TRAFFIC CHARACTERISTICS ON THE JEDDAH-MAKKAH FREEWAY, SAUDI ARABIA

A Thesis Presented to

The Faculty of the

Fritz J. and Dolores H. Russ
College of Engineering and Technology

Ohio University

In Partial Fulfillment
of the Requirement for the Degree

Master of Science

by

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JUNE, 2002
ACKNOWLEDGMENTS

I would like to thank Dr. Lloyd Herman for guiding me through these last two years. He has been an exceptional advisor as well as a good friend to me as he has led me down this path I chose. He encouraged and supported me to do my scientific trip during the winter quarter 2002, in my city Makkah, in Saudi Arabia. Also, a number of hours in his office were spent working on the project.

A special thanks goes to Dr. Ben Stuart and to Dr. Trevor Hale for being on my graduate committee.

A great thanks goes to my colleague, Engineer Hosam Abdulsalam, from the Hajj Research Center, Umm Al-Qura University for being a great helper in Saudi Arabia. Eng. Hosam spent many hours guiding me in the programming of the counters and their installation in the roadway.

Thanks also to my brother, Eng. Faisal Osra, a Ph.D. student at Ohio University, who volunteered to help install the counters. Also, special thanks go to Dr. Osama Albar, the head manager of Hajj Research Center. He supported and encouraged this project as a part of his encouragement of graduate students from Makkah.

A grateful thanks goes to my father, Abdulrhamn, and my mother, Mariam, for their efforts. They encouraged me by calling me and asking about the research. Also, thanks go to my son, Nadem, who is the closest person to me.
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CHAPTER 1

Introduction

1.1 Background

The Kingdom of Saudi Arabia is located in the southwestern portion of Asia. Viewing the desert Kingdom as a whole, the Eastern Region is, and will probably continue to be, the primary location in the country for heavy industry and manufacturing, while the Central Region is the political and administrative center of Saudi Arabia. The Western Region, in addition to carrying out its religious function for the entire Islamic world, has developed as the Kingdom’s main commercial, financial, and service activities center. Figure 1.1 shows the map of Saudi Arabia including the locations of the cities of Makkah and Jeddah.

Figure 1.1: Map of the Kingdom of Saudi Arabia
Both Makkah and Jeddah are located in the western region of Saudi Arabia. Makkah is the religious center while Jeddah is a commercial center. Makkah is at the intersection of latitude 21 to 25 degrees north and longitude 39 to 49 degrees east. It is set in a rugged landscape consisting of solid granite with rocks reaching 300 meters (1,000 feet) above sea level. Makkah is located 72 kilometers (45 miles) to the east of Jeddah. It is considered the holiest city of the entire Islamic world. The Grand Mosque of Haram is located in Makkah. Five times each day, the world's one billion Muslims, wherever they may be, turn to the holy city of Makkah to pray. The Hajj pilgrimage and Ramadan month of fasting generate the city's most intense economic activities, but there has been development of the various economic sectors of the city, such as distribution, construction, education, and governmental and professional services. The growth of Makkah City has been, and will remain, enhanced by its religious setting. Most of the population of 800,000 is concentrated in the old city, while densities in the modern residential areas are the lowest in the city. During the month of pilgrimage, the city is swells to more than 2,000,000 worshippers from other parts of the country and other Muslim nations all over the world.

Situated on Saudi Arabia’s coastline along the Red Sea, Jeddah has developed as the main commercial and distribution center within the Western Region. With the main seaport within the Kingdom, and an international airport, it acts as a transit point for the distribution of goods to other regions of the kingdom. The Ministry of Foreign Affairs and foreign embassies are located there. It also serves as the entry port for the vast majority of foreign pilgrims that come to the Kingdom by air and sea. All of these
functions have led to the development within Jeddah of a substantial services and
distribution sector. The service sector has served as a basis for the development of
light industry and is a reason why there is a higher concentration of people in Jeddah
than in Makkah. The population of Jeddah is 2,100,000 people.

Each year, there is an influx of pilgrims from all over the world to the holy city
of Makkah; they arrive largely through Jeddah to perform their religious obligations
(Omrah and Hajj). It has been the policy of the Saudi Arabian government to make
every effort to ensure the safety and comfort of pilgrims. This number increases
rapidly during the months of Ramadan and Dullhajjah. The peak of Omrah occurs
during the month of Ramadan. According to an Islamic belief, more reward for good
deeds is gained in the last third (ten days) of Ramadan compared to the rest of the
month. Visitors from around the Islamic world prefer to stay in Makkah during this
period, so the number of passenger vehicles increases during Ramadan, especially the
last ten nights, which causes traffic congestion on roads leading to Makkah.

In 1974, the Ministry of Communication recognized the need for increasing the
highway capacity between Jeddah and Makkah. Construction of the divided 8-lane
freeway between Jeddah and Makkah was completed in 1982 at a total cost of
$15,625,000. Figure 1.2 shows the alignment of the Jeddah-Makkah freeway.
Figure 1.2: The alignment of the Jeddah-Makkah freeway
The total length of the Jeddah-Makkah freeway is about 72 km (45 mile) with four lanes in each direction. The lane width is 3.65 m (12 ft), and the median island is 20 m (65.5 ft) wide. The freeway has seven overpasses. Lighting is provided along the entire alignment of the facility. The vertical alignment follows the terrain throughout the road and remains generally at natural ground level. The gradients throughout the length of the road are mild and do not exceed about 3% as measured from available photogrammetric maps and observed on site. The design speed of the freeway is 120 kph (74.5 mph). The speed limit is 110 kph (68.3 mph) for passenger cars and 80 kph (49.7 mph) for heavy trucks. In general, Saudi Arabia’s policy is that slower vehicles are passed from the left only, so the right lanes (1st and 2nd) are designed for heavy trucks and the left lanes (3rd and 4th) are designed for fast movement usually passenger cars.

There are six highways connecting different cities in the Western Region with Makkah. All of them are used throughout the year to transport goods, visitors, and pilgrims. The Jeddah-Makkah freeway has the most intense traffic volume compared to the other highways that lead to Makkah. For example, the traffic volume going toward Makkah in the month of Ramadan 2000 was 1,210,717 vehicles. This number was about 52% of the total traffic volume in the same month of all six highways, which totaled 2,271,909 vehicles. During the month of Ramadan in the year 2000, the traffic volume of the last ten days of the month was 525,133 vehicles, representing 43% of the monthly volume. The peak daily volume occurred on the 26th of the month.
and equaled 65,788 vehicles. The peak hour usually occurred between 4:00 pm and 5:00 pm.

It is crucial to understand the current flow characteristics and define the level of the service of any facility in order to plan for future traffic demand. In addition, it is necessary to determine freeway specific characteristics and calibrate the critical relationships and parameters for quantifying the existing operations. The main purpose of this research is to discuss these two issues as they relate to the Jeddah-Makkah freeway.

