an observation period.

So, the researcher uses video recording technique to collect the data, this technique has the following advantages (Al-kubaisy, 2004):

- Allowing for a large number of events to be recorded at one time.
- Recording any incident that may occur during the recorded session which may result in abnormalities in the observed data, such data session can be reviewed at a later stage and any apparent abnormalities in the data can be abstracted.

The data were collected using a digital video camera with direct display. The camera was put in front of the segments to collect the data associated with vehicle traffic flow parameter. A good vantage point was selected by researcher to give the best view to record the two segments. The camera was fixed through its stand located on the over pass bridge which was still not working. This over pass bridge lies on Al-Hadra intersection. The choice of this vantage point helped the researcher to ensure wide coverage areas for the two segments. The camera used was with wide range and high length of focal lens to get the best zoom. The camera was connected to spatial apparatus to transfer video recording from video taped to the personal computer to abstract and process observed data.

The data collected by video recording technique were made in sessions for each segment at peak hours in the morning and evening for each direction. The total duration of video recording for the two segments was (12) twelve hours; The data collected by video recording technique were made in cold weather and sunny days, but unfortunately some awkwardnesses happened during the periods of video recording, as in followings:

1. Big difficulties were faced to get permission from Falluja policemen station because it should be taken from the commander of Falluja policemen only, in addition to the shared operations room (GCC) which were controlled by the American occupied force.

2. Informing all the police stations that surround the area of survey and also all the police patrols that exist in the area of survey were to be informed that there is a process of video recording. Also obligation was signed in police station that the appearance of any military patrols is prohibited.

3. Disturbing the owners of houses that located within the area of survey because of the video recording, due to the long duration, which causes deprivation for people from their liberty and comfort.

4. Difficulties for finding best location for camera that is not affected by sun glare and reflection because detection accuracy is significantly degraded by glare from the sun and sometimes from reflections.

3.5 Data abstraction and processing

3.5.1 Data abstraction

The recorded data were abstracted from video films with the aid of computer program called "EVENT" program. This program was developed by Ali-H-Al-Neami (Al-Neami H. K., 2000). The program produces time accuracy values of the recorded data. The accuracy is up to 0.01 second. The procedure of how this program works is as follows:

The program starts counting time for successive events. The observer allocates a specified key to be pressed for each vehicle passed and vehicle type. Then a tag character is appeared for each pressed key together with the recorded time of the successive events. Data then can be stored for future use.

Another program was written using Visual Basic programming language to put the data in groups of intervals to simplify the statistical analysis of the data. Video films were played back a number of times to get the required information.

The abstracted data include the following:

1. Vehicular traffic flow and traffic composition.
2. Time headway between successive vehicles measured on a specific point on the segment.
3. Sufficient speed data measured for individual vehicles and should be statistically meaningful.

3.5.2 Data processing

3.5.2.1 Traffic Flow Data Processing

The abstracted data were processed by using again "EVENT" program and another computer program developed for this purpose. This program is called "VBANA". The program was written using Visual Basic language. It was developed to calculate the time headway between successive vehicles passing a specified point and arrange them in a class width to be satisfied for the calculation of the frequency distribution of the observed data.

The different number of vehicles and traffic composition can be distinguished from the tag character produced by the event program. This process was made for the two segments, both sides and for morning and evening periods. The output files were arranged in a manner to be understood easily. The video recording date of flow data collection was on Sunday in (16/11/2008).

The output files were classified according to the segment type, direction and period as follows:

1. Segment one, North Bound, Flow, Morning (SEG1NBFM) from (7:30-9:00) AM.
2. Segment one, North Bound, Flow, Evening (SEG1NBFE) from (3:45-5:30) PM.
3. Segment one, South Bound, Flow, Morning (SEG1SBFM) from (7:30-9:00) AM
4. Segment one, South Bound, Flow, Evening (SEG1SBFE) from (3:45-5:30) PM
5. Segment two, North Bound, Flow, Morning (SEG2NBFM) from (7:30-8:45) AM
6. Segment two, North Bound, Flow, Evening (SEG2NBFE) from (3:45-5:00) PM
7. Segment two, South Bound, Flow, Morning (SEG2SBFM) from (7:30-8:45) AM
8. Segment two, South Bound, Flow, Evening (SEG2SBFE) from (3:45-5:00) PM

3.5.2.2 Vehicular Speed Data Processing

The speed calculation is produced from dividing distance over time. To achieve that, the researcher located two points with known distance on the road before video recording, painted marks were put on these points which represent the start and the end for observed vehicles that their speed to be measured. The difference between these two points represents the distance that the observed vehicles traveled on it.

The time duration for each observed vehicle was calculated by "EVENT" analysis program through finding the difference between the two times of the start point and end point. Speed is then calculated for each observed vehicle by dividing the traveled distance by its time duration**.**

The researcher closed two hundred (200) random samples of observed vehicles to calculate their speeds for each direction (from both peak morning and evening) in each segment, which mean the total number of samples equal to eight hundred (800) vehicles. The results were put in files as following:

1. Segment one, North Bound, Speed, Morning and Evening [SEG1NBS (M+E)] from (7:30-9:00) AM and (3:30-5:00) PM.

2. Segment one, South Bound, Speed, Morning and Evening [SEG1SBS(M+E)] from (7:30-9:00) AM and (3:30-5:00) PM.

3. Segment two, North Bound, Speed, Morning and Evening [SEG2NBS(M+E)] from (7:40-9:10) AM and (3:15-5:00) PM.

4. Segment two, South Bound, Speed, Morning and Evening [SEG2SBS(M+E)] from (7:40-9:10) AM and (3:15-5:00) PM.

The above created files were used as an input files for "VBANA" program to group the speed data in intervals to be used for later data analysis and to conclude the required speed results.

The dates of video recordings for segment one and segment two in the peak hours of morning and evening periods for both directions were on Thursday in (27/11/2008) and Wednesday in (26/11/2008) respectively.

CHAPTER FOUR
DATA ANALYSIS AND RESULTS

4.1 *Observed Vehicle Arrival Headway Distributions*

Vehicle arrival distribution data were abstracted from the output file of EVENT analysis program and analyzed.

The analysis of the entire period of video recordings represent the two segments in the peak hours of morning and evening for both directions which produced various cases of flow. During the analysis of these cases, the data were classified and grouped in a number of classes, and presented graphically in the form of histograms as shown in the followings:

4.1.1 *Segment One*

4.1.1.1 *North Bound, Morning period*

Traffic flow data were analyzed depending on the processed data resulted from "EVENT" program. The classified and grouped data are shown in table (4-1), and presented graphically in the form of histogram in figure(4-1).

Table 4-1: Observed values of arrival headway for segment one, north bound, at morning period

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	70	11 – 12	11
1 – 2	251	12 – 13	15
2 – 3	206	13 – 14	13
3 – 4	117	14 – 15	6
4 – 5	80	15 – 16	7
5 – 6	65	16 – 17	6
6 – 7	55	17 – 18	3
7 – 8	35	18 – 19	7
8 – 9	26	19 – 20	5
9 – 10	27	20 – 21	1
10 – 11	18		

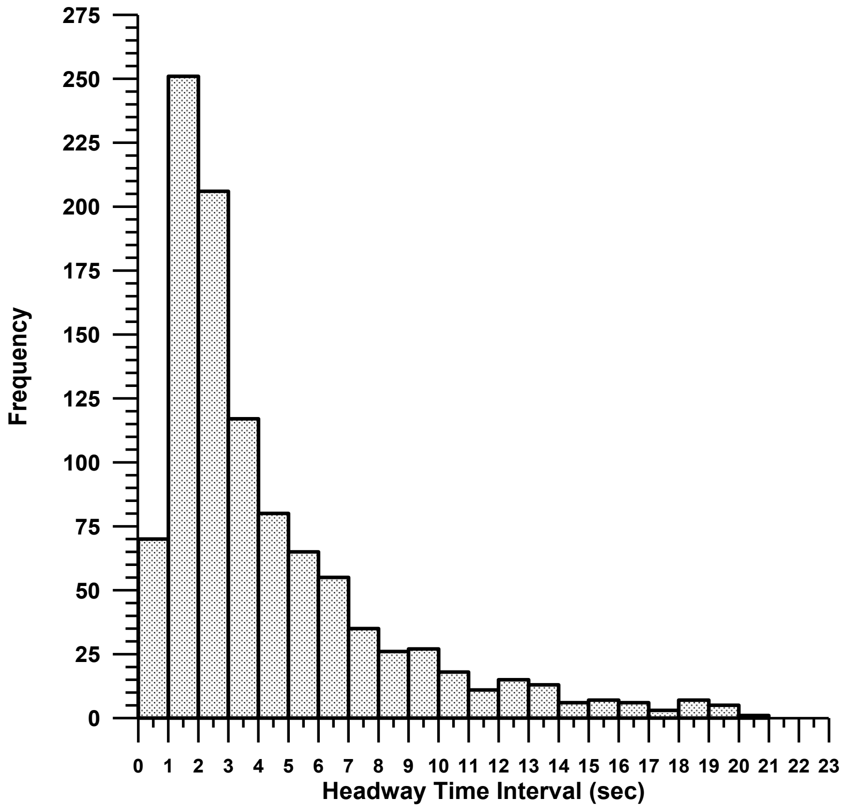

Figure 4-1: Observed arrival headway distribution for segment one, north bound, at morning period

4.1.1.2 *North Bound, Evening period*

The classified and grouped data for this case are shown in table (4-2), and presented graphically in the form of histogram as indicated in figure (4-2).

Table 4-2 : Observed values of arrival headway for segment one, north bound, at evening period

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	140	11 – 12	12
1 – 2	505	12 – 13	14
2 – 3	262	13 – 14	6
3 – 4	182	14 – 15	8
4 – 5	69	15 – 16	6
5 – 6	62	16 – 17	6
6 – 7	46	17 – 18	7
7 – 8	30	18 – 19	3
8 – 9	27	19 – 20	3
9 – 10	18	20 – 21	3
10 – 11	13		

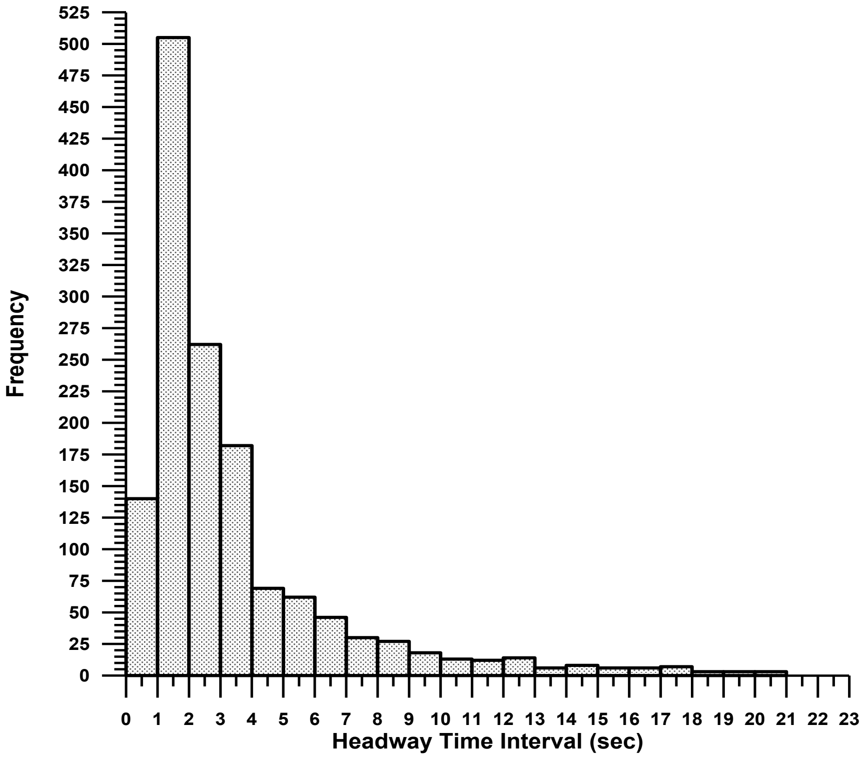

Figure 4-2: Observed arrival headway distribution for segment one, north bound, at evening periods

4.1.1.3 *North Bound, Morning and Evening periods*

Because of the lack of adequate values of data, the observed data for each morning and evening periods were combined. The reason is to get reasonable representation for the observed data. The observed data were classified and grouped in a number of classes as presented in table (4-3). They are also presented in the form of histogram as shown by figure (4-3).

Table 4-3: Observed values of arrival headway for Segment One, north bound, at morning and evening periods

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	210	11 – 12	23
1 – 2	756	12 – 13	29
2 – 3	468	13 – 14	19
3 – 4	299	14 – 15	14
4 – 5	149	15 – 16	13
5 – 6	127	16 – 17	12
6 – 7	101	17 – 18	10
7 – 8	65	18 – 19	10
8 – 9	53	19 – 20	8
9 – 10	45	20 – 21	4
10 – 11	31		

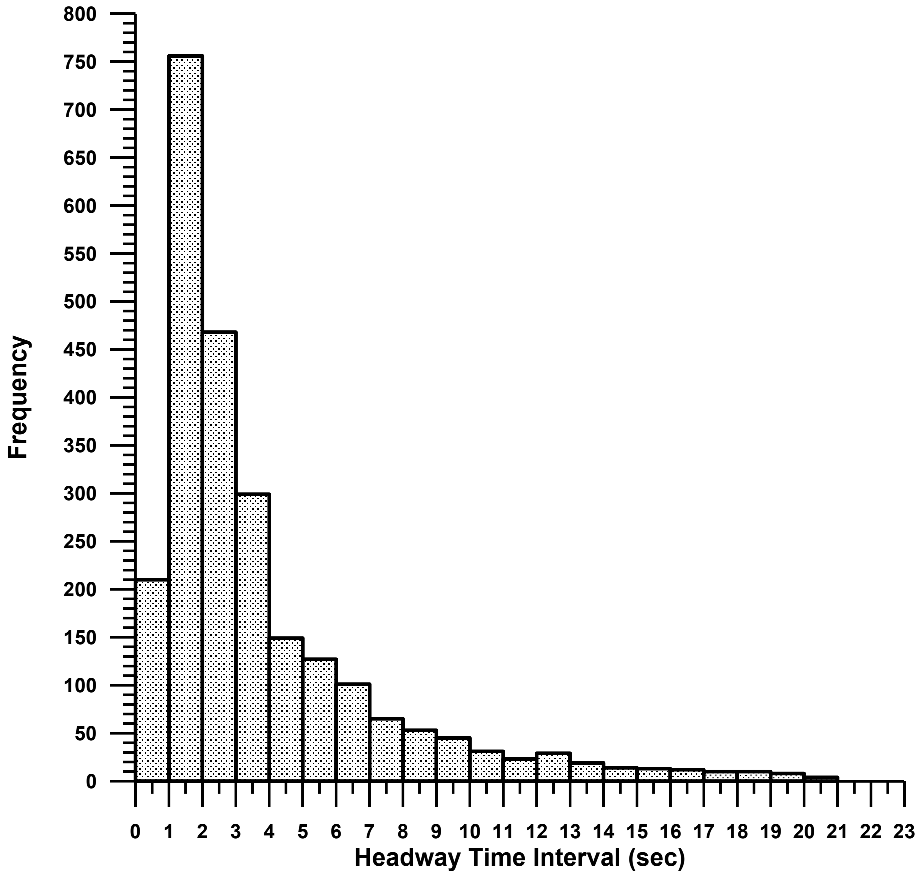

Figure 4-3: Observed arrival headway distribution for segment one, north bound, at morning and evening periods

The observed data were assumed to follow the Erlang distribution; the reason is the existence of very low probability of a small headway .To uses this distribution, it was necessary to determine the mean and variance of the average observed data. The mean and variance were found to be (3.949) and (15.857) respectively. They were then used to determine the value of (a) and (b) parameters which the Erlang equation depends on their values, but the Erlang distribution was not checked, the reason is the value of (a) ≤ 1. Appendix "B" shows the sample of calculations and their details. The alternative was to select another distribution. This time is the Negative exponential distribution. The mean and rate of flow of this distribution were found to be (2.8898) and (1246) veh/hr respectively.

The chi–square test was used to examine the difference between the observed and theoretical frequencies for this distribution. Results are presented in table (4-4). Examination of tables of chi–squared showed that at the (5%) level of significance and (6) degrees of freedom, the value of tabulated chi–squared is (12.592), while the calculated value is (211.21), which is higher than the tabulated value.

Table 4-4: Observed and theoretical values of arrival headway distribution of segment one, north bound, at morning and evening periods

Class width (sec)	Observed frequency		Theoretical frequency from negative exponential equation	
0	483		616.29	
2	384		308.5	
4	139		154.43	
6	82		77.302	
8	50		38.695	
10	27		19.37	
12	23		9.6961	
14	13		4.8536	
16	11		2.4296	
18	9	combined	1.2162	combined
20	5		0.6088	
22	4		0.3047	
24	4		0.21	
Calculated Chi-square=211.21 Tabulated Chi-square=12.592 Degree of freedom=6				

Result of analysis suggests that there is no good agreement between the observed and theoretical values of Negative exponential distribution depending on chi-square test. The disagreement is attributed to the following :

1. The existence of frequency of observed arrival vehicles in the class between (0–1) second .

2. As (R. Tapio Luttinen , 1996) stated, the main problem of goodness –of–fit tests in headway studies is that the chi–squared test is based on the grouping of data, this makes the tests less powerful than the tests based on the empirical distribution function .The goodness of fits has been tested by three methods :

    1. Graphical evaluation.
    2. Chi–square test.
    3. K–S test.

Therefore, the closeness of fit between the theoretical Negative exponential distribution and observed data can be demonstrated by the graphical method to show probable agreement.