Freeways are multi-lane, divided highways with full access control designed to carry through traffic at high speed. The Highway Capacity Manual (HCM) divides the freeway into three components for analysis and design: ramps, weaving, and basic sections (HCM 1985). The analysis of the basic section is regarded as the most straightforward because the traffic maneuver here is simple, i.e., without merging, diverging, or weaving interactions. The quality of the traffic flows in any transportation facility is defined by the different levels of service. A level of service is a letter designation that describes a range of operating conditions on a particular type of facility. Six levels of service are defined; using the letters A through F. Level of service A represents the best level of service, and generally describes the operations of free-flow. Level of service F represents the worst operating conditions.

An understanding of interrelations among basic characteristics of vehicular traffic flow, such as volume, speed, and density, is of prime importance to the practicing traffic engineer. From the standpoint of design, knowledge of speed-flow-
density characteristics is required for the prediction of highway capacity. Those concerned with traffic operations are faced with the problem of providing an adequate level of service, which calls for understanding of the entire range of relationships. The relationship between speed and flow of basic sections on freeways is typically represented by the speed-flow curves in the Highway Capacity Manual. The relationship between speed and flow is governed by the interaction between the driver, the vehicle, and the highway elements. Therefore, it is, influenced by driver behavior, vehicle performance, and highway design and all of these have changed over time. Drivers are able to travel longer distances and are better acquainted with the freeway system. Vehicles are also designed to give better performance, and highway elements have been improved for increased safety and capacity. In light of these factors, changes in the speed-flow curves can be expected. The objective of the current research developed from the lack of information about specifying the level of service (LOS) on this intensive traffic volume freeway.

The purpose of this study is to examine the speed flow characteristics of a basic section on the Jeddah-Makkah freeway. Chapter 1 ends with the specific objectives of this study. Chapter 2 reviews previous studies and developments of speed-flow-density relationships and the level of service. Chapter 3 explains all the methodologies implemented during the Jeddah-Makkah freeway project. This includes site selection, time period selection, data acquisition, field procedures, data reduction, and data analysis. Chapter 4 presents all the results of volume variations, free-flow speed, speed distributions, passenger car equivalent, lane distributions, and speed-
flow-density relationships. Chapter 5 consists of the conclusions that can be drawn about the traffic characteristics on the selected section of the Jeddah-Makkah freeway.

1.2 Objectives of this study

The specific objectives of this project are:

1. Estimation of the level of service values for this important highway, using the U.S. Highway Capacity Manual.

2. Definition of levels of service of a basic section on the Jeddah-Makkah freeway based on the actual speed-flow and speed-density relationships.

3. Identification of significant differences in results between HCM and highway specific results.

4. Creation of a database for the eastern direction (toward Makkah) of the freeway, including determining the traffic volumes, vehicle classifications, average speeds, and headways in each lane.

5. Determination of the traffic volume and density in the peak hour.

6. Determination of the free-flow speed in every lane.
2.1 Speed-flow-density relationships

The analysis procedures for basic freeway sections are based on the relationships among speed, flow, and density under ideal traffic and roadway conditions. These relationships define the level of service criteria and are the base against which all adjustment factors are calibrated. The rate of flow is the product of speed and density. Figure 2.1 illustrates the conventional or idealized form of two-dimensional relationships between flow and speed, flow and density, and speed and density. The relationship between speed and density is monotonically decreasing; that is, as density increases, speed decreases. Flow is zero when either speed or density is zero. Speed becomes zero when the density is so high that all movement stops. When density is zero, there are no vehicles on the road. As density increases from zero, flow increases. As density continues to increase, a point is reached where the corresponding decrease in speed is sufficient to cause flow, the product of speed and density, to decrease. The point of maximum flow is capacity. Figure 2.1 also illustrates several important parameters, which are defined as follows:

\[ c = \text{capacity, the maximum rate of flow (vehicles per hour, vph)} \]
\[ S_c = \text{critical speed, the speed at which capacity occurs (miles per hour, mph)} \]
\[ D_c = \text{critical density, the density at which capacity occurs (veh. per mile, vpm)} \]
S_f = free-flow speed, the theoretical speed of traffic when density is zero (mph).

Figure 2.1: Basic form of speed-flow-density relationships (TRB 1965)

Figure 2.2 shows a more realistic form of the speed versus flow relationship for multilane uninterrupted flow based on more recent research. It reflects four distinct regions of flow, and illustrates several critical definitions. Region 1 describes a range in which speed is insensitive to flow levels. The constant speed in this range is defined as the free-flow speed. Region 2 describes a range at which speed begins to decline in response to increasing levels of flow. This region terminates at point 2, the point of maximum flow, which is defined as capacity. When arrival flow exceeds the capacity of a segment to discharge flow, a queue forms. The discharge from the queue is relatively stable, and is shown as Region 3 in Figure 2.2. Region 4 depicts flow...
conditions within the queue that forms behind a segment or point at which arrival flow exceeds the capacity to discharge. Such a point is referred to as a “breakdown point.” Regions 1 and 2 represent the uncongested flow while Regions 3 and 4 represent the queue discharge and within the queue flows respectively (Hall, Hurdle, and Banks 1992). This understanding of the speed-flow relationship for a freeway was developed over a period of time as described in the following paragraphs.

Figure 2.2: General form of multilane speed-flow relationships (Schoen, May, and Urbanik 1994)

It has been proposed that the best speed-volume diagram description can be obtained from three distinct zones, which has lead to a clearer understanding of the theory of traffic flow (Underwood 1960). These zones are normal (uncongested) flow, unstable (queue discharge) flow, and forced (within the queue) flow.

Five regression hypotheses of speed against density were analyzed: linear forms, the Greenberg exponential curve, Underwood’s transported exponential curve,
Edie’s discontinuous exponential form, and May’s suggestions (Drake, Schofer, and May 1967). The research predicted that the mean free speed test using Greenshield’s hypothesis was superior to that of all other hypotheses. On the one hand, the values of the modified Greenberg hypothesis were low for maximum flow, mean free speed, and optimum speed. On the other hand, the Edie formulation gave the best estimates of the fundamental parameters (mean flow speed, optimum speed, maximum flow). Figure 2.3 shows the illustration of speed-density hypotheses for some of the models mentioned above. Figure 2.4 shows the locations of upstream, bottleneck, and downstream portions in the speed-flow-density relationships.

Figure 2.3: Speed-density hypotheses (Drake, Schofer, and May 1975)
Figure 2.4: Location of upstream, bottleneck, and downstream portions in the speed-flow-density relationships (Drake, Schofer, and May 1975)

Speeds on uncongested freeway sections and in bottlenecks are not a function of flow but only of whether the vehicles are or are not being discharged from an upstream queue (Hurdle and Datta 1983). This hypothesis was based on the data of the publication. Also, the review of research confirmed that urban freeway speeds remain high until the flow reaches at least 75 percent of the roadway capacity and that 2000 pcph is still a good estimate of the capacity of a North American freeway lane under ideal conditions. On the other hand, Hurdle and Datta found an average speed of almost 80 kph (50 mph) for capacity flows, much higher than the 50 kph (30 mph) that is so often given in traffic engineering books.