Figure (4-4) shows this comparison. The comparison of the observed and calculated headway frequency is also shown in figure (4-5). The line x = y has been superimposed in the plot to facilitate comparison. The observed and calculated values are not matched exactly, however, they are closely enough to consider that the observed values may follow the Negative exponential distribution.

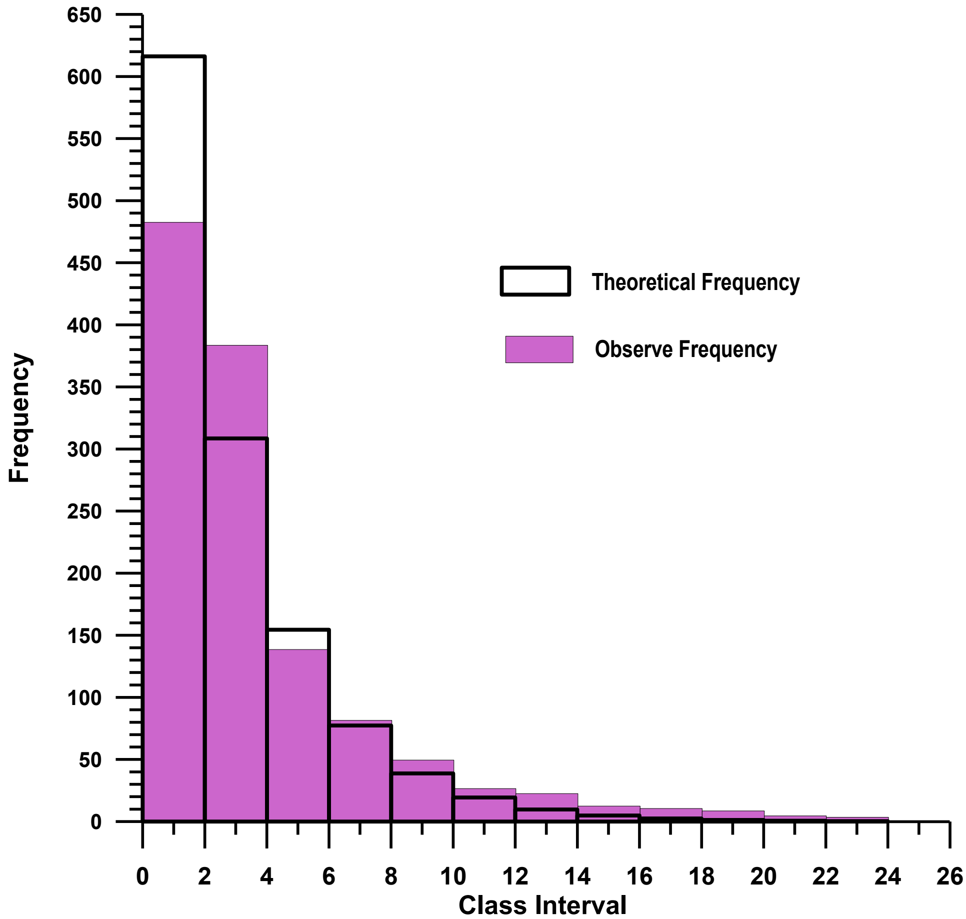

Figure 4-4: Comparison between observed and theoretical headway distribution for segment one, north Bound, at morning and evening periods

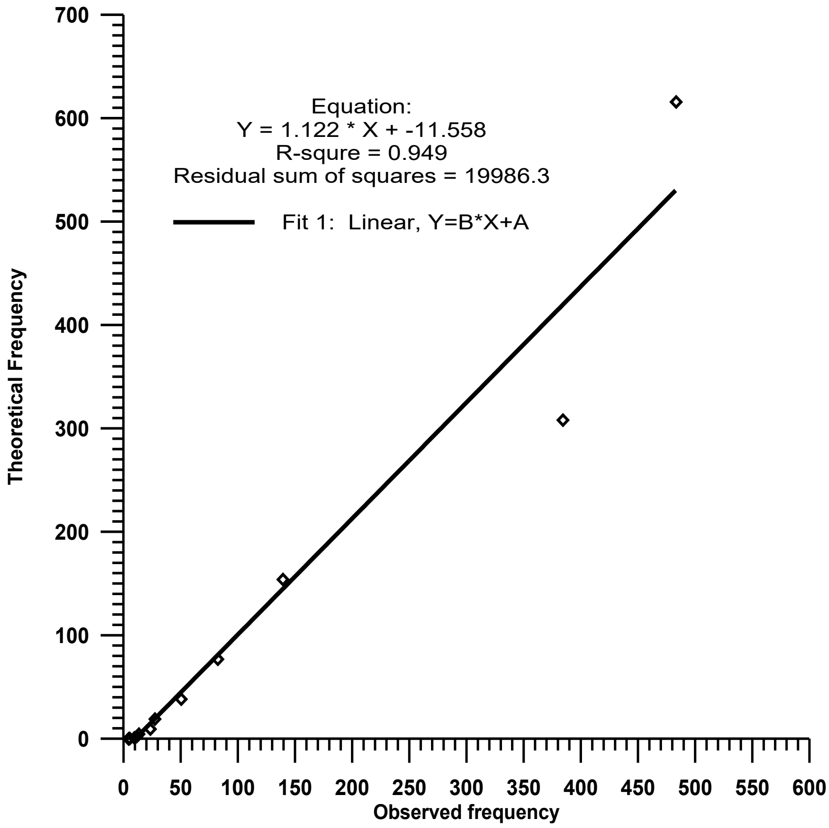

Figure 4-5: Comparison between the observed and theoretical headway distributions

4.1.1.4 *South Bound, Morning period*

Traffic flow data were analyzed depending on the processed data resulted from " EVENT " program. The classified and grouped data are shown in table (4-5), and presented graphically in the form of histogram in figure (4-6).

Table 4-5 : Observed values of arrival headway for segment one, south bound, at morning period

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	138	11 – 12	21
1 – 2	292	12 – 13	12
2 – 3	219	13 – 14	13
3 – 4	115	14 – 15	4
4 – 5	99	15 – 16	3
5 – 6	65	16 – 17	5
6 – 7	55	17 – 18	1
7 – 8	36	18 – 19	3
8 – 9	26	19 – 20	2
9 – 10	30	20 – 21	7
10 – 11	29		

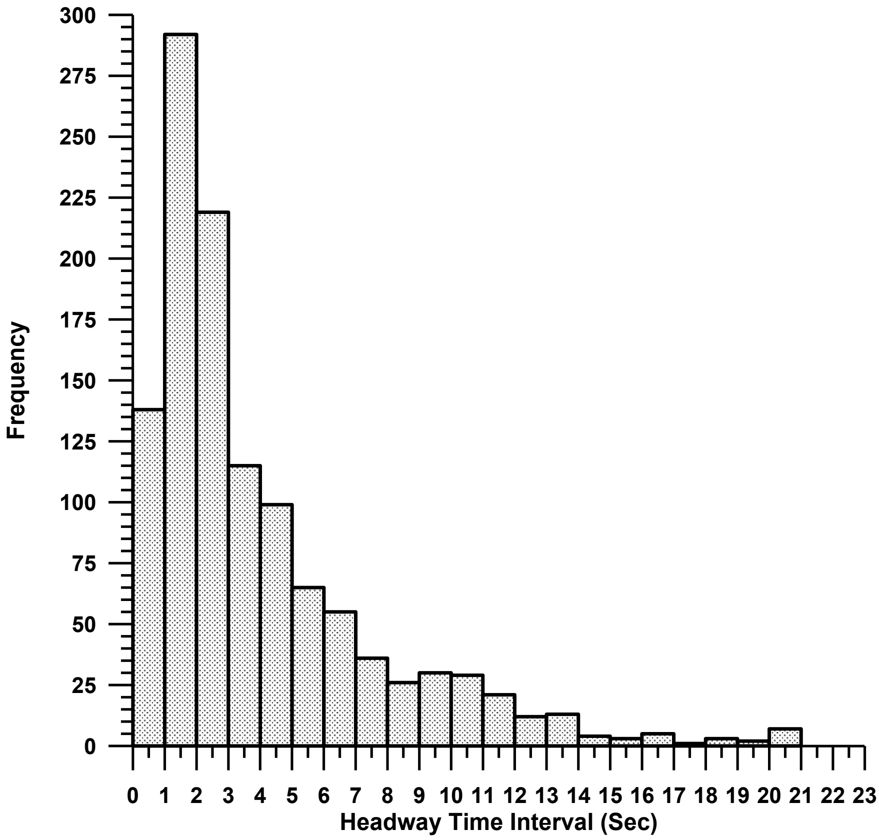

Figure 4-6: Observed arrival headway distribution for segment one, south bound, at morning period

4.1.1.5 *South Bound, Evening period*

In the same manner, the classified and grouped data for this case are shown in table (4-6), and presented graphically in the form of histogram as indicated in figure(4-7).

Table 4-6: Observed values of arrival headway for segment one, south bound, at evening period

Class width	Observed frequency	Class width	Observed frequency
0 – 1	25	11 – 12	13
1 – 2	202	12 – 13	12
2 – 3	240	13 – 14	10
3 – 4	194	14 – 15	13
4 – 5	88	15 – 16	10
5 – 6	62	16 – 17	7
6 – 7	46	17 – 18	8
7 – 8	31	18 – 19	9
8 – 9	36	19 – 20	2
9 – 10	21	20 – 21	5
10 – 11	15		

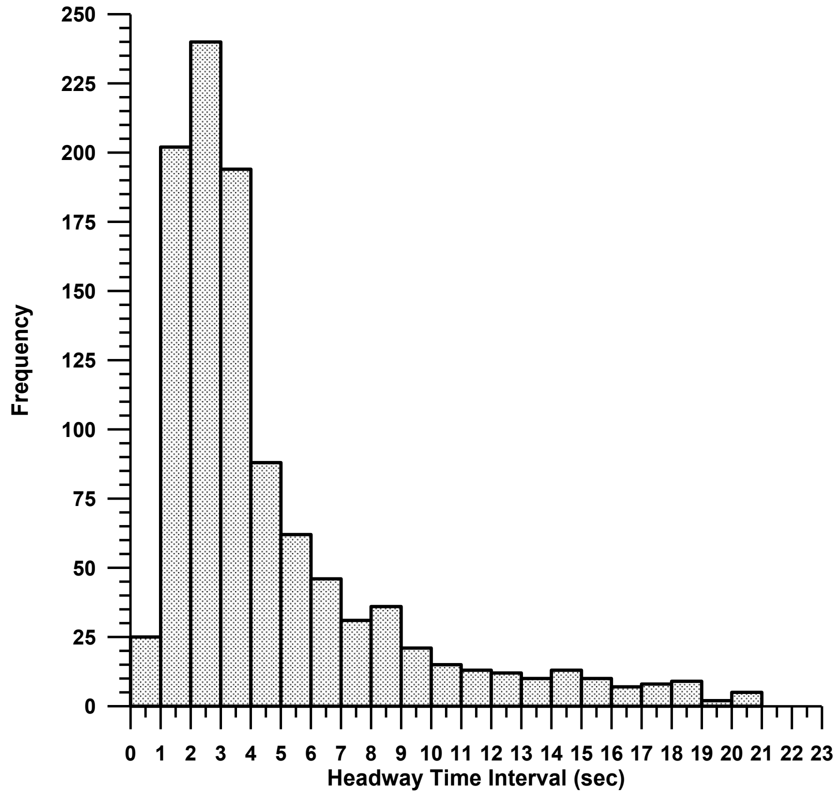

Figure 4-7: Observed arrival headway distribution for segment one, south bound, at evening period

4.1.1.6 *South Bound, Morning and Evening periods*

Because of the lack of adequate values of data, the combining operation of the observed data for both morning and evening periods was made in order to get acceptable and reasonable representation for the observed data. The observed data were classified and grouped in a number of classes as presented in table (4-7). They are also presented in the form of histogram as shown by figure (4-8).

Table 4-7: Observed values of arrival headway for segment one, south bound, at morning and evening periods

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	163	11 – 12	34
1 – 2	494	12 – 13	24
2 – 3	459	13 – 14	23
3 – 4	309	14 – 15	17
4 – 5	187	15 – 16	13
5 – 6	127	16 – 17	12
6 – 7	101	17 – 18	9
7 – 8	67	18 – 19	12
8 – 9	62	19 – 20	4
9 – 10	51	20 – 21	12
10 – 11	44		

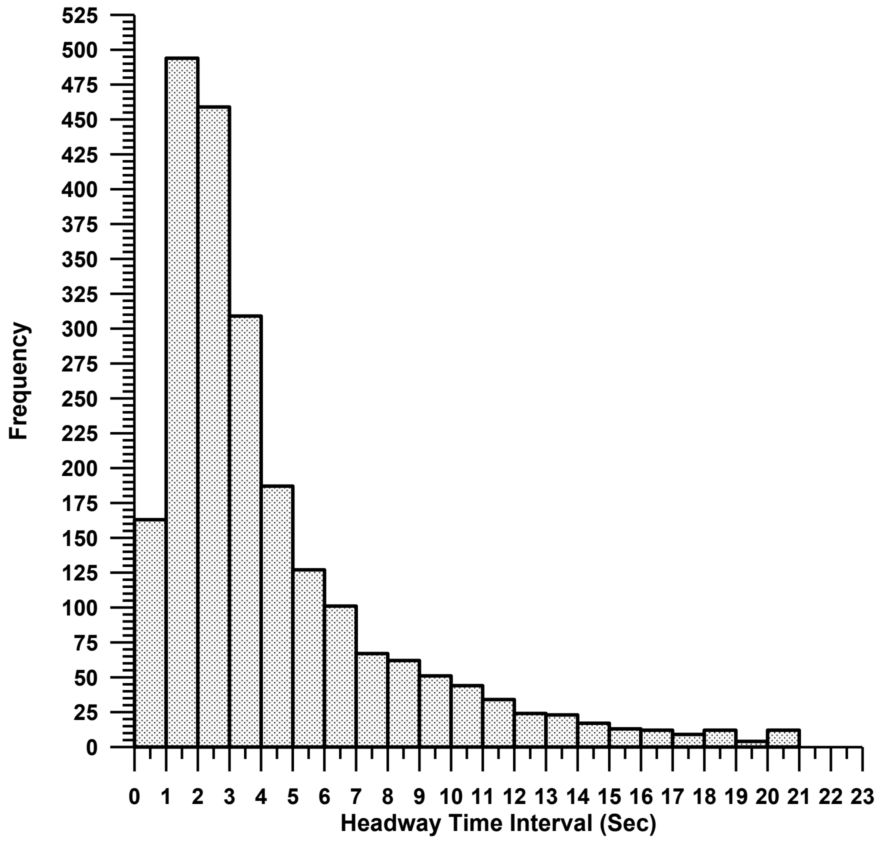

Figure 4-8: Observed arrival headway distribution for segment one, south bound, at morning and evening periods

The observed data were assumed to follow the Erlang distribution; the reason is the existence of very low probability of a small headway as in the previous case .To uses this distribution, it was necessary to determine the mean and variance of the average observed data. The mean and variance were found to be (4.2) and (12.7924), respectively. They were then used to determine the value of (a) and (b) parameters which the Erlang equation depends on their values. Appendix "B" shows the sample of calculations and their details. The rate of flow for this distribution was found to be (857) veh/hr. The theoretical frequencies were then calculated from Erlang equation. The chi–square test was used to examine the difference between the observed and theoretical frequencies for this distribution. Results are presented in table (4-8) Examination of tables of chi–squared showed that at the (5%) level of significance and (11) degrees of freedom, the value of tabulated chi–squared is (19.675), while the calculated value is (126.15), which is higher than the tabulated value which would be expected from an Erlang distribution.

Table 4-8: Observed and theoretical values of arrival headway distribution for segment one, south bound, at morning and evening periods

Class width (sec)	Observed frequency	Theoretical frequency from Erlang equation	Class width (sec)	Observed frequency		Theoretical frequency from Erlang equation	
0-1	82	98.85	10-11	22		17.78	
1-2	247	184.232	11-12	17		12.1	
2-3	229	190.76	12-13	12		8.17	
3-4	155	165.92	13-14	11	combined	5.482	combined
4-5	94	132.53	14-15	8		3.66	
5-6	64	100.63	15-16	6		2.43	
6-7	51	73.89	16-17	6		1.61	
7-8	34	52.964	17-18	6		1.1	
8-9	31	37.292	18-19	4		0.695	
9-10	26	25.894	19-20	2		0.455	
Calculated Chi-square= 126.15 Tabulated Chi-square= 19.675 Degree of freedom= 11							

Note: the class width (0-1) means (0- 0.9999), and so on for the remaining classes.

The alternative was to select another distribution, this time is the Negative exponential distribution, but this distribution was not checked. The reason is the existence of frequency of observed arrival vehicles in the class between (0–1) second. Results of analysis that presented in table (4-8) suggest that there is no good agreement between the observed and theoretical values of Erlang distribution depending on chi-square test. This is attributed to the same reasons that presented in the pervious section of north bound.