At low to moderate flows, speed is insensitive to flow (Persaud and Hurdle 1988). Also, the authors suggested that speed at higher flows does decrease with
increasing flow. Moreover, they believed the concept that the upper branch of the entire speed-flow diagram represents unsteady free-flow conditions, the lower branch represents unsteady operation characteristics of conditions in a queue, and capacity occurs where the two branches meet.

A study of the speed-flow characteristics of basic freeway sections was performed near the Caldecott Tunnel located in San Jose, California (Chin and May 1990). The study found that by using detector information, well-defined speed-flow relationships could be determined because a large number of data can be obtained readily available, a wide range of flows can be observed, and speed measurements can be done precisely. In addition, the authors proposed if video recording is employed for gathering speed-flow information, 1) the camera needs to be as vertical as possible to utilize a good vantage point, and 2) a sufficiently long period of observation covering a wide range of flow should be obtained. These requirements would certainly make site selection more difficult and data extraction more tedious. Chin and May found that the HCM speed-flow curve for freeway sections is not representative of the speed-flow characteristics at the Caldecott Tunnel. The capacity value of 2000 passenger car per hour per lane (pcphpl) in the 1985 HCM curves seems to be low compared to the high flow values observed at most of the sites. A serious recalibration of the 1985 HCM curves for freeways may be necessary. Finally, they found that the JHK (a company which provides technical consulting in the areas of traffic management) speed-flow curves for multilane highway analysis seem to predict the conditions at the sites studied better than the HCM speed-flow curves, especially in the case of the
Caldecott Tunnel experiment. This result is due to the use of lower values of equivalent factors for heavy vehicles and a higher capacity value in the analysis. However, the estimated free-flow speeds based on speed limit and adjustment for non-ideal lateral clearance is not always in agreement with the speeds observed. From their data results, a speed drop of only 5 mph from no flow to capacity seems to be too small. A drop of 10 mph seems more likely.

There is no flow rate reduction on the Queen Elizabeth Way in Mississauga, west of Toronto, Ontario across three lanes of a level expressway when there is an upstream queue, although there is a speed reduction (Hall and Hall 1990). The authors stated that there might not be just one speed-flow curve for a specific freeway type. Also, the maximum observed flow in the queue depends on the net rate of flow at the downstream bottlenecks. They proposed that it would be necessary to identify two speed-flow curves for a given facility type, one in the bottleneck and one in the queue. The curve portion in the bottleneck is used to identify both operations expected before the upstream breakdown (a horizontal line) and the speeds expected (at the same maximum flow) at different distances downstream of the head of the queue (see Figure 2.4). On the other hand, the curve in the queue is necessary to combine data from several different locations to arrive at a complete representation of the lower portion of the speed-flow curve. On a freeway system, the flow rates seen at a particular location will depend on the net entering flows between that location and the primary bottleneck. Hence, one location cannot be expected to provide a large range of
congested flows. Therefore, the speed-flow relationships cannot be simply ascertained at a congested location nor can capacity be identified.

The capacity of freeway segments might be reached in only one or two lanes before a bottleneck forms (Urbanik, Hinshaw, and Barnes 1991). The publication stated that plots of 1-minute flow rates would show the extent of the scatter in the observed traffic flow, without illustrating the patterns that can be seen from the 5- and 15-minute flows. Through data results, the average 5-min flow rate has more fluctuations in traffic flow than 15-minute flows.

It is impossible to identify the shape of the speed-flow-occupancy (density) relationships from one location (Hall, Hurdle, and Banks 1992). Actually, uncongested operations on freeways are consistent with the speed-flow figure of multilane rural highways approved for a revised Chapter 7 of the Highway Capacity Manual (HCM 1985). The authors hypothesized that the speed at the right-hand end of a congested branch of the speed-flow curve (Region 3) is constant for similar types of freeways and similar driver populations. Thus, a 20 percent reduction in speed of from 100 kph (60mph) to 80kph (50mph) in Region 3 depends on free-flow speeds, which are probably affected by posted speed limits and level of enforcement (see Figure 2.2). The publication raised several questions about the speed-flow relationship. One, in particular was, “the difficulty in firmly fixing the shape and location of the curve” beyond about 1500 vehicles per hour.
Generally, it has been noted that if regression analysis is to be used to try to develop equations from the data, then it might be better to retain the data for the shortest available interval (Hall, Pushkar, and Shi 1993).

Two issues were raised related to the effect of location on congested regime flow-occupancy data and relationships (Hsu and Banks 1993). The first of these dealt with was the effect of traffic entering or exiting the freeway between the point of observation and the bottleneck. It has been shown that the clusters of data representing congested flow at adjacent locations should be offset by a flow equal to the average difference between traffic entering and traffic exiting between the two locations. The authors concluded that maximum flow would decrease in the upstream direction when flow entering exceeds flow exiting and vice versa. The second issue concerned the effect of lane drops that occur in the section occupied by the queue. They found that when locations upstream and downstream of the lane drop are compared, the approximate slopes of the congested regime of the flow-occupancy relationship are roughly proportional to the reciprocal of the number of the lanes. This implies that average densities remain approximately unchanged across the lane drop. As this issue was investigated only for the case of drops from five to four lanes, the conclusion must be considered to be tentative.

2.2 Level of service and capacity of freeway

A level of service (LOS) describes a range of operating conditions on a particular type of facility. Six levels of service are defined, using the letters A through F. Level of service A represents the best level of service, and generally describes
operations of free-flow on freeways. Level of service F represents the worst operating
conditions and occurs when the arrival flow is greater than departure flow for a
segment. LOS E most often represents flow at or near capacity. Other letters (B, C,
and D) identify intermediate conditions. Figure 2.5 illustrates levels of service for a
freeway segment, defined in terms of density.

![Figure 2.5: Levels of service for freeways](image)

The Highway Capacity Manual defines the capacity of a facility as “the
maximum hourly rate at which persons or vehicles can be reasonably expected to
traverse a point or uniform segment of a lane or roadway during a given time period
under prevailing roadway, traffic, and control conditions.” Roadway conditions refer
to geometric characteristics of the facility such as lane and shoulder widths. Traffic
conditions refer to the mix of passenger cars, trucks, buses, and recreational vehicles
in the traffic stream (HCM 1985).
The standards for freeway level of service were recommended (Roess, McShane, and Pignataro 1980). The development of recalibrated standards for level of service is based on certain characteristics such as average running speed should be used to establish speed criteria for the various levels of service (A through F). In addition, levels of service are defined by using the relationship between speed and density. The research by Roess, McShane, and Pignataro concluded, because of the previous characteristics, speed remains relatively constant over a wide range of volumes (1600 pcphpl-2000 pcphpl) and then deteriorates rapidly over a relatively small volume range as 2000 pcphpl is approached. The authors also proposed that design at LOS C, D, and E on a freeway with 112 kmh (70 mph) average speed should not be attempted because all are in a fairly unstable range of flow and a small error in estimated volumes would mean regular breakdowns. This leaves just two choices for a design level of service: A or B. Density is not observed directly but computed from speed and volume. Finally, they mentioned that the traditional use of levels of service C and D for urban design should be altered because both are too close to the 2000 pcphpl mark for reasonable stability.