Therefore the closeness of fit between the theoretical Erlang distribution and observed data can be demonstrated by the graphical method to show probable agreement. Figure (4-9) shows this comparison. The comparison of the observed and calculated headway frequency is also shown in figure (4-10). The line x = y has been superimposed in the plot to facilitate comparison. The observed and calculated values are not matched exactly, however they are closely enough to consider that the observed values may follow the Erlang distribution.

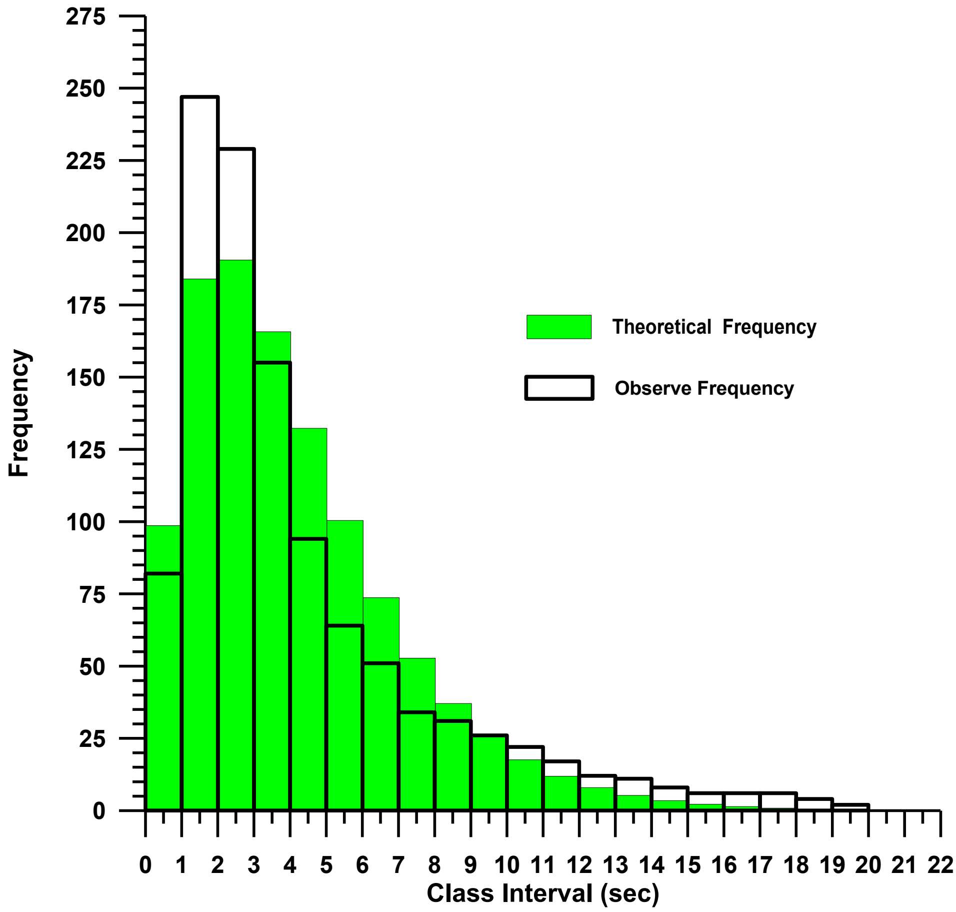

Figure 4-9: Comparison between of observed and theoretical headway distribution for segment one, south bound, at morning and evening periods

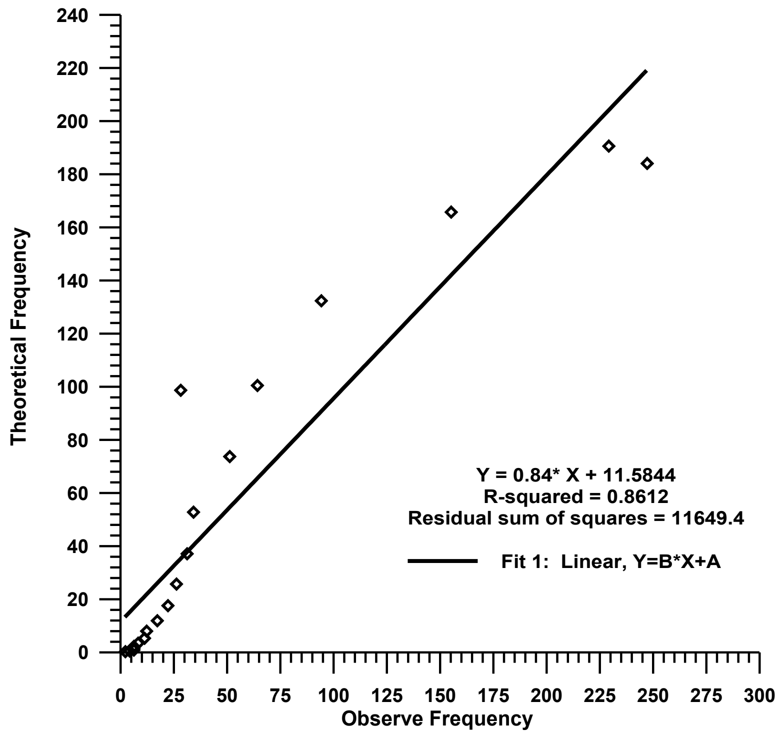

Figure 4-10: Comparison between the observed and theoretical headway distributions

4.1.2 *Segment Two*

4.1.2.1 *North Bound, Morning period*

Traffic flow data which were analyzed depending on the processed data that resulted from "EVENT" program were classified and grouped as shown in table (4-9), and presented graphically in the form of histogram as in figure (4-11).

Table 4-9: Observed values of arrival headway for segment two, north bound, at morning period

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	29	11 – 12	13
1 – 2	92	12 – 13	10
2 – 3	85	13 – 14	9
3 – 4	65	14 – 15	7
4 – 5	45	15 – 16	6
5 – 6	36	16 – 17	6
6 – 7	30	17 – 18	3
7 – 8	18	18 – 19	4
8 – 9	21	19 – 20	7
9 – 10	18	20 – 21	5
10 – 11	9		

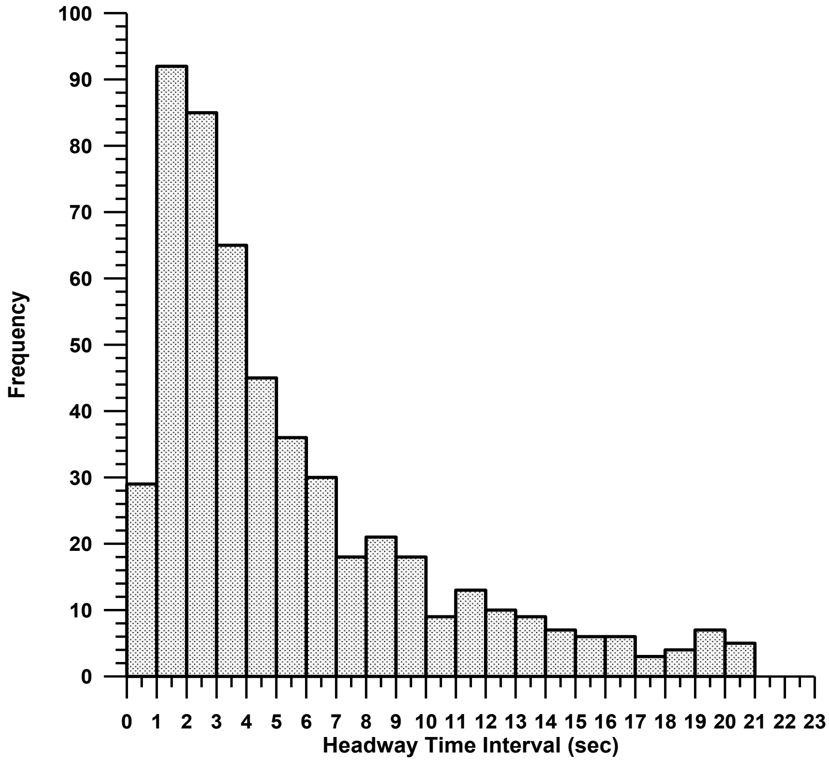

Figure 4-11: Observed arrival headway distribution for segment two, north bound, at morning period

4.1.2.2 *North Bound, Evening period*

The classified and grouped data for this case are shown in table (4-10), and presented graphically in the form of histogram as indicated in figure (4-12).

Table 4-10: Observed values of arrival headway for segment two, north bound, at evening period

Class width	Observed frequency	Class width	Observed frequency
0 – 1	67	10 - 11	22
1 – 2	177	11 – 12	7
2 – 3	126	12 – 13	15
3 – 4	86	13 – 14	10
4 – 5	76	14 – 15	7
5 – 6	47	15 – 16	6
6 – 7	36	16 – 17	8
7 – 8	24	17 – 18	6
8 – 9	23	18 – 19	6
9 – 10	17	19 – 20	1

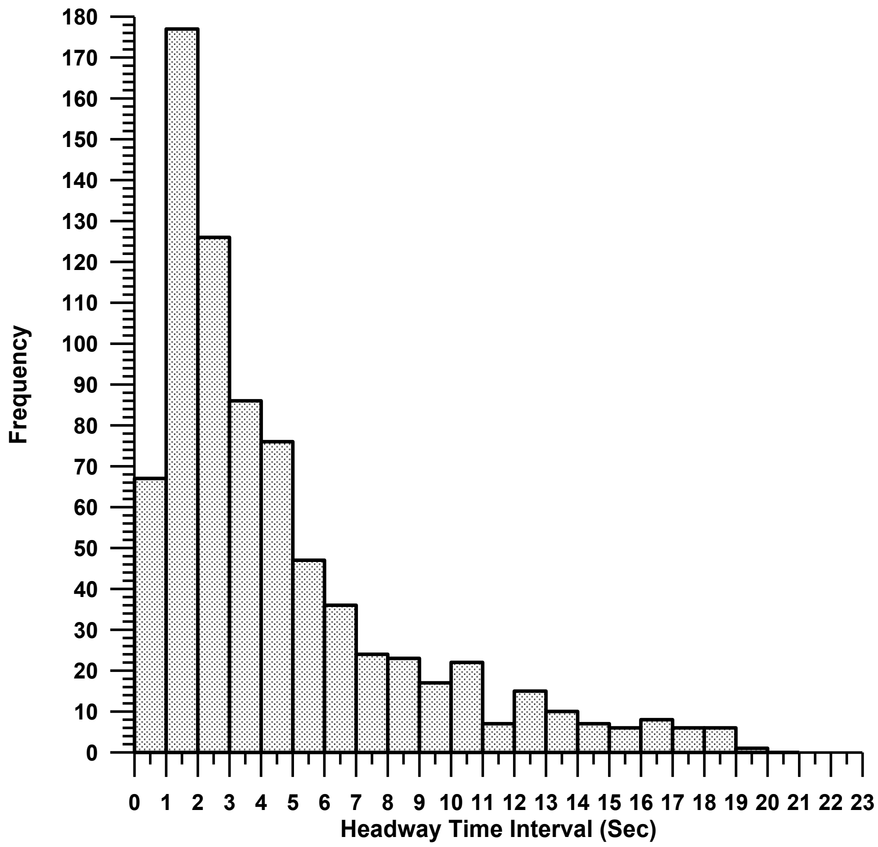

Figure 4-12: Observed arrival headway distribution for segment two, north bound, at evening period

4.1.2.3 *North Bound, Morning and Evening periods*

The observed data for each morning and evening periods were combined. The reason is to get reasonable representation for the observed data. The observed data were classified and grouped in a number of classes as presented in table (4-11). They are also presented in the form of histogram as shown by figure (4-13).

Table 4-11: Observed values of arrival headway for segment two, north bound, at morning and evening periods

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	96	11 – 12	20
1 – 2	269	12 – 13	25
2 – 3	211	13 – 14	19
3 – 4	151	14 – 15	14
4 – 5	121	15 – 16	12
5 – 6	83	16 – 17	14
6 – 7	66	17 – 18	9
7 – 8	42	18 – 19	10
8 – 9	44	19 – 20	7
9 – 10	35	20 – 21	6
10 – 11	31		

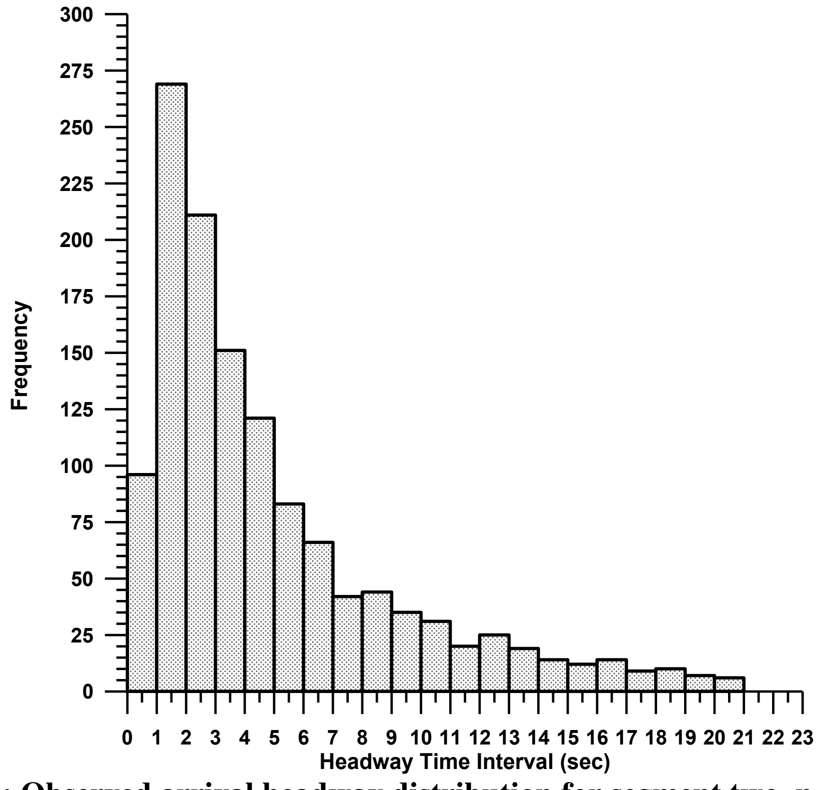

Figure 4-13: Observed arrival headway distribution for segment two, north bound, at morning and evening periods

The observed data were assumed to follow the Erlang distribution. The reason is the existence of very low probability of a small headway. To use this distribution, it was necessary to determine the mean and variance of the average observed data. The mean and variance were found to be (4.899) and (18.576), respectively. They were then used to determine the value of (a) and (b) parameters which the Erlang equation depends on their values. Appendix "B" shows the sample of calculations and their details. The rate of flow for this distribution was found to be (735) veh/hr. The theoretical frequencies were then calculated from Erlang equation. The chi–square test was used to examine the difference between the observed and theoretical frequencies for this distribution. Results are presented in table (4-12). Examination of tables of chi–squared showed that at the (5%) level of significance and (11) degrees of freedom, the value of tabulated chi–squared is (19.675), while the calculated value is (93.55), which is higher than the tabulated value which would be expected from an Erlang distribution.

Table 4-12: Observed and theoretical values of arrival headway distribution for segment two, north bound, at morning and evening periods, assumed to follow Erlang distribution.

Class width, (sec)	Observed frequency	Theoretical frequency from Erlang equation	Class width, (sec)	Observed frequency		Theoretical frequency from Erlang equation	
0-1	48	43.71	11-12	12		11.3	
1-2	135	87.2	12-13	10		8.2	
2-3	106	96.64	13-14	9		5.87	
3-4	76	89.97	14-15	7		4.2	
4-5	61	76.92	15-16	6		2.98	
5-6	42	62.52	16-17	7	Combined	2.11	Combined
6-7	33	49.13	17-18	5		1.49	
7-8	21	37.7	18-19	5		1.1	
8-9	22	28.41	19-20	4		0.73	
9-10	17	21.12	20-21	3		0.51	
10-11	15	15.52					

Calculated Chi-square = 93.55
Tabulated Chi-square = 19.675
Degree of freedom = 11

Note: the class width (0-1) means (0- 0.9999), and so on for the remaining classes.

The alternative was to select another distribution; this time is the Negative exponential distribution. The mean and rate of flow of this distribution were found to be (3.94) and (914) veh/hr, respectively. The chi–square test was used to examine the difference between the observed and theoretical frequencies for this distribution. Results are presented in table (4-13). Examination of tables of chi–squared showed that at the (5%) level of significance and (6) degrees of freedom, the value of tabulated chi–squared is (12.592), while the calculated value is (57.1), which is higher than the tabulated value.

Table 4-13: Observed and theoretical values of arrival head-way distribution of segment two, north bound, morning and evening periods

Class width (sec)	Observed frequency		Theoretical frequency from negative exponential eq.	
0	183		257.3	
2	182		154.82	
4	103		93.15	
6	54		56.1	
8	39		33.73	
10	27		20.29	
12	19		12.21	
14	3		7.35	
16	2	combined	4.42	combined
18	9		2.66	
20	5		4.02	

Calculated Chi-square = 57.1
Tabulated Chi-square=12.592
Degree of freedom= 6

Results of analysis suggest that there is no good agreement between the observed and theoretical values of this distribution depending on chi-square test. This is attributed to the same reasons that showed in the case of north bound of segment one.

As mentioned previously, the goodness of fits has been tested by three methods, therefore the closeness of fit between the theoretical Negative exponential distribution and observed data can be demonstrated by the graphical method to show probable agreement. Figure (4-14) shows this comparison. The comparison of the observed and calculated headway frequency is also shown in figure (4-15). The line x = y has been superimposed in the plot to facilitate comparison. The observed and calculated values are not matched exactly, however they are closely enough to consider that the observed values may follow the Negative exponential distribution.

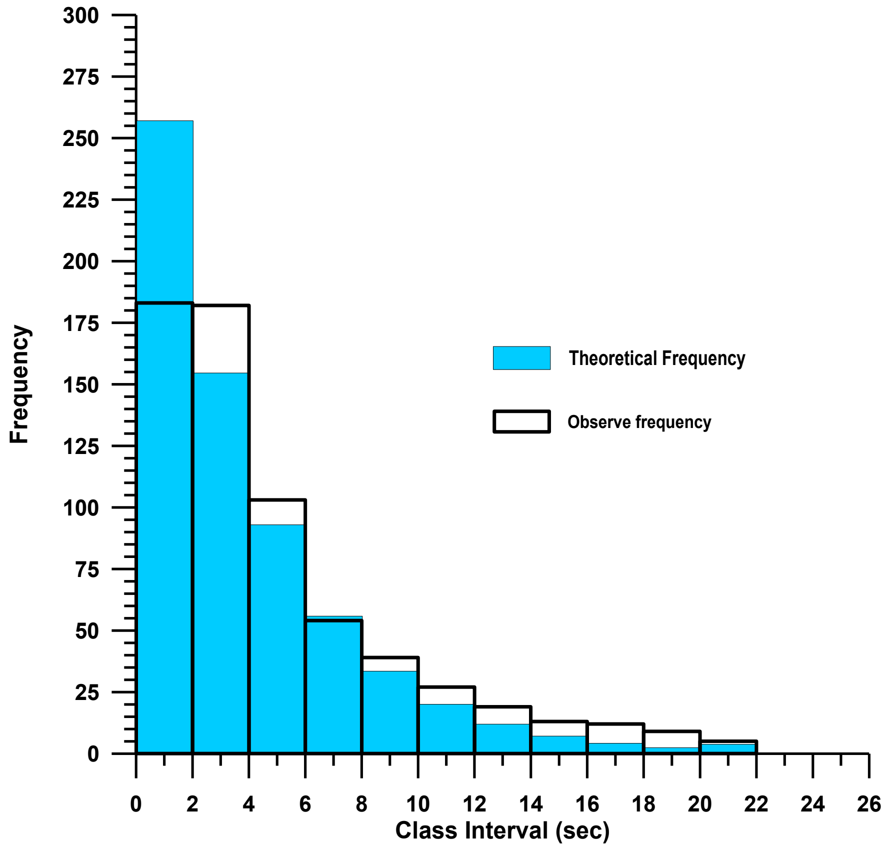

Figure 4-14: Comparison between observed and theoretical headway distribution for segment two, north bound, at morning and evening periods

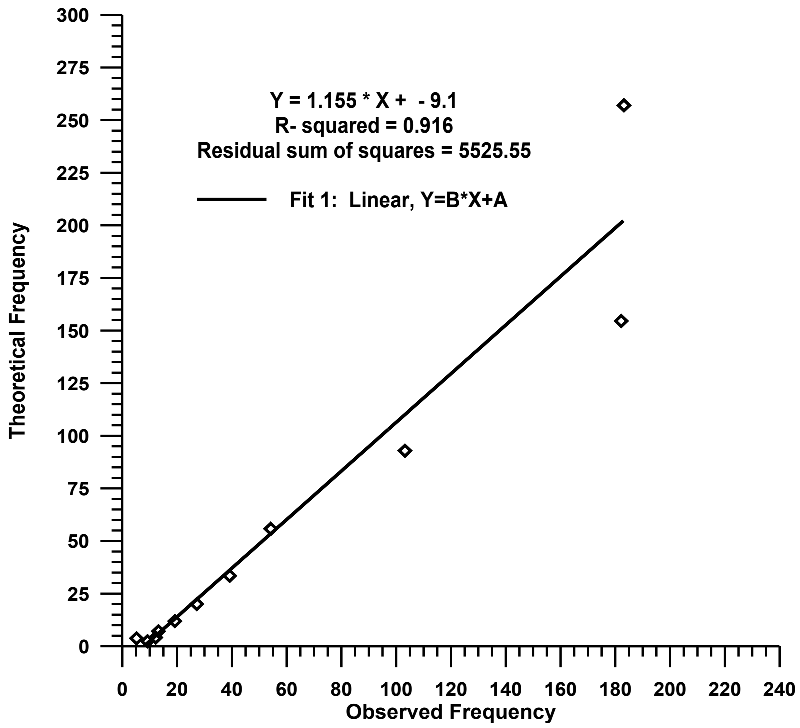

Figure 4-15: Comparison between the observed and theoretical headway distributions

4.1.2.4 *South Bound, Morning period*

The classified and grouped data are explained in table (4-14), and presented graphically in the form of histogram in figure (4-16).

Table 4-14: Observed values of arrival headway for segment two, south bound, at morning period

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	17	11 – 12	16
1 – 2	46	12 – 13	5
2 – 3	41	13 – 14	4
3 – 4	39	14 – 15	7
4 – 5	29	15 – 16	9
5 – 6	25	16 – 17	5
6 – 7	11	17 – 18	7
7 – 8	14	18 – 19	3
8 – 9	17	19 – 20	4
9 – 10	18	20 – 21	5
10 – 11	10		

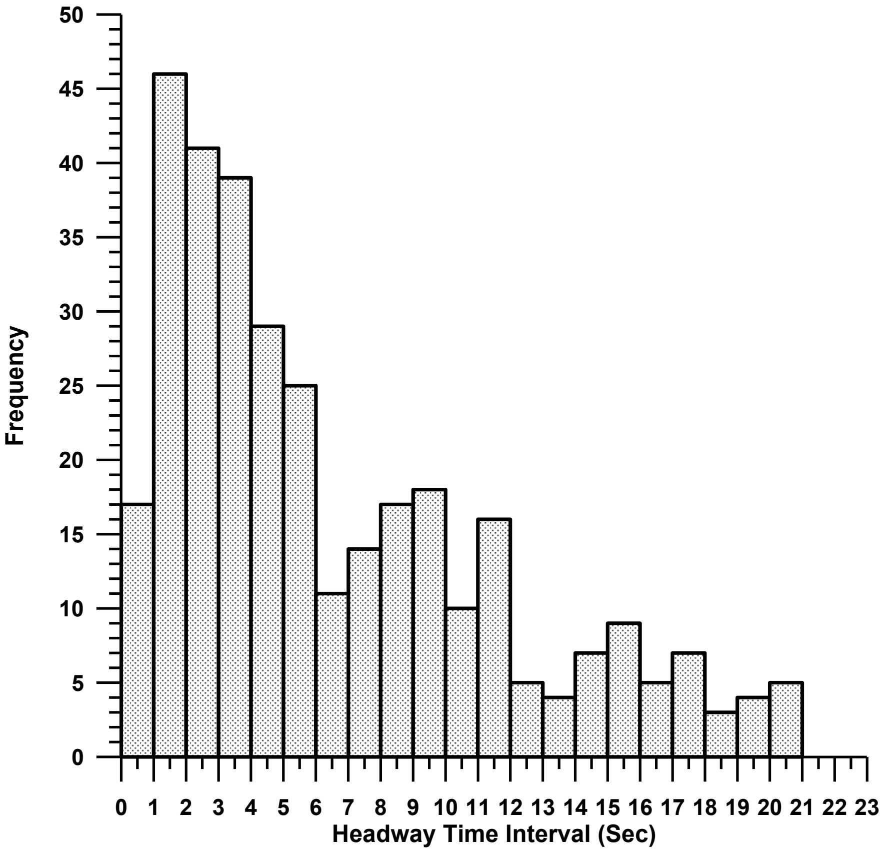

Figure 4-16: Observed arrival headway distribution for segment two, south bound, at morning period

4.1.2.5 *South Bound, Evening period*

As in the previous cases, the classified and grouped data are explained in table (4-15), and presented graphically in the form of histogram in figure (4-17).

Table 4-15: Observed values of arrival headway for segment two, south bound, at evening period

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	15	11 – 12	12
1 – 2	38	12 – 13	13
2 – 3	38	13 – 14	7
3 – 4	27	14 – 15	5
4 – 5	28	15 – 16	9
5 – 6	30	16 – 17	4
6 – 7	24	17 – 18	4
7 – 8	21	18 – 19	4
8 – 9	15	19 – 20	6
9 – 10	22	20 – 21	3
10 – 11	23		

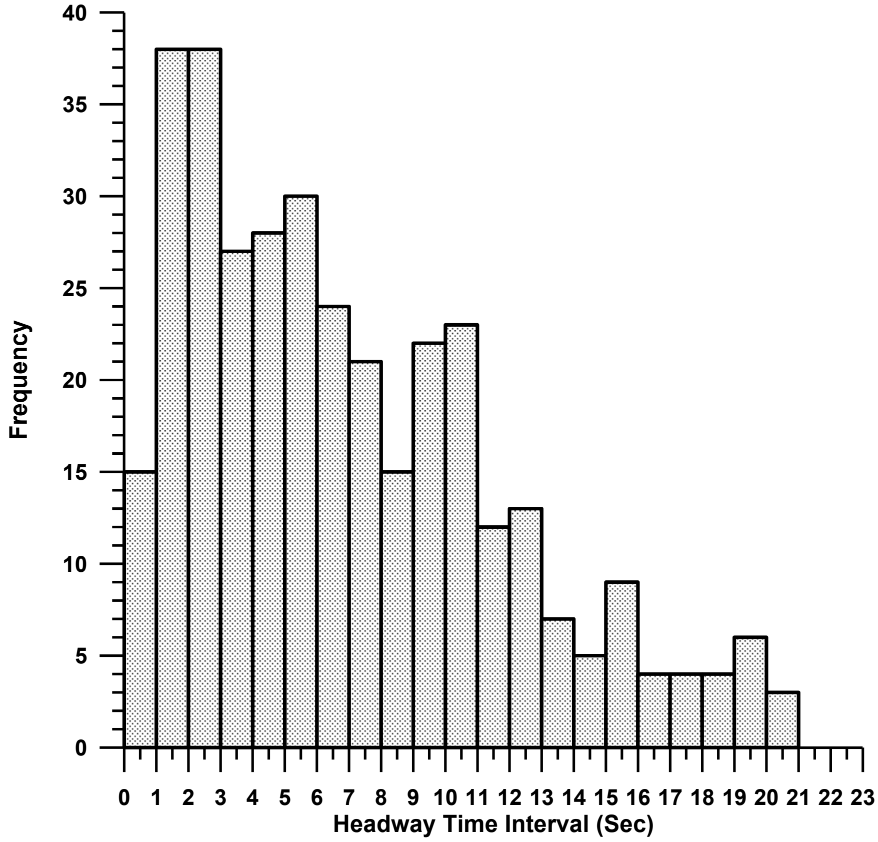

Figure 4-17: Observed arrival headway distribution for segment two, south bound, at evening period

4.1.2.6 *South Bound, Morning and Evening periods*

Because of the lack of adequate values of data, the combining operation of the observed data for both morning and evening periods were made in order to get acceptable and reasonable representation for the observed data. The observed data were classified and grouped in a number of classes as shown in table (4-16), and also presented graphically in the form of histogram as indicated in figure (4-18).

Table 4-16: Observed values of arrival headway for segment two, south bound, at morning evening periods

Class width(sec)	Observed frequency	Class width(sec)	Observed frequency
0 – 1	32	11 – 12	28
1 – 2	84	12 – 13	18
2 – 3	79	13 – 14	11
3 – 4	66	14 – 15	12
4 – 5	57	15 – 16	18
5 – 6	55	16 – 17	9
6 – 7	35	17 – 18	11
7 – 8	25	18 – 19	7
8 – 9	32	19 – 20	10
9 – 10	40	20 – 21	8
10 – 11	33		

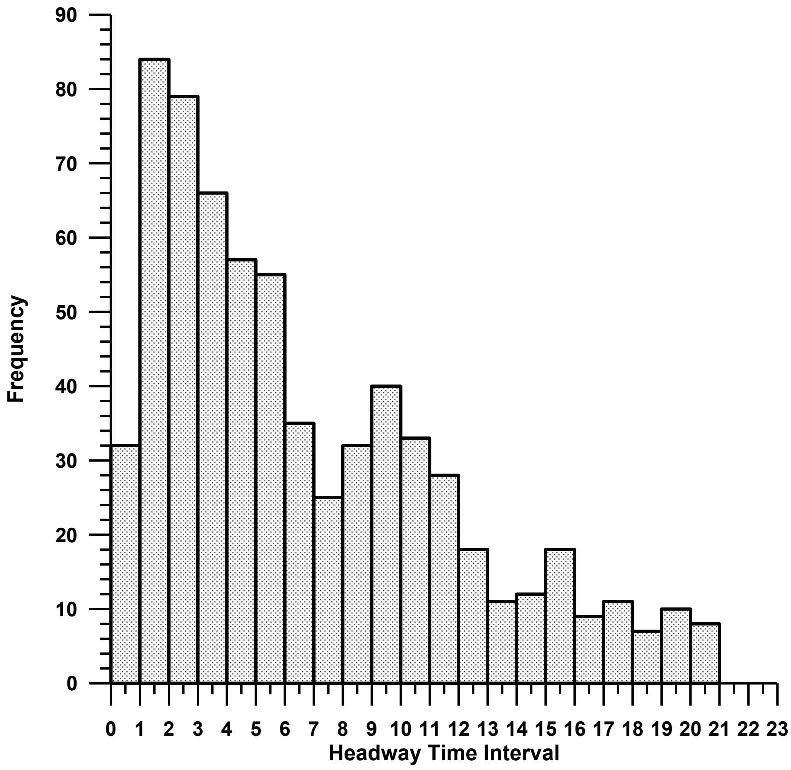

Figure 4-18: Observed arrival headway distribution for segment two, south bound, at morning and evening periods

The observed data were assumed to follow the Erlang distribution. The reason is the existence of very low probability of a small headway. To use this distribution, it was necessary to determine the mean and variance of the average observed data. The mean and variance were found to be (7.74) and (38.374), respectively. They were then used to determine the value of (a) and (b) parameters which the Erlang equation depends on their values. Appendix "B" shows the sample of calculations and their details. The rate of flow for this distribution was found to be (465) veh/hr. The theoretical frequencies were then calculated from Erlang equation. The chi–square test was used to examine the difference between the observed and theoretical frequencies for this distribution. Results are presented in table (4-17). Examination of tables of chi–squared showed that at the (5%) level of significance and (11) degrees of freedom, the value of tabulated chi–squared is (19.675), while the calculated value is (31.033), which is higher than the tabulated value which would be expected from an Erlang distribution.

Table 4-17: Observed and theoretical values of arrival headway distribution for segment two, south bound, at morning and evening periods

Class width (sec)	Observed frequency	Theoretical frequency From Erlang equation	Class width (sec)	Observed frequency		Theoretical frequency From Erlang equation	
0-1	16	10.68	13-14	6		10.02	
1-2	42	24.743	14-15	6		8.32	
2-3	39	31.85	15-16	9		6.864	
3-4	33	34.43	16-17	5		5.64	
4-5	29	34.2	17-18	5		4.62	
5-6	28	32.27	18-19	4		3.774	
6-7	18	29.46	19-20	5	combined	3.1	combined
7-8	18	26.25	20-21	4		2.5	
8-9	16	22.97	21-22	5		2.02	
9-10	20	19.83	22-23	5		1.63	
10-11	17	16.93	23-24	6		1.32	
11-12	14	14.32	24-25	2		1.1	
12-13	9	12.02	25-26	3		0.85	
Calculated Chi-square = 31.033 Tabulated Chi-square = 19.675 Degree of freedom = 11							

Note: the class width (0-1) means (0- 0.9999), and so on for the remaining classes.

The alternative was to select another distribution. This time is the Negative exponential distribution, but this distribution was not checked. The reason is the existence of frequency of observed arrival vehicles in the class between (0–1) second.

Results of analysis that presented in table (4-17) suggest that there is no good agreement between the observed and theoretical values of Erlang distribution depending on chi-square test. This is attributed to the same reasons that mentioned in the previous cases.

Therefore, the closeness of fit between the theoretical Erlang distribution and observed data can be demonstrated by the graphical method to show probable agreement. Figure (4-19) shows this comparison .The comparison of the observed and calculated headway frequency is also shown in figure (4-20). The line x = y has been superimposed in the plot to facilitate comparison. The observed and calculated values are not matched exactly, however, they are closely enough to consider that the observed values may follow the Erlang distribution.