The 1985 HCM used the worst 15 minutes of the analysis hour instead of using the full hour as in the 1965 HCM to characterize the operating conditions (Roess and McShane, and Pignataro April 1987). Also, the volume to capacity (v/c) ratio is used to define LOS in the 1965 HCM, whereas the maximum rate of flow accommodated in a 15-minute period is used to define LOS in the 1985 HCM. On the other hand, the authors mentioned that the capacity on freeways under ideal conditions was similar for
both HCM 1965 and HCM 1985 and equaled 2000 pcphpl. Finally, they confirmed the concept of person-capacity that is introduced in the 1985 HCM and used to cover transit and pedestrian methodologies that deal with person units rather than vehicle units.

The 1985 HCM does not use average travel speed as a measure of effectiveness for freeway sections (Roess and McShane May 1987). Density was selected as the primary measure of effectiveness for basic freeway sections because it varied with flow throughout the entire range of flows from 0 pcphpl to capacity. In addition, LOS for basic freeway sections in the 1985 HCM was selected based on calibrated speed-flow-density relationships for freeways under ideal conditions. These curves were modified in the 1985 HCM to reflect the gradual trend of increasing speeds at low volumes on the nation’s highways. Also, speed is generally insensitive to flow rate at capacity ranges from 0 to 1600 pcphpl (change in average speed is about 3 mph). While capacity is approached, speed drops rapidly with small increases in flow, i.e., in the range of 1600-2000 pcphpl, speed drops from 53 to 30 mph.

The reduction in the capacity flow as an average from over 6,400 vph to under 6,100 vph on the Queen Elizabeth Way (QEW) after queue formation appeared to be about 5 to 6 percent (Hall and Duah 1991). The authors stated that once congestion occurs on a facility, it is the queue discharge flow rate that will govern the time to recovery. Also, if queues can be delayed during high flows, there is a 5 to 6 percent “bonus” flow available.
2.3 Passenger car equivalent

Passenger car equivalent accounts for the displacement of passenger cars due to both the size of the heavy vehicle and the gaps created in the traffic stream as a result of their non-uniform operating characteristics.

The recalibration of truck, bus, and recreational vehicle equivalents for specific grades was performed based on the results of freeway simulations performed by the Midwest Research Institute (MRI), and truck weight-to-power-ratio studies and operating characteristics (Linzer, Roess, and McShane 1980). Linzer, Roess, and McShane also identified the correction factors for the recalibration of recreational vehicles under ideal conditions as they appeared in the 1965 HCM. These factors are based on automobile equivalents to one truck or bus under specified traffic and roadway conditions. The weight to power ratio of typical trucks is used to determine motor vehicle performance characteristics. The parameters used in MRI simulations include flow rate, distribution to lane by vehicle type, spot speeds, lane-changing frequencies, vehicle population, and overall travel speeds. The design information includes the following parameters: number of lanes, design speed, grade, total flow rate, percentage of trucks, implied capacity, service level, operating speed, and percentage of implied capacity. The implied capacity equals the simulation flow rate divided by the capacity. It is used because an actual test to obtain capacities has not been made at each location. Further, the authors proposed the MRI concept of percentage reference trucks as a good base for future studies of this field.
Various approaches to calibration and interpretation of passenger equivalent values in highway capacity analysis have been presented (Roess and Messer 1984). Roess and Messer concluded that the recommended revisions to passenger car equivalent values for multilane uninterrupted flow are based on evaluation of the latest research and data and should result in improved accuracy of analysis procedures for these types of facilities.
CHAPTER 3
Methodology

Chapter 3 describes the procedures used during the Jeddah-Makkah freeway project, including site selection, time period selection, data acquisition, field procedures, data reduction, and data analysis.

3.1 Site selection

A typical cross-section of the Jeddah-Makkah freeway was selected for this research, so the effects of any weaving area turbulence is negligible and can be ignored. The selected site is a portion of the Jeddah-Makkah freeway spaced 41.5 km from Makkah City. The data was collected for traffic entering Makkah only since this direction was more congested than the direction toward Jeddah. The section has four lanes in each direction with a total width of 15m. The lane width is 3.65m. A median island separates the two directions with a width of 20 m. Side shoulders are provided. The width of the right shoulder is 2.5 m while the left shoulder is 2m wide. The grade is level at this part of the freeway. The selected portion of the road is located under, BAHRA NO. 2, one of the overpasses that exist along the entire alignment of the Jeddah-Makkah freeway. The following overpass toward Makkah is spaced 18 meters from BAHRA NO. 2. Figure 3.1 shows a diagram of the selected site on the Jeddah-Makkah freeway. It represents the typical cross-section of the selected site; locations of the traffic counters and the two cameras are marked.
3.2 Time period selection

The analysis procedures for basic freeway sections are based on the relationships between speed, flow, and density under ideal traffic and roadway conditions. To study these relationships, data must be collected throughout the range of stable flow from very light traffic to capacity operation. The research took place during Ramadan (December 2001), which is the ninth month according to the Al-Hijri calendar. The most intensive traffic volume of the year takes place during Ramadan. It
represents 11% of the total annual traffic volume. This month is characterized in the Islamic religion by fasting from sunrise until sunset. In addition, the rewards for good deeds are multiplied in this month, especially during the last ten days, compared to the rest of the year. Therefore, the 26th day during that month was selected to perform the data collection. According to the Prophet Mohamed, during the 26th night of Ramadan, known as Al-Qadr, the rewards are equivalent to the rewards of 1000 months. The 26th of Ramadan is the most crowded night of the last ten days of Ramadan because both visitors and locals like to go to Makkah to pray in the Holy Mosque and do other religious activities. The peak hour of traffic on the 26th of Ramadan occurs between 4:00 pm and 5:00 pm, so the Jeddah-Makkah freeway is heavily used. The direction from Jeddah to Makkah has more intensive traffic than that from Makkah because the times of the religious events are specified and limited, while people who leave Makkah can do so at any time during the day. On the 26th of Ramadan 2001, the daily traffic volume headed towards Makkah on this freeway was the most intense of the last ten days of the month in both directions. According to the traffic counter, 65,788 vehicles were eastbound, which is about 5.5% of the total volume in the whole month. On the other hand, the number on the same day going in the other direction (Makkah-Jeddah) was 57,387 vehicles.