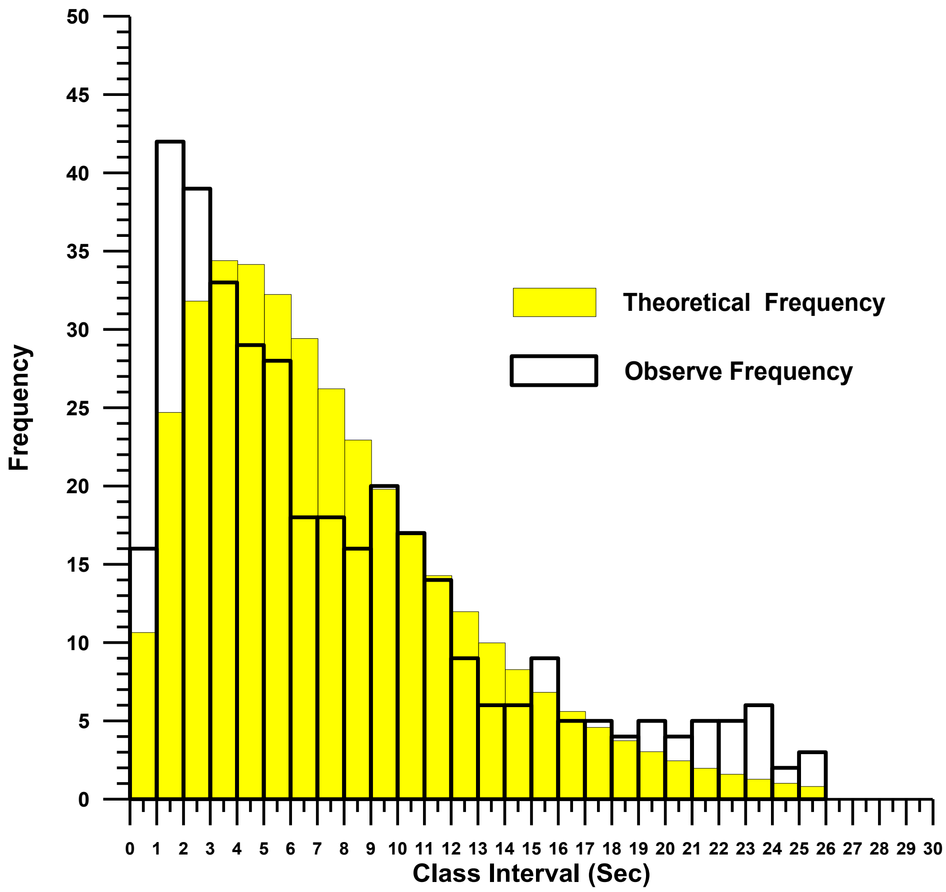

Figure 4-19: Comparison between observed and theoretical headway distribution for segment two, south bound, at morning and evening periods

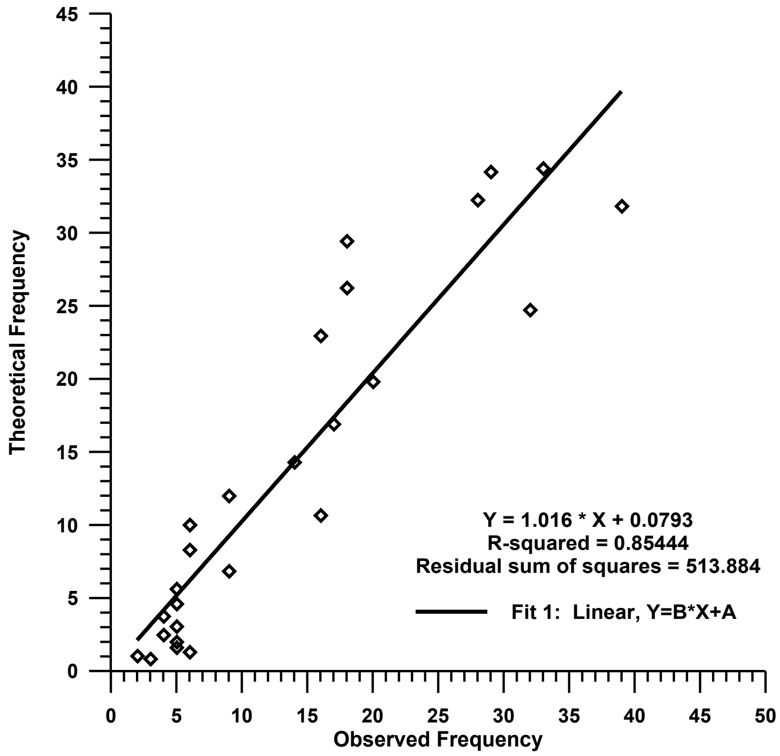

Figure 4-20: Comparison between the observed and theoretical headway distributions

4.2 *Traffic Flow*

The total traffic flow for the two segments in morning and evening periods and their combinations are presented in table (4-18) as in the following:

Table 4-18: Total flow for the two segments of al-tharthar highway section

Segment number	**Total flow (veh/hr)**					
	North bound			South bound		
	Morning	Evening	Combining	Morning	Evening	Combining
Segment one	741	939	840	839	700	770
Segment two	615	736	676	427	376	402

4.3 *Traffic Composition*

The traffic composition and the percentage of each type of vehicles for segment one and segment two in both directions and in morning and evening periods and their combining periods are presented in tables (4-19) and (4-20) as in following :

Table 4-19: Traffic composition for segment one

Traffic vehicle type	**% for each type**					
	North bound			South bound		
	Morning	Evening	Combining	Morning	Evening	Combining
Passenger cars	71.39	73.69	72.74	74.5	74.29	74.42
Tracks	10.26	12.67	11.67	10.85	10.57	10.78
Buses	5.13	1.28	2.97	2.86	1	2.08
Bicycles	3.64	1.71	2.5	4.17	5.43	4.67
Others	9.58	10.65	10.12	7.62	8.71	8.05

Table 4-20: Traffic composition for segment two

Traffic vehicle type	**% for each type**					
	North bound			South bound		
	Morning	Evening	Combining	Morning	Evening	Combining
Passenger cars	76.75	74.86	75.74	70.96	70.75	70.9
Tracks	7.32	8.30	7.84	8.20	9.04	8.71
Buses	3.74	1.90	2.80	3.51	1.86	2.73
Bicycles	5.20	4.21	4.59	8.43	7.71	7.96
Others	6.99	10.73	9.03	8.90	10.64	9.7

In table (4-20), the percentage of bicycles in south bound of segment two is reasonably high. The reason is the periods of video recording which were done in peak hours and during the time that the pupils using bicycles go to or leave their schools, and also the people use the bicycles to avoid passing through the check points.

4.4 *Analysis of Vehicular Speed Distribution*

These speed data were abstracted from video films for all selected sites of this study as explained in details in the previous chapter. After the abstraction, the data were analyzed statistically to find mean, standard deviation, maximum and minimum values, and then to find the distribution of speed which can be used to represent the observed data. The mean and standard deviation (parameters of Normal distribution) were calculated using the method of transformation of cumulative Normal distribution to linear relationship using the probit method of analysis. The probit value was regressed against the class width. Linear regression is used to find the best fit line which represents the data. The adopted linear model is as given below:

$$Y = a + b X \quad \text{................................ (4-1)}$$

Where:

Y is the probit value

X is the mid class interval

a and b are coefficients

The analysis was done for the two segments and both directions, with (200) random values of measured data of observed speed for each direction, which were collected from both morning and evening periods, as in the following:

4.4.1 *Segment One*

4.4.1.1 *North Bound, Morning and Evening periods*

The coefficients of the regression model are presented in equation below:

$$Y = 0.0718 + 0.098*X$$

This equation is used to calculate the mean and standard deviation of the Normal distribution. Substituting Y equal to 5 results in X value, which represents the mean. The reciprocal of the coefficient of X results in the standard deviation of the Normal distribution. Their values are presented in table (4-21). The probit method of analysis is represented graphically in figure (4-22), while the speed data were grouped in a number of classes and presented graphically in the form of histogram as shown in figure (4-21). The collected sample size of speed data was (200) vehicles, (77) of them were taken from the near lane and the remaining from the far lane of the road.

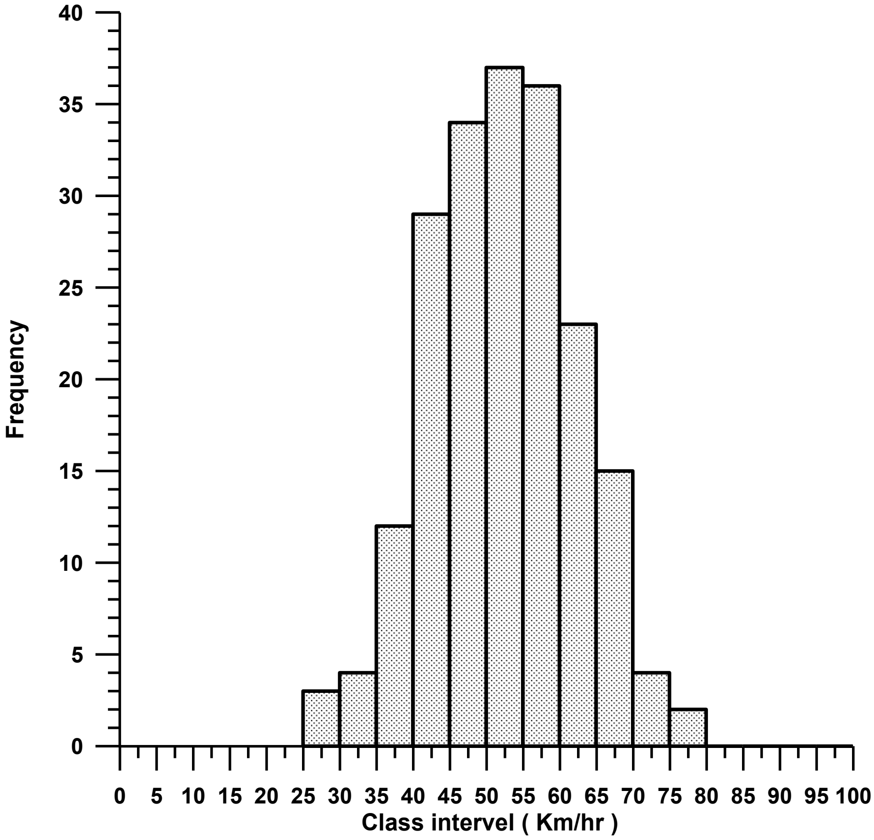

Figure 4-21: Observed distribution of vehicular speed data for segment one, north bound, at morning and evening periods

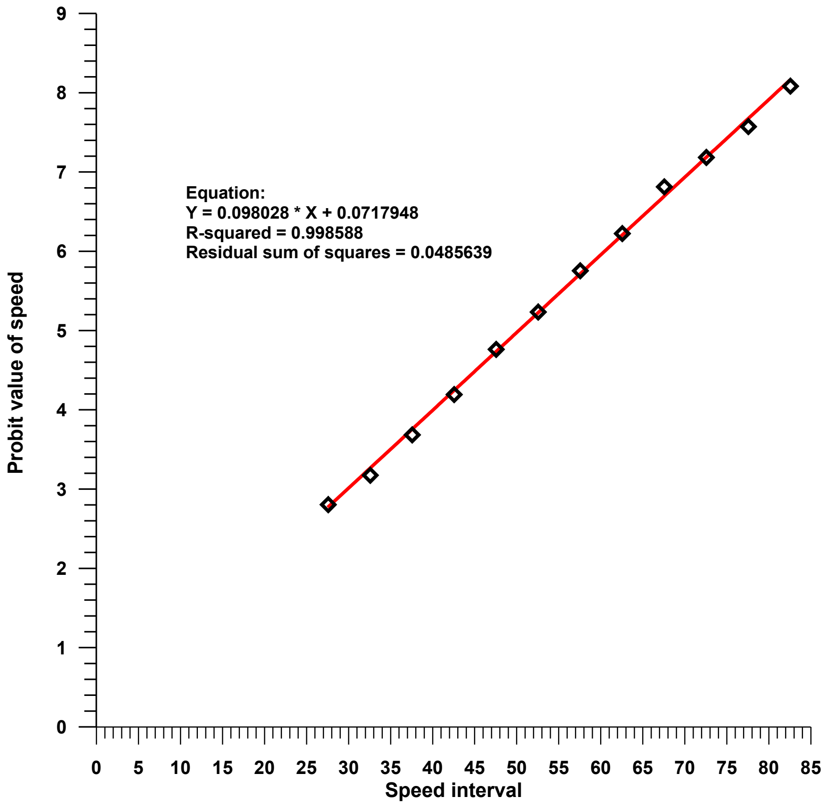

Figure 4-22: Transformation of cumulative Normal distribution of observed speed data to linear relationship using probit method for segment one, north bound, at morning and evening periods

Table 4-21: Statistical analysis of observed speed data for segment one, north bound at morning and evening periods

Function	Value
Mean by probit method	50.288 Km/hr
Space mean speed	50.72 Km/hr
Standard deviation	10 .2 Km/hr
Max. value	78.86 km/hr
Min. value	27.126 Km/hr

The observed data were assumed to follow Normal distribution. The Chi-square test was used to test the validity that the observed data follow this distribution. Theoretical frequencies were calculated and presented in table (4-22). The table also contains results of the Chi-square test performed. At the 5% level of significance and at (5) degrees of freedom the tabulated value was found to be (11.1), which is higher than the calculated value (9.64), therefore, there is good agreement between observed and theoretical data, so, the observed data may be represented by a Normal distribution with mean and standard deviation of (50.288) and (10.2) respectively.

Table 4-22: Statistical test results of observed speed distribution for segment one, north bound, at morning and evening periods

Class width (Km/hr)	Observed Frequency		Theoretical Frequency	
25 - 30	3		3.34	
30 - 35	4	combined	8.96	combined
35 - 40	12		18.12	
40 - 45	29		28.7	
45 - 50	34		37.32	
50 - 55	37		33.28	
55 - 60	36		30.36	
60 - 65	23		19.14	
65 - 70	15		9.62	
70 - 75	4	combined	3.8	combined
75 - 80	2		1.2	
Calculated chi-square value = 9.64 Tabulated chi-square value (5%) = 11.1 Degree of freedom= 5				

The minimum sample size for Normal distribution, (n) required for any statistical analysis can be determined from the following formula (Nicholas J. Garber, 2003, et al):

$$n = \left(\frac{Z * S}{E} \right)^2 \text{ ....} \quad (4\text{-}2)$$

Where:

n = Minimum sample size.
Z = Confidence level.
S = Estimate of the standard deviation.
E = Range of error, which is equal to 2.414 (km/hr).
at 5% level of significance, the confidence level (z) is equal to (1.96).

Therefore, the minimum sample size $n = \left(\frac{1.96 * 10.2}{2.414} \right)^2 = 69$

The minimum number of speed collected that satisfies the requirement of this case is less than the actual sample size that collected in the field which is equal to (200). In addition, (Nicholas J. Garber ,2003, et al.) mentioned that the duration of the study should be such that the minimum number of vehicle speeds required for statistical analysis is recorded, typically, the duration is at least (1) hour and the sample size is at lest 30 vehicles.

4.4.1.2 ***South Bound, Morning and Evening periods***

The coefficients of the regression model are presented in equation below:

Y = 0.0985636 * X + 0.0890453

This equation is used to calculate the mean and standard deviation of the Normal distribution. Substituting Y equal to 5 results in X value, which represent the mean. The reciprocal of the coefficient of X result in the standard deviation of the Normal distribution. Their values are presented in table (4-23). The probit method of analysis is represented graphically in figure (4-24), while the speed data were grouped in a number of classes and presented graphically in the form of histogram as shown in figure (4-23). The collected sample size of speed data was (200) vehicles, (56) of them were taken from outer lane (near the shoulder) and the remaining from inner lane (beside the concrete median).

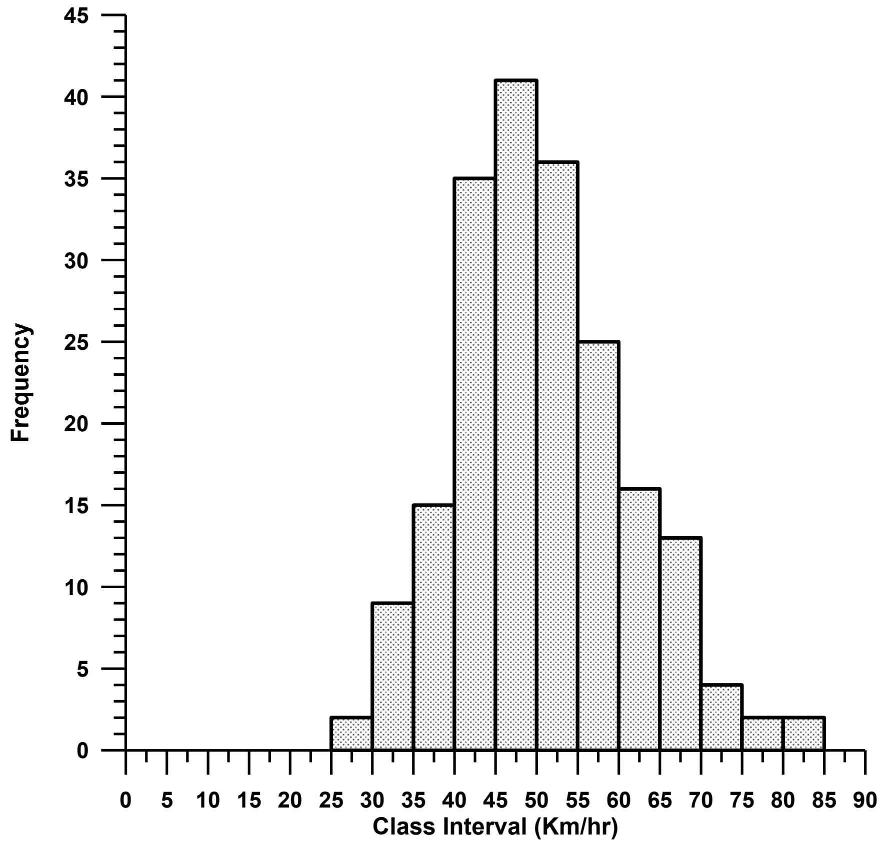

Figure 4-23: Observed distribution of vehicular speed data for segment one, south bound, at morning and evening periods

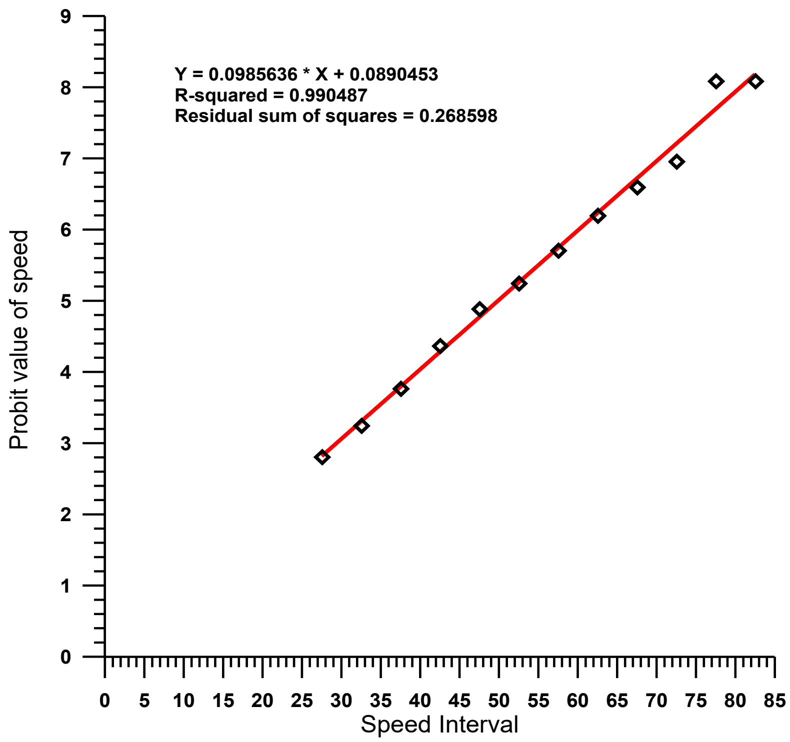

Figure 4-24: Transformation of cumulative Normal distribution of observed speed data to linear relationship using probit method for segment one, south bound, at morning and evening periods

Table 4-23: Statistical analysis of observed speed data for segment one, south bound, at morning and evening periods

Function	Value
Mean by probit method	50.11 Km/hr
Space mean speed	48.84 Km/hr
Standard deviation	10.2 Km/hr
Max. value	83.51 km/hr
Min. value	26.45 Km/hr

The observed data were assumed to follow Normal distribution. Chi-square test was used to test the validity that the observed data follow this distribution. Theoretical frequencies were calculated and presented in table (4-24). The table also contains results of the Chi-square test performed. At the 5% level of significance and at (5) degrees of freedom the tabulated value was found to be (11.1),which is higher than the calculated value (6.47), therefore, there is a good agreement between observed and theoretical data, so, the observed data may be represented by a Normal distribution with mean and standard deviation of (50.11) and (10.2), respectively.

Table 4-24: Statistical test results of observed speed distribution for segment one, south bound, at morning and evening periods

Class width (Km/hr)	Observed Frequency		Theoretical Frequency	
25-30	2		3.5	
30 - 35	9	Combined	9	Combined
35 - 40	15		18.34	
40 - 45	35		29.48	
45 - 50	41		37.5	
50 - 55	36		36.08	
55 - 60	25		29.92	
60 - 65	16		18.78	
65 - 70	13		9.3	
70 - 75	4	Combined	3.66	Combined
75 - 80	2		1.12	
80 - 85	2		0.28	
Calculated chi-square value = 6.47 Tabulated chi-square value (5%) = 11.1 Degree of freedom = 5				

The minimum sample size for Normal distribution, (n) required for statistical analysis for this bound of segment one can be determined from equation (4-2) as the previous case:

The minimum sample size $$n = \left(\frac{1.96 * 10.2}{2.414} \right)^2 = 69$$

Therefore, the minimum number of speed collected that satisfies the requirement of this case is less than the actual sample size that collected in the field which is equal to (200), in addition, as mentioned in the previous case by (Nicholas J. Garber, 2003, et al), the minimum sample size is at least 30 vehicles, which is less than actual collected sample size in the field.

4.4.2 *Segment Two*

4.4.2.1 *North Bound, Morning and Evening periods*

The coefficients of the regression model are presented in equation below:

$$Y = 0.0925*X + 0.9362$$

This equation is used to calculate the mean and standard deviation of the Normal distribution by using the same method that explained in the previous cases. Their values are presented in table (4-25). The probit method of analysis is represented graphically in figure (4-26), while the speed data were grouped in a number of classes and presented graphically in the form of histogram as shown in figure (4-25). The collected sample size of speed data was (200) vehicles, (73) of them were taken from the near lane and the remaining from the far lane of the road.

Table 4-25: Statistical analysis of observed speed data for segment two, north bound at morning and evening periods

Function	Value
Mean by probit method	43.95 Km/hr
Space mean speed	42.55 Km/hr
Standard deviation	10.814 Km/hr
Max. value	73.434 km/hr
Min. value	20.28 Km/hr

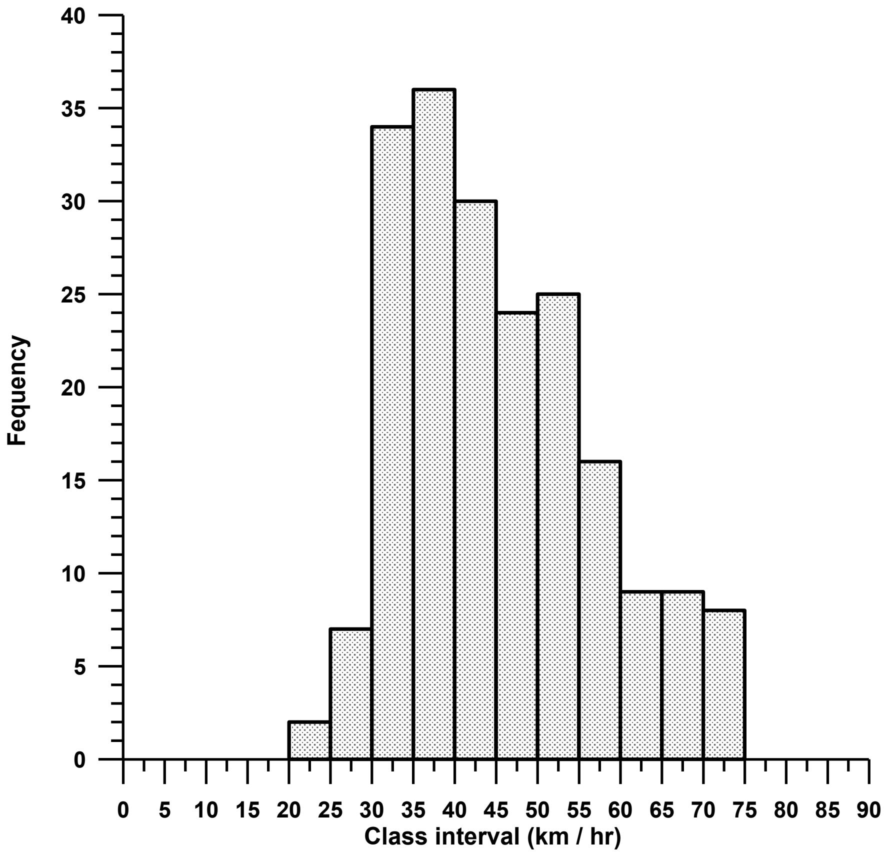

Figure 4-25: Observed distribution of vehicular speed data for segment two, north bound, at morning and evening periods

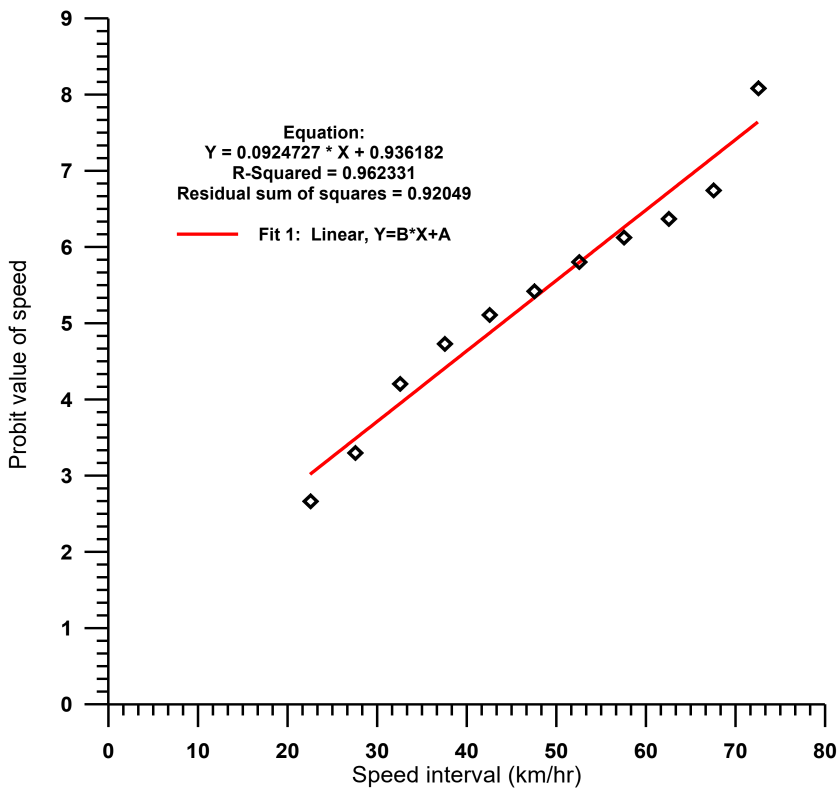

Figure 4-26: Transformation of cumulative Normal distribution of observed speed data to linear relationship using probit method for segment two, north bound, at morning and evening periods

The minimum sample size for Normal distribution, (n) required for statistical analysis for the north bound of segment two can be determined from equation (4-2) as in the previous case:

$$\text{The sample size} \quad n = \left(\frac{1.96 * 10.814}{2.414} \right)^2 = 77$$

Therefore, the minimum number of speed collected is less than the actual sample size that collected in the field, in addition, as mentioned previously by (Nicholas J. Garber, 2003, et al.), the minimum sample size is at least 30 vehicles, which is less than actual collected sample size.

The observed data were assumed to follow Normal distribution. Chi-square test was used to test the validity that the observed data follow this distribution. Theoretical frequencies were calculated and presented in table (4-26). The table also contains results of the Chi-square test performed. At the 5% level of significance and at (5) degrees of freedom, the tabulated value was found to be (11.1), which is less than the calculated value (32.17), therefore, there is no good agreement between the observed and theoretical values to represent the observed data by Normal distribution.

Table 4-26: Statistical test results of observed speed distribution for segment two, north bound, at morning and evening periods

Class width (Km/hr)	Observed Frequency		Theoretical Frequency	
20 - 25	2	combined	5.38	combined
25 - 30	7		11.68	
30 - 35	34		21.06	
35 - 40	36		30.75	
40 - 45	30		20.764	
45 - 50	24		34.734	
50 - 55	25		26.856	
55 - 60	16		16.908	
60 - 65	9		2.596	
65 - 70	9	combined	9.58	combined
70 - 75	8		1.2	
Calculated chi-square value = 32.17 Tabulated chi-square value (5%) = 11.1 at degree of freedom= 5				

The alternative was to select another distribution. The Log normal distribution was chosen to represent the observed data. The mean and standard deviation of the observed data were found to be 45.45 and 11.875, respectively. This distribution was tested by graphical method.

The method assumes that the curve of a cumulative Lognormal distribution is a straight line in case of using graph paper having a log scale on one axis and a normal probability scale on the other(16, Al-kubaisy , 2004). Table (4-27) shows in details the results of performed analysis .Figure (4-27) indicates the graphical representation of this method.

Then regression line is used to find the best fit of the data. The coefficient of determination (R^2) is found to be (0.9819).The examination of (R^2) points out that the observed data may be represented by Log normal distribution because of high value of coefficient of determination.

Table 4-27: Statistical test results for the observed speed by Log normal distribution for segment two, north bound , at morning and evening periods

Class width(km/hr)	Log of column 1	Observed frequency	%Cumulative frequency
20 - 25	1.398	2	1
25 - 30	1.477	7	4.5
30 - 35	1.544	34	21.5
35 - 40	1.602	36	39.5
40 - 45	1.653	30	54.5
45 - 50	1.699	24	66.5
50 - 55	1.7404	25	79
55 - 60	1.778	16	87
60 - 65	1.813	9	91.5
65 - 70	1.845	9	96
70 - 75	1.875	8	100
calculated mean=45.45 km/hr calculated standard deviation=11.875 km/hr coefficient of determination (R^2)=0.9819			

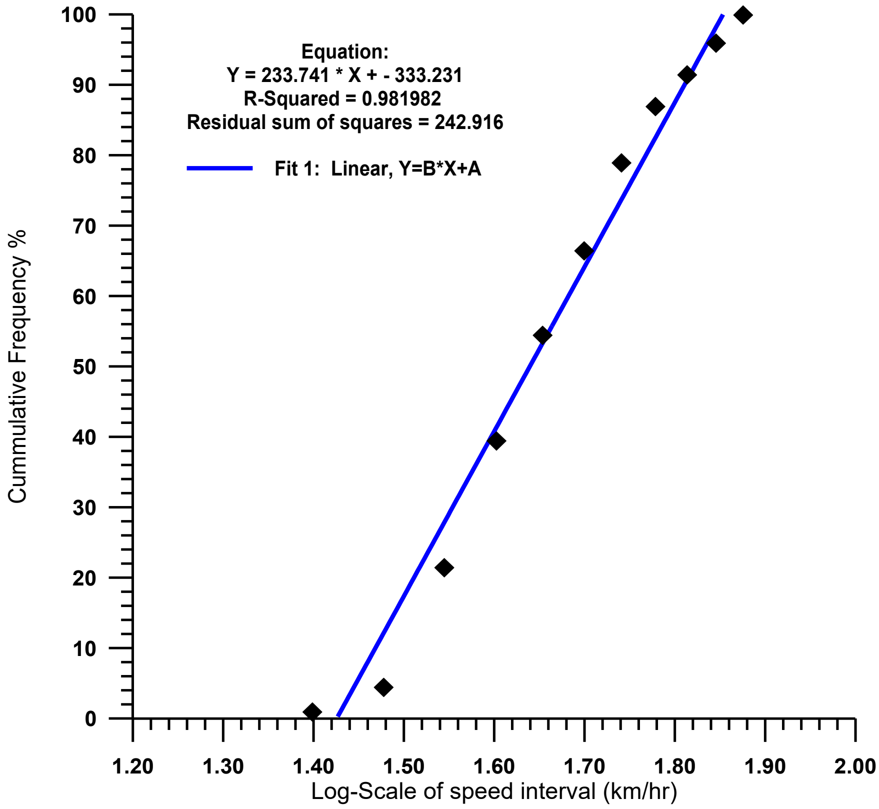

Figure 4-27: Transformation of percentage cumulative speed Lognormal distribution to linear relationship by changing the normal X scale to log scale for segment two, north bound at morning and evening periods

4.4.2.2 *South Bound, Morning and Evening periods*

The coefficients of the regression model are presented in equation below:

$$Y = 0.093 + 0.09847 * X$$

This equation is used to calculate the mean and standard deviation of the Normal distribution. Their values are presented in table (4-28). The probit method is represented graphically in figure (4-29), while the speed data were grouped in a number of classes and presented graphically in the form of histogram as shown in figure (4-28). The collected sample size of speed data was (200) vehicles, (85) of them were taken from the near lane and the remaining from the far lane of the road. The minimum sample size, (n) required for this case is found to be (69), which is less than the actual sample size that collected in the field.