3.3 Data acquisition

Data was collected for the following parameters: volume, speed, classification, and headway. The attempt to collect data over a broad range of traffic flow rates was done in order to provide the necessary data points for regression analysis. Data must
be collected throughout the range of stable flow from very light traffic to capacity operation. If data are distributed throughout the range required, a minimum of 50 to 100 data points should be collected [HCM 1997]. Another requirement is to gather data on one day in order to have the low-volume hours based on the same type of vehicle and driver characteristics as the high volume hours. Data should also be collected where lane widths and lateral clearances are standard.

Two data acquisition methods were used in this research: 1) portable traffic counters and 2) video image recorders and subsequent machine vision processing. The HI-STAR portable traffic counter has the capability of monitoring traffic flow conditions. The concept behind all HI-STAR counters is the technology of magnetic imaging of motor vehicles. Vehicle Magnetic Imaging (VMI) is a technique in which very sensitive and low-power sensors acquire energy-related signals from the earth’s magnetic field. Every motor vehicle has various parts that are constructed from ferrous metal, which include iron and many other common elements. These metal parts have many levels of magnetic mass that exhibit properties somewhat like a magnet. When a vehicle passes over the HI-STAR counter, the magnetic mass of the vehicles metal parts interfaces with the earth’s magnetic field. This action creates electrical signal changes in the HI-STAR sensors. These signals are directly proportional to the vehicle’s magnetic mass.

As a result, the HI-STAR computer can count each vehicle, measure its speed, determine its approximate length, and report occupancy. The computer’s small size provides for easy installation at virtually any traffic location.
3.4 Field procedure

Since the main purpose of collecting data is to develop the curvilinear relationship between speed and flow, the values of the volume and the corresponding speed are required. Traffic counters and video image methods are used to collect the data for both volume and speed.

3.4.1 Portable traffic counter set-up and installation

The traffic counter Nu-metrics model NC-90A was used to count the vehicles and record their corresponding speed. A total of four counters was utilized, one in each lane. Figure 3.2 shows the traffic counter (NC-90 A) used in this study. The NC-90 A counter measures 305 × 140 × 16 mm (6.5 × 5.5 × 6.25 inches).

![Traffic counter model NC-90A](image)

Figure 3.2: Traffic counter model NC-90A

The period of the study was 22 hours, starting at 8:00 am of day 26 and ending at 6:00 am on the 27th of Ramadan. Five-minute intervals were programmed in the traffic counter to collect the data during that period, so a total of 264 (5-min) periods
was generated. The one-minute interval is the smallest period for which the counter can collect data but the five-minute interval was used for this research because of memory shortage, especially for long periods.

The design of the HI-STAR electronic circuits and computer section utilizes various timing techniques to conserve battery power. When the system is first powered, only one of the sensor circuits is activated. When no traffic is present, the computer is in a “sleep” mode drawing very little power. Upon detecting a vehicle, the computer switches to a “run” mode where most of the power is consumed conducting measurements. During this measuring period, when the vehicle is present, the computer will switch between the “run” and “sleep” modes to further conserve power. After a vehicle passes the HI-STAR, the computer returns again to the “sleep” mode [HI-STAR manual 1995]. Traffic volume and the vehicle’s speed are collected. Since the HI-STAR counters inherently measure vehicle speed, headway can be calculated as the inverse of the volume value. The usage speed unit is miles per hour, which is then converted to kilometers per hour. As the vehicle continues to approach the counter, the computer records the sensor’s electrical changes and computes the vehicle’s speed. This calculation occurs very quickly and quite often before the vehicle even passes over the counter. A HI-STAR counter completes the vehicle’s speed measurement in less than 20 milliseconds for vehicles traveling at 50 mph (80 kph). A total of 15 speed bins are available to categorize the speed values in the programming of the counter. The speed range selected for this project was 60 kph to 145 kph, because this is the average range of vehicle speed.
It was desired to use a speed range of 5 kph for each bin in order to provide good precision; however, there were not enough bins available to cover the overall speed range of interest. Therefore, a speed increment of 10 kph was used for the speed bins between 60 kph and 90 kph, while speed increments of 5 kph were used for speeds between 90 kph and 145 kph, because the majority of vehicles show this range of speed according to previous records and observations.

Once the speed is known, vehicle length is just a matter of measuring how long in time the vehicles are over the counter. The traffic counter is set to categorize the vehicle types to passenger car, small bus, big bus, and truck. They are grouped and differentiated by length, using class bins. Each bin has a specific value. If the vehicle passes the HI-STAR counter and its length is equal to or less than the recorded value for the first bin then the vehicle is counted within that bin.

Four vehicle classification bins were used. They were set to classify the lengths 6 m, 9 m, 12 m, and 15 m, which represent passenger cars, small buses, big buses, and trucks, respectively. The original units of the vehicle lengths used in the programming of the HI-STAR counter were in feet (19 ft, 30 ft, 40 ft, 50 ft), which were then converted to meters. A total of 15 speed bins and 8 general type class bins were provided in the HI-STAR.

Before installation of counters, the police responsible for detecting vehicles that exceed the speed limit helped the installation crew to control the traffic movement. They began to transfer the traffic fleet to the first and second lanes by blocking the third and fourth lanes and preventing the vehicles from using them. Two
police cars stopped obliquely 500 m apart from the location of the installation in lanes 3 and 4 to prevent vehicles driving in these lanes. Traffic movement was converted temporarily to lanes 1 and 2 during the installation of the counters in lanes 4 and 3, respectively. Figure 3.3 shows the police cars blocking the traffic movement in lanes 3 and 4 and directing the entire movement to lanes 1 and 2. Figure 3.4 depicts the second and last phase of the installation by converting the traffic fleet to lanes 3 and 4 in order to install the counters on lanes 1 and 2.

Figure 3.3: The blocking of traffic movement in lanes 3 and 4 by police cars
The HI-STAR counters were installed a horizontal distance of 48 m from the overpass. The counter is designed to be installed in the middle of the lane. The plastic rectangular cover used to maintain the counter and save it from damage is shown in Figure 3.5. The cover was capped on the counter and fixed in the ground with 8 nails using a handgun. Traffic flow direction is indicated by an arrow on the cover, which is also used to check proper installation. Figure 3.6 shows the installation of the cover in the ground.
Figure 3.5: The plastic cover of the traffic counter

Figure 3.6: The installation process of the traffic counter
3.4.2 Video camera set-up and position

Two video cameras were used to record the traffic movement during the period from 4:00 pm to 5:00 pm on the 26th day of Ramadan. According to records of previous years, the peak hour on this day of the month occurred at this time. Both cameras were at a height to permit observers to see the entire roadway with no difficulties. One-minute intervals were used to analyze the volume and the corresponding speed values. One camera was used to count the volume in each lane according to passenger car, small bus, big bus, and truck. It was located on the overpass with a vertical height of 7 meters above the freeway. The camera lens was oriented to the traffic flow such that the vehicles were moving toward the camera.