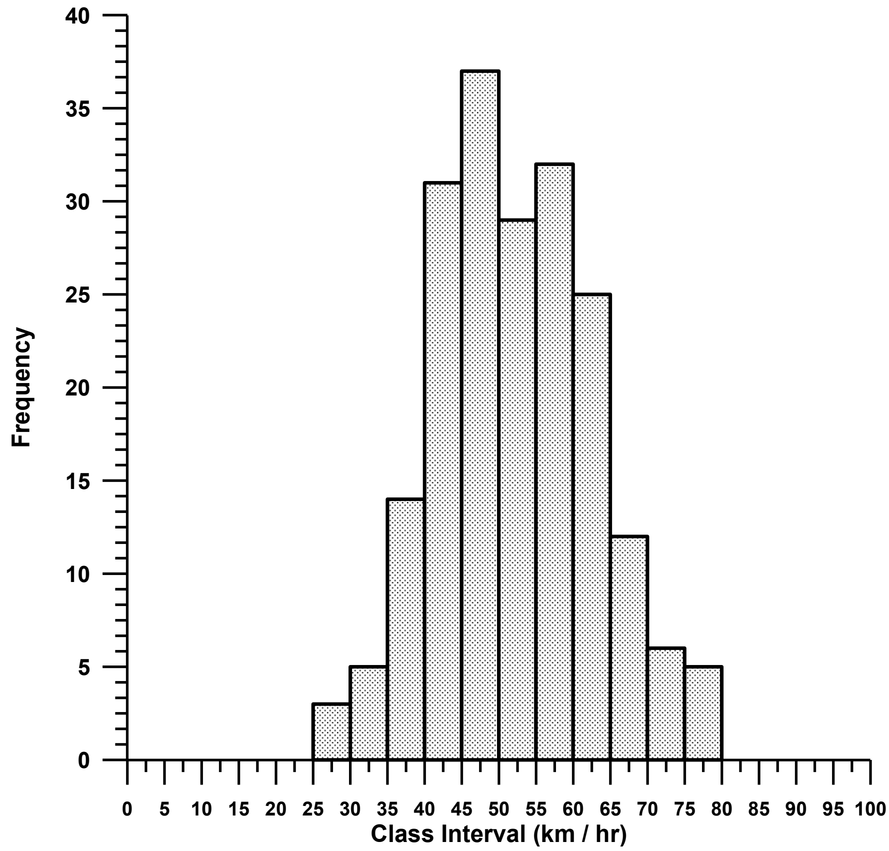

Figure 4-28: Observed distribution of vehicular speed data for segment two, south bound, at morning and evening periods

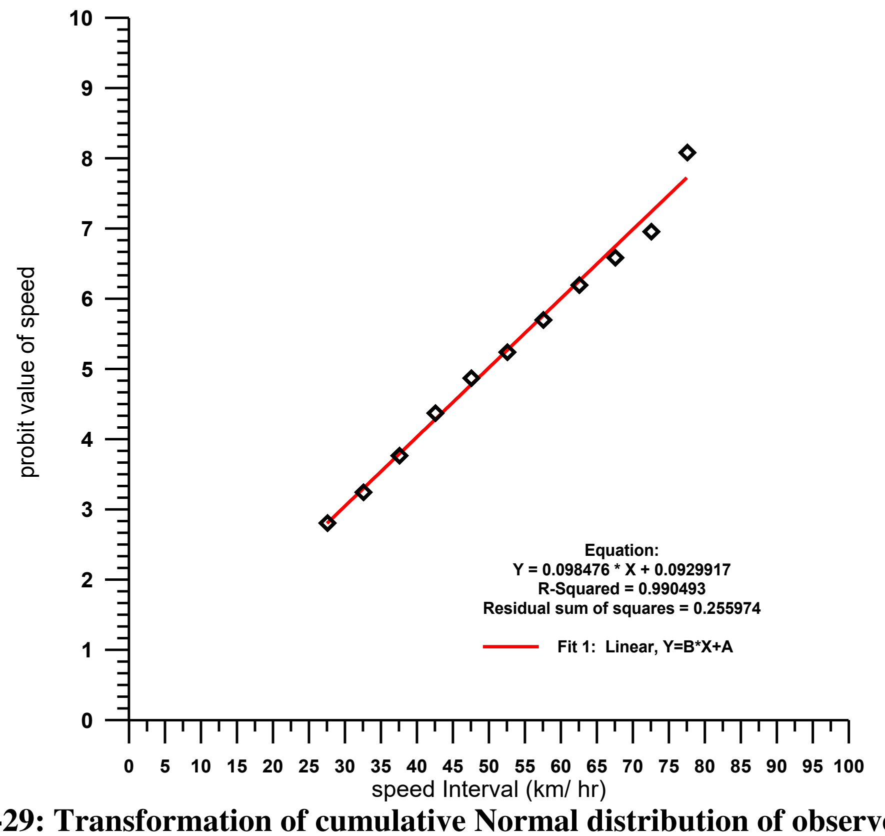

Figure 4-29: Transformation of cumulative Normal distribution of observed speed data to linear relationship using probit method for segment two, south bound, at morning and evening periods

Table 4-28: Statistical analysis of observed speed data for segment two, south bound at morning and evening periods

Function	Value
Mean by probit method	49.83 Km/hr
Space mean speed	50.13 Km/hr
Standard deviation	10.2 Km/hr
Max. value	78.873 km/hr
Min. value	25.968 Km/hr

The observed data were assumed to follow Normal distribution. Chi-square test was used to test the validity that the observed data follow this distribution. Theoretical frequencies were calculated and presented in table (4-29). The table also contains results of the Chi-square test performed. At the 5% level of significance and at (5) degrees of freedom the tabulated value was found to be (11.1),which is less than the calculated value (15.18), therefore, there is no good agreement between the observed and theoretical values to represent the observed data by Normal distribution .

Table 4-29: Statistical test results of observed speed distribution for segment two, south bound, at morning and evening periods

Class width (Km/hr)	Observed Frequency		Theoretical Frequency	
25-30	3	Combined	3.69	Combined
30-35	5		9.35	
35-40	14		18.82	
40-45	31		29.9	
45-50	37		34.67	
50-55	29		37.21	
55-60	32		28.97	
60-65	25		18.17	
65-70	12		8.82	
70-75	6	Combined	3.38	Combined
75-80	5		1.1	
Calculated chi-square value = 15.18 Tabulated chi-square value (5%) = 11.1 Degree of freedom= 5				

The alternative was to select another distribution; this time is the Log normal distribution which was chosen to represent the observed data. The mean and standard deviation of the observed data were found to be 52.28 and 10.68, respectively. This distribution was tested by graphical method that explained in the preceding section. The method is presented graphically in the figure (4-30), and results of the analysis performed are presented in table (4-30).

The regression line is used to find the best fit of the data. The coefficient of determination (R^2) is found to be (0.9625). The examination of (R^2) refers to the capability to represent the observed data by Log normal distribution because of high value of coefficient of determination.

Table 4-30: Statistical test results for the observed speed by Log normal distribution for segment two, south bound, at morning and evening periods

Class width(km/hr)	Log of column 1	Observed frequency	%Cumulative frequency
25-30	1.477	3	1.51
30-35	1.544	5	4.02
35-40	1.602	14	11.06
40-45	1.653	31	26.63
45-50	1.699	37	45.23
50-55	1.7404	29	59.799
55-60	1.778	32	75.88
60-65	1.813	25	88.44
65-70	1.845	12	94.47
70-75	1.875	6	97.49
75-80	1.903	5	100
calculated mean= 52.28 km/hr calculated standard deviation= 10.68 km/hr coefficient of determination (R^2)= 0.9625			

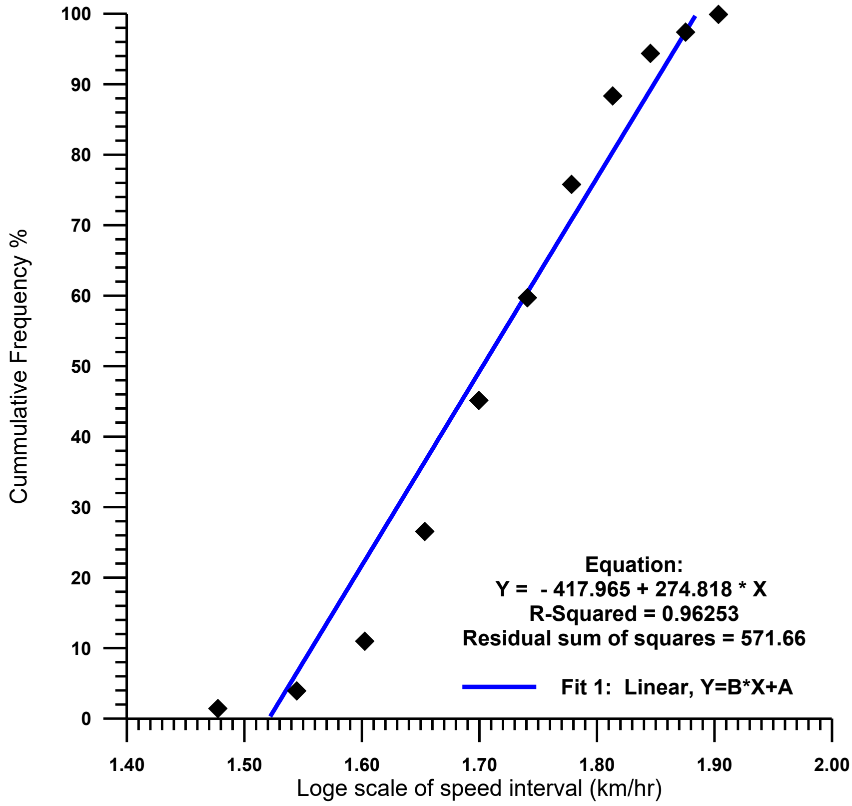

Figure 4-30: Transformation of percentage cumulative speed Lognormal distribution to linear relationship by changing the normal X scale to log scale for segment two, south bound at morning and evening periods

To complete the issue of the speed study section, table (4-31) below, indicates the concluded mean speed and the type of distribution for each segment of the road.

Table 4-31: Concluded mean speed and types of distribution for the two segments

Segment number	Mean speed (km/hr)	Type of distribution
One	49.78	Normal
Two	46.34	Log normal

The difference in the two segments is attributed to the following:

1. Existence of local roads opened to segment two, which led to make the entering vehicles moving slowly in the beginning will increase the number of vehicles that have low speeds.
2. The change in distribution is due to the existence of police check point in the beginning of segment two. The drivers after passing the check point will increase their speed until they reach their desired speed, which increase the number of vehicles that have speeds more than the mean speed. This will change the distribution of the observed data.

CHAPTER FIVE
DEVELOPMENT OF REGRISSION MODEL

5.1 General

The main objective of this chapter is to develop general regression models of speed flow density relationships at highway section. The traffic flow is the product of average space mean speed and density. This relationship is well known in the traffic engineering community as the fundamental relation of traffic flow. This means that if two parameters are known, the third can be estimated easily. Graphically, the relationship can be presented in fundamental diagrams. Fundamental diagrams are basic traffic flow curves that represent correlations between speed and density, speed and flow, and flow and density. Basic flow diagrams assume a linear relationship between speed and density and a parabolic relationship between flow and density for traffic stream characteristics observed on an uninterrupted traffic flow.

5.2 Development of Regression Model

Data survey was made to develop a regression model. The model which needs to be developed for this study represents speed-density relationship. Another set of data was collected from segments other than that used before. Speed data were collected by taking two reference points with known distances, and different durations of time which vary between (90, 60, 30, 15 and 10) minutes for the whole of 12 hours from video recording for both segments in the two directions (north and south), in morning and evening periods. The speed of each vehicle passing between these two points was calculated through dividing the distance over time that the vehicle spent between the two points. Several values of average speed were obtained; also the traffic flow was computed in the same time of each duration. The data survey of average speed and traffic flow are presented in table (5–1). The density can be calculated by substituting the values of average speed and traffic flow in relationship [Q=V*D].Their values are also shown in table (5–1).

Table 5-1: Surveyed data used for regression model development

Aver. space mean speed Km/hr	Flow Veh/hr	Density Veh/km	Aver. space mean speed Km/hr	Flow Veh/hr	Density Veh/km
47.863	575	12.014	38.284	786	20.531
45.197	658	14.559	41.936	558	13.306
48.497	508	10.475	41.63	519	12.467
45.896	588	12.812	40.962	539	13.159
52.21	460	8.810	43.105	542	12.574
45.8	556	12.141	43.02	604	14.041
45.98	620	13.484	44.732	574	12.832
48.75	664	13.621	43.879	484	11.030
40.38	648	16.048	34.313	464	29.571
53.35	620	6.973	42.604	452	10.610
44.67	582	13.031	20.00	660	33.00
40.22	794	19.741	46.252	425	9.189
47.22	672	14.231	47.284	433	9.157
36.28	916	25.244	50.867	434	8.532
51.21	576	11.247	44.215	410	9.273
37.15	906	24.389	43.002	460	10.697
35.26	1074	30.461	48.808	418	8.564
45.894	804	17.519	52.418	388	7.402
43.077	768	17.829	47.266	444	9.394
42.961	672	15.642	54.129	424	7.833
38.368	792	20.643	44.324	412	9.295
37.416	812	21.702	41.666	522	12.528
39.972	744	18.613	48.521	410	8.449
40.09	716	17.859	46.234	436	9.430
38.402	668	17.395	52.515	381	7.255
37.881	802	21.172	55.13	358	6.494
40.03	730	18.236	47.265	456	9.648
38.596	778	20.157	53.761	378	7.031
39.257	692	17.627	45.826	484	10.562
38.286	834	21.784	49.004	428	8.734
36.462	846	23.202	52.063	380	7.299
39.457	804	20.376	55.593	376	6.763
			54.625	340	6.224

The speed and density values were analyzed statistically through using regression analysis computer program. The main objective is to develop a statistical model, which represents the speed-density relationship. The regression technique results in an equation which is described as below:

$$Y = 57.065 - 0.9024 * X$$

It is important to mention that this model is applicable for free flow speed equal to (57.065) or less.

The determination coefficient (R^2) for this model is equal to (0.804). This value suggests that the obtained regression model explains about 80 % of the observed scattered data. The remaining 20% is not explained due to the random nature of other variables, which were drawn from statistical distribution. Figure (5-1) shows the scattered plot and the developed regression model for the relationship between average speed and density.

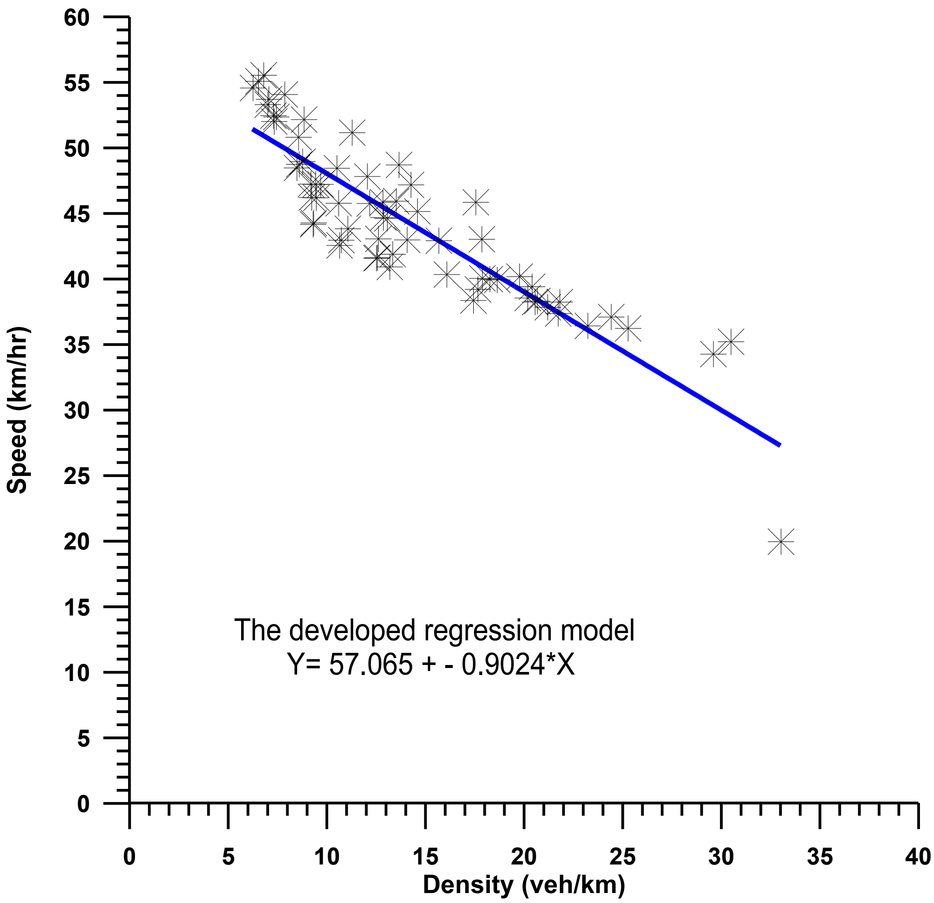

Figure 5-1: Relationship between the average speed and density value

5.3 Determination of Speed-Density Relationship

The above data obtained in the previous section which is analyzed statistically using linear regression technique is used to develop a statistical model that represents the relationship between speed and density .The general form of the adopted model for linear regression analysis is as below:

$$Y = a + bX$$

where:

Y = represent the average speed of vehicles in the highway
a = constant
b = coefficient of X , and
X = is the density of vehicles.

The above linear model is based on Greendhields linear model of the speed-density relationship, which is given below;

$$V = V_f - \frac{V_f}{D_j} D \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(5-1)}$$

where:

V = the average space mean speed of traffic flow,
V_f = the free flow speed,
D_j = the jam density of the traffic stream, and
D = average density of the stream.

The similarity between the adopted model for linear regression analysis and the general form of Greenshields model indicates that:

Y is equal to the value of V
a is equal to the value of V_f ,
$b = \frac{V_f}{D_j}$, and X is equivalent to the value of D

So, the final form of regressed speed-density relationship is presented as in following:

$V = 57.065 - 0.9024 * D$ …......…… (5-2) The developed regression model

5.4 Determination of Flow-Density Relationship

The flow-density relationship can be derived as described below:

$$V = V_f - \frac{V_f}{D_j} D \quad \text{and;} \quad Q = VD$$

where:

$Q =$ The traffic flow, therefore, by substitution

$Q = (V_f - \frac{V_f}{D_j} D) D$ and by simplification

$$Q = V_f D - \frac{V_f}{D_j} D^2$$

Substituting the constant values of the above developed regression model, results in;

$$Q = 57.065 * D - 0.9024 * D^2 \quad \ldots\ldots(5\text{-}3)$$

Equation (5-3) is the flow-density relationship.

5.5 Determination of Speed-Flow Relationship

The speed-flow relationship can be derived as described below:

$V = 57.065 - 0.9024 * D$ equation …..(5-2), therefore,

$D = (57.065 - V) / 0.9024$ and

$Q = 57.065 * D - 0.9024 * D^2$ equation …. (5-3), by substitution, the equation of the speed-flow relationship is as follows:

$$Q = 57.065 * [(57.065 - V)/0.9024] - 0.9024[(57.065 - V)/0.9024]^2 \quad \ldots\ldots (5\text{-}4)$$

The developed regression model that is presented in equation (5-2) is applicable for free flow speed equal to (57.065) or less. This result is compared with other similar studies in Baghdad city as shown in table (5-2) below:

Table 5-2: Comparison between the average free flow speed value of this study with other similar studies in Baghdad city

Study	2001 Younis Ahmed conducted on arterial road	2004 Al-kubaisi conducted on freeways	This study
Average free flow speed (km/hr)	55	70.85	57.065

The observed data for this study seems to be approximately consistent with the study of Younis Ahmed 2001, while it is underestimated compared with that produced by Al-kubaisi study 2004, the reason is attributed to the fact that Al-kubaisi study was conducted on freeways which have higher design speed limits.

5.6 *Verification of the Developed Regression Speed-Density Model*

Verification of the developed regression model output is achieved by selecting a proposed highway section similar to the two segments of Al-Tharthar urban street. The following assumptions are considered for this test:

1. Flow varied from 500 to 2500 veh/hr with an interval of 100 veh/hr.
2. Vehicles arrive at highway section at random with Negative exponential distribution of vehicle inter-arrival times.
3. Traffic composition is 95% passenger cars, and 5% heavy goods vehicles (trucks).

The **FWASIM** (**F**reeway **W**eaving **A**rea **SI**mulation **M**odel) is used to calculate the average speed and density, which corresponds to each level of vehicle flow. The density values produced from FWASIM are used as an input data for the regression model to calculate the speed parameter of the regression model expressed by equation (5-2). The obtained speed values of regression model are then compared with that predicted by the FWASIM model. Table (5-3) shows the data obtained from FWASIM program (predicted) and that obtained from regression model (calculated). The calculated speed from regression model and that predicted by FWASIM program is also plotted against the density variation as shown in figure (5-2). Comparison of calculated and predicted speed results is presented in figure (5-3).

Table 5-3: Predicted and calculated speed values

Flow veh/h	Predicated speed, km/hr from FWASIM program	Predicated density veh/km from FWASIM program	Calculated speed from equation (5-2) km/hr
500	85.8	3	54.36
600	84.9	3	54.36
700	84.5	4	53.46
800	83	5	52.553
900	82.5	5	52.553
1000	81.8	6	51.651
1100	80.8	7	50.75
1200	80.8	7	50.75
1300	79.5	8	49.85
1400	79.2	9	48.94
1500	78.1	10	48.041
1600	77.7	10	48.041
1700	76.2	11	47.14
1800	76.4	12	46.24
1900	74	13	45.33
2000	74.3	14	44.43
2100	74.2	14	44.43
2200	72.7	15	43.53
2300	72.1	16	42.63
2400	70.8	17	41.724
2500	69.7	18	40.822

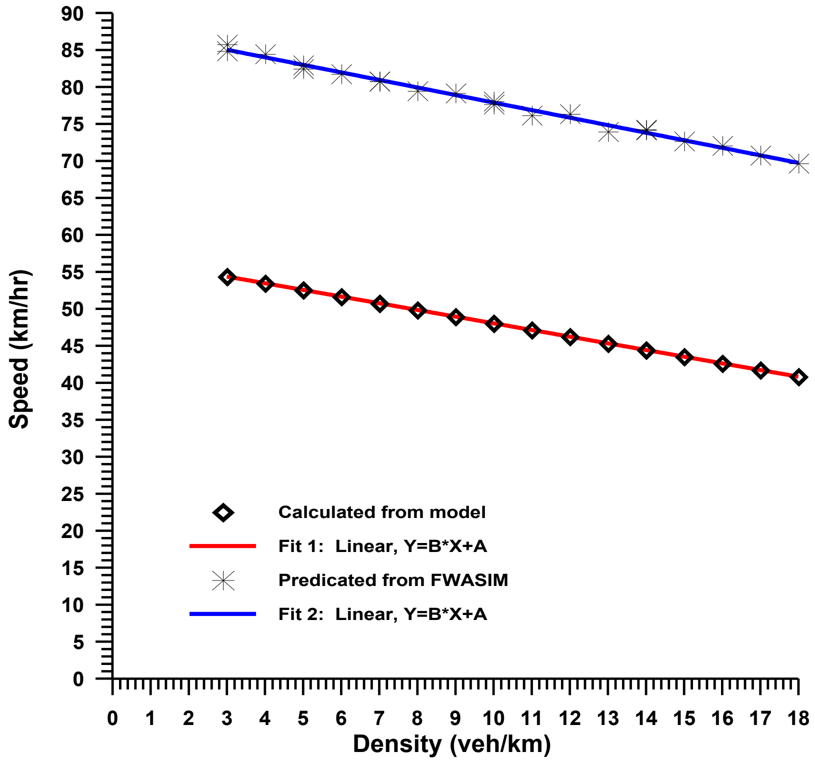

Figure 5-2: Effect of density variation on average traffic speed

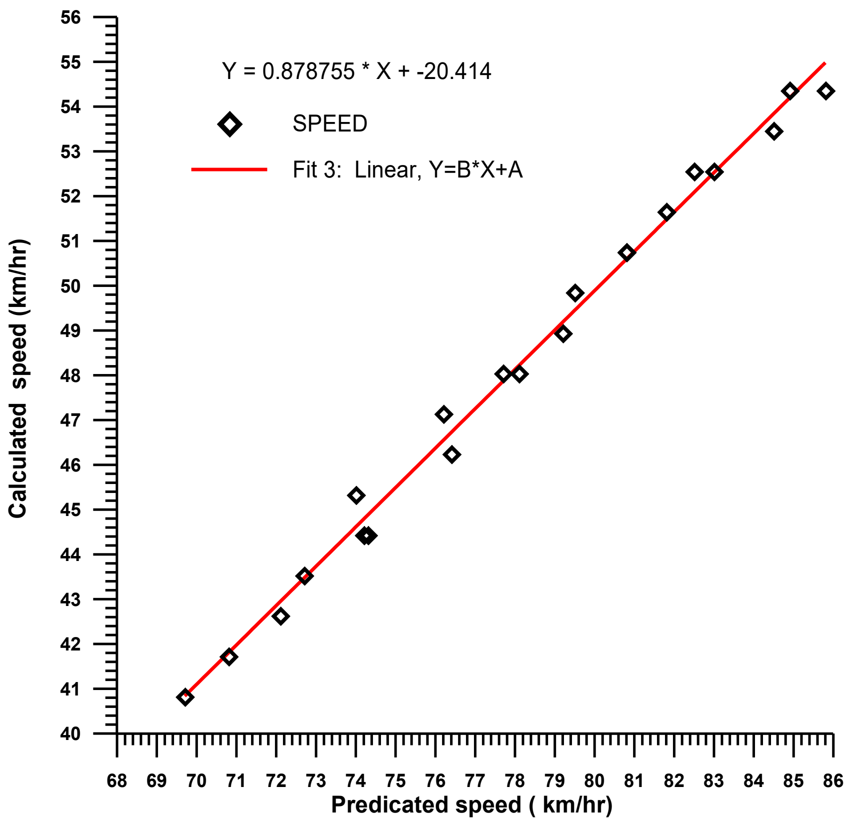

Figure 5-3: Comparison of the developed regression model and FWASIM program outputs

Figure (5-2) illustrates that the trend of both models goes symmetrical, however, there is a lower estimate of the relationship shown by regressed model. This may be attributed to the following:

1. The FAWSIM model is classified as microscopic because of its nature of handling the movements of vehicles and resolution and it is a mathematical model that captures the movement of individual vehicles, while the developed regression model considered as a macroscopic model because it is a mathematical model that employs traffic flow rate variables like (flow, speed and density ….etc).
2. The FWASIM model designed for freeways that deals with high speeds, while the developed regression model derived from the data collected on highway section that lies in commercial and residential regions which restricted the movement of vehicles.

5.7 Determination the classification of highway

To find the classification of Al-Tharthar highway section, the design speed should be calculated which represents the 85th–percentile space mean speeds (It is the speed below which 85 percent of the vehicles travel and above which 15 percent of vehicles travel) .

The 85th–percentile space mean speed is obtained from the cumulative frequency distribution curve as (49 km/hr), that illustrates in figure (5-4).The speed intervals and their cumulative percentages are presented in table (5-4).

Table 5-4: Observed values of space mean speed and their cumulative percentages for Al-Tharthar urban street section

Space mean speed class (km/hr)	Observed frequency	Cumulative % of all observations
30 – 35	1	1.56
35 – 40	14	23.43
40 – 45	19	53.11
45 – 50	18	81.24
50 – 55	10	96.87
55 – 60	2	100

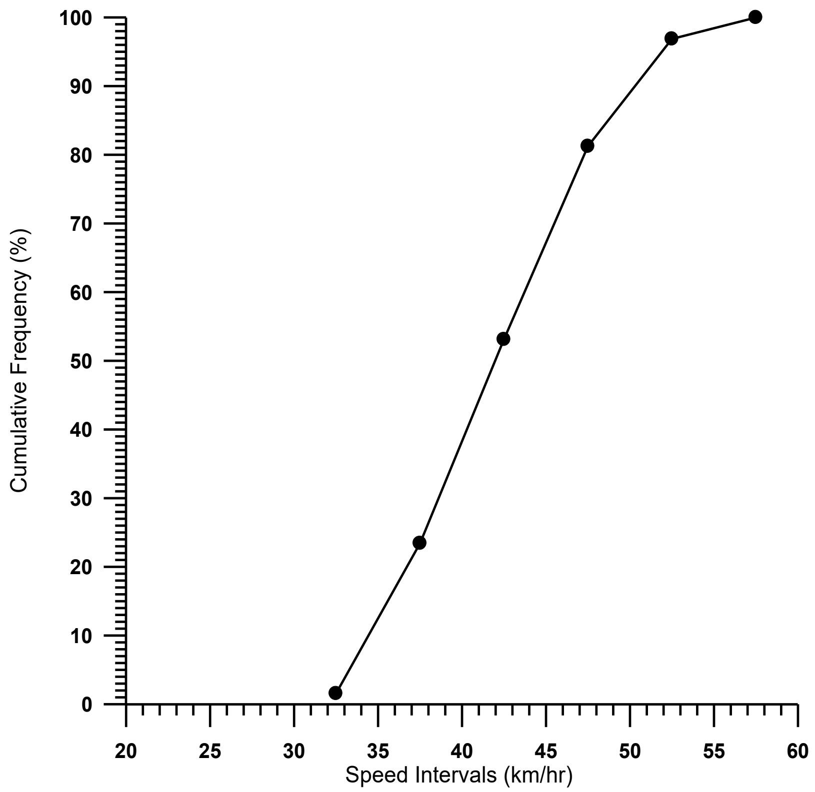

Figure 5-4: Cumulative frequency distribution curve for speed values of Al - Tharthar urban street section

Design speed is a factor in the design of collector streets. For consistency in design, a design speed of 48.28 km/hr [30 mph] or higher should be used for urban collector streets, depending on available right-of-way, terrain, adjacent development, likely pedestrian presence, and other site controls (AASHTO, 2001).

According to the above highway classification standards Al-Tharthar urban street section can be classified as a collector street because the design speed is equal to (49 km/hr), which located in an urbanized area in Falluja city.

5.8 Determination Level of Service

The term level of service is a quality 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, comfort and convenience (HCM2000).

The highway capacity manual gives six levels of service and defines six corresponding volumes for numbers of highway types, these volumes are referred to service volumes (hourly volume) which may be defined as the maximum number of vehicles that can pass over a given section of a lane during a specified time periods while operating conditions are maintained corresponding to the selected level of service. Although the HCM2000 recommends definitive values such as density, which are assigned to the level of service limits for each type of highway in the manual, no explanation is given of how these values were obtained in the field.

The surveyed highway (Al-Tharthar street) in this study can be considered as a divided multilane collector street connect located in an urbanized area in Falluja city with a traffic volumes range from (15000 to 40000) vehicles/day.

Any of the following three performance characteristics can describe the level of service (LOS) for a multilane highway:

1. Flow rate (pc/hr/ln).
2. Average passenger car speed.
3. Density.

Al- Tharthar street has the following performance characteristics:

1. Average speed = 44.27 pc speed.
2. Density = 14.2 pc/km/hr.

Highway capacity manual (HCM2000) introduces a figure (21-3) for speed–flow curves with level of service criteria for multilane highways including the above three performance characteristics. The following table can be concluded from the figure:

Table 5-5: Level of service criteria that concluded from figure (21-3) at HCM2000 in pc/mile/hr

LOS	Density (pc/mile/hr)
A	< 11
B	≥ 11 – < 18
C	≥ 18 – < 26
D	≥ 26 – < 35
E	≥ 35 – < 45
F	> 45

For comparison purpose the value of density according to table (5-5) above will be equivalent to the following values in pc/km/hr.

Table 5-6: Level of service criteria that concluded from figure (21-3) at HCM2000 in pc/km/hr

LOS	Density (pc/km/hr)
A	< 6.84
B	≥ 6.84 – < 11.19
C	≥ 11.19 – < 16.16
D	≥ 16.16 – < 21.75
E	≥ 21.75 – < 27.96
F	> 27.96

The density value for Al-Tharthar street lies between 11.19 and 16.16, therefore, its level of service is (C).

CHAPTER SIX
CONCLUSIONS
AND RECOMMENDATIONS

This chapter includes the conclusions, recommendations and proposals for future research.

6.1 Conclusions

Based on the findings of this study, the followings can be concluded:

1. The developed regression models explained in equations (5-2), (5-3) and (5-4) may be used to predict the speed-density, flow-density and speed-flow relationships respectively for Al-Tharthar urban street section in Falluja city and for similar highways in the similar areas .

2. The trend of the developed linear regression model output for predicting the speed-density relationship agrees well with that predicted by the **FWASIM** (**F**reeway **W**eaving **A**rea **SI**mulation **M**odel). However; the developed regression model result underestimates the model predicted by the **FWASIM** program.

3. The vehicle arrival distribution for the two segments in the north direction at both (morning and evening) periods was found to follow Negative exponential distribution; while in the south direction at both (morning and evening) periods was found to follow Erlang distribution. Both distributions were examined by graphical evaluation method.

4. The average speed of segment one with both directions and the south bound of segment two are approximately consistent which are closely to be (50.1 km/hr), also they have equal standard deviation (10.2) except the north bound of segment two which has an average speed equal to (43.95 km/hr) and standard deviation (10.814).

5. The free flow speed that calculated for Al-Tharthar highway section was found to be (57.065 km/hr) which can be applicable to similar highway sections .

6.2 Recommendations

1. It is recommended to change the road geometry, such as increase of the lane width to reach the lane width standards in order to improve the traffic capacity along Al-Tharthar highway section which leads to decrease delay and increase speed, especially during the peak periods.
2. Because of the existence of great width of shoulders along the both sides of the highway, it is recommended to make an On-Street parking on Al-Tharthar highway section to prevent stopping on the street which affects the capacity and speed on the road.
3. It is recommended to use the developed regression model to evaluate the speed-flow-density relationships for the selected highway and for all highways within the same facilities and specifications in the city.

6.3 Recommendations for Further Study

1. Further work is required for developing the predicted model. Improving the Quality of collected data, and estimating the model should be done with various and much data that provide the required variability to obtain the effect of various conditions, such as the effect of geometric characteristics of a roadway, for example, lane width, grade, pavement surface quality, on traffic behavior in the highway segment.
2. Making future studies for other streets in Falluja city to cover the large annual growth rate of vehicles in the city.
3. Improving the developed regression model to evaluate the fundamental traffic flow in other cites in Iraq.

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Figure A-1: Map of the location of al-tharthar urban street section in Falluja city

Figure A-2: Aerial photo of al-tharthar urban street section in Falluja city

B. Computational Steps of theoretical frequencies of headway distributions

The following computational steps which were used to calculate the theoretical values of headway distributions for the two segment and both directions are as shown below :

Sample of calculation

"Segment two - north bound, at morning and evening periods"

1. Computation of Erlang distribution theoretical frequencies :

$$* \text{ Mean time headway } h = \frac{\sum Fi * Ui}{\sum Fi} = \frac{3155}{644} = 4.899 \sec$$

$$* \text{ Variance } S^2 = \frac{\sum Fi * (Ui - h)^2}{\sum Fi - 1} = \frac{11944.44}{643} = 18.576 \sec^2$$

$$* \text{ Rate of flow } q = \frac{1}{h} = \frac{1}{4.899} = 0.204 \text{ veh/sec}$$

where:

Fi = Observed arrival headway
Ui = Mid class interval

(R.J. Salter, 1989) mentioned that the probability density function of the Erlang distribution may be computed by the following equation:

$$f(t) = \frac{(qa)^a}{(a-1)!} * t^{a-1} * e^{-aqt}$$

where:

* a= 1, 2, 3 ……
* q is the rate of flow (veh/sec), the reciprocal of the mean time headway
* the parameter a is an integer .
* mean time headway ($\bar{u}$) = a/b .
* variance (s^2) = a/b^2 .
* time headway (t) = mid class interval