The total period of recording was 5 hours, from 2:00 pm to 7:00 pm. A couple of minutes during this period were not recorded due to battery change and rest times. However, the focus for analysis was on the peak hour for the analysis in order to record the values of the flow and corresponding speed. The camera was set up above the roadway in the mid-point of the direction toward Makkah to cover the entire width of four lanes. The second camera was set up laterally on the off-ramp of the overpass in the direction to Makkah. It was used to analyze and determine the speed values during the peak hour for all vehicles in each lane. It also was used to categorize the vehicles into the same four vehicle types mentioned earlier. This camera was oriented perpendicular to the traffic flow. The speed was detected by specifying a zone of 18 m long within the captured video image. Specifying a longitudinal section in the road with a length of 18 meters is necessary for the detection method for speed. The speed
value is then calculated by dividing this amount of distance by the required time elapsed to pass this zone. One of the tasks of the second camera was to make a check of the volume and the speed values recorded in the traffic counters because its location was on the same lateral line of the counters; therefore the values were supposed to be the same. The values of the volume and the speed recorded from the second camera were added to represent the 5-minute interval of the same time as appeared on the counters.

3.5 Data reduction

The HI-STAR Data Management Software (HDM) was used to program and retrieve data from a portable Hi-STAR counter for assimilation of count, speed, classification, temperature, and wet/dry roadway conditions. The data in the traffic counter was downloaded to the hard disk and saved as two Excel files. The first file contains the information for temperature, humidity, period start, total volume, and categorized volume by type. The second file contained the number of vehicles in each speed bin.

In addition, the peak hour in that day, 4:00 - 5:00 pm, was analyzed manually by using video taping and then transferring the recording period from normal video tape to a computer file with (avi) file format in order to use it with one of the software video editors. The software used was called Ulead Media Studio Pro 6.0, Video Editor 6.0(2).

Since the vehicles in the freeway move rapidly and can pass the distance of 18 m in less than one second, a fraction of a second was used for more accuracy. One
complete second was set to be 29.5 frames. The error was plus or minus one frame, which equals 0.03 seconds. Then the elapse time (difference between section entrance and exit) to pass this distance was recorded. After that, the speed value was calculated by dividing the distance by the time. This procedure was done for all lanes with different vehicle types.

3.6 Data analysis

The main data base file obtained from the traffic counters of both volume count and speed detection for each lane was converted to the spreadsheet in Excel file format in order to conduct different analysis procedures. Data was analyzed and tabled for the determination of the peak 15-minute, average headways, passenger car equivalent, free-flow speed, and speed-flow-density values in all lanes. In addition, data was plotted for lane distribution, comparison between 5-minute and 15-minute interval variations along the length of the study, distribution of speed, and scatter diagrams of speed-flow-density relationships. Finally, the calibrated speed-flow, speed-density, and flow-density curves were developed and plotted using the regression analysis. To examine the accuracy of the traffic counters, the detector signals were checked by comparing them with video observation at the site. Vehicle counts obtained from detector information and aggregated in 15-minute intervals were compared with the corresponding counts made on the video over a period of the peak hour.
3.6.1 Volume variations (fluctuations)

The 5-minute and 15-minute volume data obtained from the traffic counters were plotted versus the corresponding time for the entire study length to present the volume fluctuations. The 15-minute volume was aggregated from the 5-minute data.

3.6.2 Peak 15-minute volume

According to the 1985 HCM, most capacity procedures deal with the rate of flow within the worst 15 minutes of the peak hour, so the highest 15-minute volume within the peak hour between 4:00 pm and 5:00 pm was tabled and the corresponding time was also recorded for every lane. The data was aggregated from 5-minute to 15-minute intervals. After that, the peak hour factor was calculated and tabled for all lanes. The PHF is the result of dividing the peak hourly volume by the peak 15-minute rate of flow. The denominator containing the maximum (peak) 15-minute must be multiplied by 4 because any hour consists of four 15-minute intervals.

3.6.3 Average headways

The average headway was determined for 1-minute intervals during the peak hour (4:00 pm-5:00 pm) and the peak 15-minutes within the peak hour. For the analysis of the headways within 1-minute intervals, four types of headways were observed:

P-P: passenger car following a passenger car
P-T: passenger car following a truck
T-P: truck following a passenger car
T-T: truck following a truck
The average headway of all vehicles was then calculated for each lane by using the following equation.

\[ H_a = P_T \times H_{aTT} + P_T \times (1 - P_T) \times H_{aTP} + (1 - P_T) \times P_T \times H_{aPT} + (1 - P_T)^2 \times H_{aPP} \]  \hspace{1cm} (3.1)

where; \( P_T \) = proportion of trucks in traffic stream

\((1 - P_T)\) = proportion of passenger cars in traffic stream

\( H_{aPP}, H_{aPT}, H_{aTP}, H_{aTT} \) = average headways for the indicated paired vehicle types (sec).

The comparison between the average 1-minute interval headway using the previous equation in the peak hour, peak 15-minutes within the peak hour and the average headway within a 1-minute interval in the peak hour without using the equation was conducted and tabled. The headway values for each 1-minute interval of the entire peak hour was determined by dividing the time of 60 seconds (one minute) over the number of vehicles of the different types in that particular interval.

### 3.6.4 Passenger car equivalents

The determination of passenger car equivalents for trucks and buses was performed using several approaches.

In the first approach, the passenger car equivalent (PCE) factor for the heavy vehicles and buses was determined using the results of field data to calibrate the heavy-vehicle factor. This determination was conducted for only lanes 1 and 2. In lanes 3 and 4, the percentage of trucks was 0.1% and zero respectively, so no PCE for heavy trucks appeared. The following equation was used to determine the passenger car equivalent for the heavy trucks using the field data:
\[ E_T = [(1-P_T) \times (H_{aPT} + H_{aTP} + H_{aPP}) + P_T \times H_{aTT}] / H_{aPP} \quad (3.2) \]

where; \( E_T \) = passenger car equivalent for trucks and buses.

The second approach made use of the Highway Design Manual developed for Saudi Arabia. The Highway Design Manual is one of a series of manuals covering highway design and construction, prepared in a cooperative effort by the Ministry of Communications of the Kingdom of Saudi Arabia and the Federal Highway Administration of the United States. It is the second of four volumes comprising the Highway Design Manual. The passenger car equivalents of trucks and/or buses on multi-lane highways are presented [DAR Al-Handasah 1974].

In the third approach, the Highway Capacity Manual (HCM 1985) represents the value of PCE for trucks/buses at specific grades based on typical U.S. data. Passenger car equivalents flow from the definition of the heavy vehicle factor and can be related to average headways in a traffic stream.

The passenger car equivalent values for heavy vehicles and/or buses determined from the three methods were calculated and tabled for all lanes. The passenger car equivalent for small buses was estimated to be 1.3.

3.6.5 Free-flow speed (FFS)

Free-flow speed has been defined as the speed that results when density and flow are “zero,” i.e., they intercept on the speed scale. In practical terms, however, the free-flow speed is a constant value that prevails over a significant portion of the speed-flow curves. Where no actual data is available, the free-flow speed can be estimated as:
where $S_F = \text{prevailing free-flow speed, kph,}$

$S_{fi} = \text{free-flow speed under ideal conditions, kph,}$

$f_c = \text{adjustment for lateral clearance,}$

$f_w = \text{adjustment for lane width,}$

$f_n = \text{adjustment for number of lanes, and}$

$f_id = \text{adjustment for interchange}$

The free-flow speed under ideal conditions ($S_{fi}$) is estimated from either the speed limit or the 85th percentile speed for the facility (if available) or similar facilities on each lane. In this case the 85th percentile speed was available from the cumulative speed distribution curve. Once the ideal free-flow speed is established, the previous adjustments are applied. According to the geometry of the selected site on the freeway, all of the adjustments are zero except for the number of lanes ($f_n$), which has a value of 2.4 kph (1.5 mph) because the number of lanes is four in one direction. This amount is deducted from the ideal free-flow speed ($S_{fi}$) to determine the prevailing free-flow speed ($S_F$), which is presented as the estimated free-flow speed. The estimated free-flow speeds in lanes 1, 2, 3, and 4 are 98.5 kph, 98.5 kph, 109.6 kph, and 116.3 kph, respectively. The second method for approximating the “ideal” free-flow speed is to relate it to existing speed limits. Ideal free-flow speed was taken to be 8 kph (5 mph) greater than the speed limit of 110 kph (68 mph) on the Jeddah-Makkah freeway, based on the HCM recommended procedure.
The free-flow speed can be measured on freeways as the average speed of traffic streams with flow rates lower than 1000 pcphpl. On freeways, average speeds observed at flow levels less than 1000 pcphpl are preferred to ensure that the speed is in Region 1 (see Figure 2.2) and therefore unaffected by flow. Such measurements reflect all of the prevailing conditions at the location under study and require no further adjustment. The measured value of the free-flow speed is determined under four different category times. The first period is the speed measured at low flow rates lower than 1000 pcphpl (between 100 pcphpl and 200 pcphpl in lanes 1 and 2, and between 400 pcphpl and 500 pcphpl in lanes 3 and 4). In addition, the free-flow speed is measured at high flow rates lower than 1000 pcphpl, that is, between 800 pcphpl and 900 pcphpl. The third approach is the measurement of the average speed corresponding to flow rates less than 1000 pcphpl during the daytime. This period was selected from 8:00 am to 6:00 pm. Finally, the same measurement was established for the same circumstances, but in the nighttime between 6:00 pm and 6:00 am. The main purpose of measuring the free-flow speed by day and night divisions is studying if darkness is an influence on flow and speed values. The appropriate value of the measured free-flow speed for use in the analysis procedures is taken as daytime. The sample size in the daytime category is the largest number compared to the other three measured criteria. Therefore, it is the best representative data to obtain the measured free-flow speed.
3.6.6 Speed distribution

The relationship between the percentage of the vehicles and the corresponding average speed was plotted for each and all lanes. The frequency values were the number of vehicles (various types) during the entire study length that traverse at a specific speed. These values were taken from the speed file generating from the traffic counters. After that, the cumulative speed distribution curve for all lanes was generated to determine the 85th percentile speed. The 85th percentile speed is defined as the speed at which 85% to 90% of the vehicles are traveling at or below. This value is used to define the free-flow speed as described above. Two plot sets were generated for both speed distribution and also for cumulative speed distribution curves. One plot represented speed distribution curve for each individual lane, so a total of four curves appeared together in the first plot. The other plot with one curve illustrated the speed distribution for the entire roadway.

3.6.7 Lane distribution

The percentages of vehicles by type (passenger car, small bus, big bus, and truck) existing in each lane were determined at both the peak hour period (4:00 pm-5:00 pm) and the entire study length of 22 hours. In addition, the average percentages of vehicle types were determined for all lanes combined.

3.6.8 Accuracy of traffic counters and video image

The detector signals were checked by comparing them with video observations at the site. Vehicle counts obtained from detector information in 5-minute intervals were compared with the corresponding counts made on the video over a period of 5
minutes after aggregation from the original 1-minute interval between the period of
6:15 pm and 6:20 pm.

3.6.9 Speed-flow-density relationships

The 15-minute intervals for the total length of the study (22 hours) were used
to calibrate the relationships of flow-density and speed-density curves for the entire
roadway. These relationships were generated in order to define level of service
criteria. Through these relationships, speed and flow are variables depending on the
density. The obtained data was merged for all lanes. The average value of speed per
lane was taken in every 15-minute interval. In addition, the summation of the flow
values for 15-minute intervals in each lane was taken to represent the data of the flow
in all lanes. The linear regression analysis was performed through Excel to develop a
mathematical model with the most appropriate fit to the obtained data.

Several mathematical forms were attempted to define mathematical
relationships that adequately describe the plotted data. The equation was calibrated to
minimize the differences between the observed data and the relationship calibrated.
The most important statistical value generated after performing the regression analysis
is coefficient of determination ($R^2$). The coefficient of determination is the proportion
of the observed variation in the dependent variable that has been explained by the set
of independent variables used in the equation. Coefficients of determination in the
range of 0.75 to 0.85 should be achievable with a reasonable mathematical model
(Mood and Grabell 1963).
A linear model was used to calibrate the flow-density relationship of the obtained data. A number of functions were evaluated, including linear, logarithmic, power, and exponential forms, to describe the speed-density relationship. The usage criteria to select the model form depend on the value of $R^2$ and the shape pattern, which simulates the ideal relationship between speed and density. An elliptical model fit the data better than any of the other alternatives tested as indicated by the $R^2$ value. Therefore, it was chosen for the calibration of the speed-density relationship.

According to the obtained data, the jam density could not be reached so its value was estimated at 40 pc/km/lane. In addition, the speed-flow relationship was plotted.

3.6.10 Developing level of service criteria

The HCM specifies that density is the most appropriate measure by which to define levels of service for freeways. The reason is that speed is seen to be constant through a large range of flows, and is therefore not a good measure of service quality. Flow is a point measure not discernible by drivers. This leaves density, which describes proximity to other vehicles, and which varies with flow throughout the full range of flow. Members of the Transportation Research Board Committee on Highway Capacity and Quality of Flow, whose mission is to promote innovation and progress in transportation by simulating and conducting research, make a collective judgment concerning the appropriate break points for the various levels of service (A through E) used to develop the level of service criteria. The corresponding values of speed and flow at the break points, which are the densities that define the boundaries
between one level of service and the next level of service, were determined through
the calibrated mathematical models of speed-density and flow-density relationships.

3.6.11 Driver population

Almost all the driver users on the freeway in this project are considered as
commuters. The users of the Jeddah-Makkah freeway are classified as residents and
nonresidents. The residents use the facility as drivers while the nonresidents ride the
buses.

A resident is a local worker in Jeddah, with a home city of Makkah. His use of
this freeway is a daily habit that can make him very familiar with the roadway. This
case is also true for the local people who use the Jeddah-Makkah freeway for vacation
trips on the weekends or scheduled jobs during the weekdays.

The nonresidents (visitors) are the people who come from outside the Saudi
Arabia by air, sea, and vehicles. The visitors coming by air and sea are organized to
reach Makkah by using the buses of the Omrah or Hajj agencies that offer this service
as a required item in their packages. The visitors make contact with these agencies in
order to visit Makkah at any time of the year. The agency is responsible for issuing the
visa and providing local transportation, which is usually by bus. The drivers of those
buses are either Saudi or non-Saudi persons who have had drivers training before
obtaining employment. None of them is a first time driver on the freeway. Finally, the
people who come from neighboring countries in their private vehicles for the same
purpose have used this freeway before, and are familiar with the correct directions to
Makkah. As a result, all the drivers of the freeway are familiar with this freeway and are considered as commuters.
CHAPTER 4  
Results and Discussion

Chapter 4 presents all the results of volume variations, free-flow speed, speed distributions, passenger car equivalent, lane distributions, and speed-flow-density relationships. In this chapter, the results of the Jeddah-Makkah Freeway Project are presented. Plots of volume fluctuations (5-minute and 15-minute), speed distributions, cumulative speed distributions, lane distributions, and speed-flow-density relationships for the entire roadway are provided. In addition, the results of the peak 15 minutes, peak hour factors, headways, passenger car equivalents for heavy trucks and buses, free-flow speeds, and percentages of vehicles by type in all lanes are included.

4.1 Volume variations

In Figure 4.1 the 15-minute interval volume of passenger cars is plotted versus the corresponding time of the total study length (22 hours). The 5-minute interval volume is presented in Figure 4.2. The 15-minute volume chart has less fluctuation and variation than the 5-minute. Therefore, the data was aggregated from 5-minute intervals to 15-minute intervals in order to use it for the calibration of speed-flow-density relationships. Throughout the 5-minute data, the maximum volume occurring in all lanes was in lane 4, with 274 and 272 passenger cars at 4:50 pm and 11:15 pm respectively. Furthermore, the maximum volume obtained during the 15-minute
interval occurred in lane 3 with 559 passenger cars at 4:45 pm. Both results occurred during the peak hour (4:00-5:00 pm).

Figure 4.1: Five-minute volume fluctuations in all lanes

Figure 4.2: Fifteen-minute volume fluctuations in all lanes
4.2 Peak hour factor (PHF)

The peak 15-minute period based on the entire data of 22 hours was determined for all lanes and is shown in Table 4.1 below. The period of the peak 15-minute is also shown in Table 4.1. This period is located in the third quarter of the peak hour time (4:00 pm-5:00 pm) between 4:35 pm and 4:56 pm. The peak hour factor was determined. Its corresponding values are 0.85, 0.81, 0.9, and 0.87 in lanes 1, 2, 3, and 4 respectively.

Table 4.1: The peak 15-minute and the peak hour factor in all lanes for the peak hour of 4:00-5:00 pm

<table>
<thead>
<tr>
<th>LANE</th>
<th>Peak Hour Volume (vph)</th>
<th>Peak 15-minute Volume (veh)</th>
<th>Peak Hour Factor (PHF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>548</td>
<td>162 4:36-4:51 pm</td>
<td>0.85</td>
</tr>
<tr>
<td>2</td>
<td>1141</td>
<td>351 4:41-4:56 pm</td>
<td>0.81</td>
</tr>
<tr>
<td>3</td>
<td>1948</td>
<td>543 4:35-4:50 pm</td>
<td>0.9</td>
</tr>
<tr>
<td>4</td>
<td>2808</td>
<td>811 4:36-4:51 pm</td>
<td>0.87</td>
</tr>
</tbody>
</table>

4.3 Average headway

The headway results in Table 4.2 are represented after the analysis of headways within 1-minute intervals for the peak hour. In addition, the headways obtained within 1-minute intervals for the peak 15-minute within the peak hour are shown. These values were calculated by using the equation (3.1) and were compared to the headway values obtained from the peak hour within 1-minute intervals. The
ordinary average was used without taking into consideration the headway types as in the equation 3.1 (the basic definition of headway). From the table below, it can be seen that there is a slight difference in the average headway during the peak hour by using the equation and taking the ordinary average. For instance, the difference in the average 1-minute headway in lane 4 between using the equation and taking the ordinary mean of the headway data was unnoticeable (0.04 sec).

Table 4.2: Comparison of average headway, $H_a$ (sec) weighted by vehicle type as determined by equation (3.1) and the average measured headway (unweighted)

<table>
<thead>
<tr>
<th>Equation Usage</th>
<th>Detection Period</th>
<th>LANE 1</th>
<th>LANE 2</th>
<th>LANE 3</th>
<th>LANE 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(3.1)</td>
<td>1-min (peak 15-min)</td>
<td>5.19</td>
<td>2.50</td>
<td>1.68</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>1-min (peak hour)</td>
<td>6.42</td>
<td>3.19</td>
<td>1.88</td>
<td>1.37</td>
</tr>
<tr>
<td>No</td>
<td>1-min (peak hour)</td>
<td>7.03</td>
<td>3.47</td>
<td>1.88</td>
<td>1.41</td>
</tr>
</tbody>
</table>

4.4 Errors for headways

The average measured vehicle headways for the four possible paired vehicle types (i.e., passenger car following a passenger car, $H_{p,p}$, etc.) in each lane are shown in Table 4.3. The standard deviation and the 95% confidence bound, which depends on the sample size and standard deviation, are also given for each headway category. The proportion of heavy trucks ($P_T$) is given for lanes 1 and 2 only, since essentially no trucks operate in lanes 3 and 4. The average headway ($H_a$), which was calculated
using equation (3.1), is also shown for each lane. The average headways for lane 1 and lane 2 were then used to determine the passenger car equivalents for trucks and buses as described in section 4.5.

Table 4.3: Headway results and the average headway ($H_a$) obtained from equation (3.1) for each lane

<table>
<thead>
<tr>
<th>LANE 1</th>
<th>Headway Type</th>
<th>Sample Size</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Error (95% confidence)</th>
<th>$P_T$</th>
<th>$H_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_p-p</td>
<td>164</td>
<td>4.95</td>
<td>4.39</td>
<td>0.672</td>
<td>1.35</td>
<td>0.37</td>
<td>6.42</td>
</tr>
<tr>
<td>H_p-t</td>
<td>104</td>
<td>7.19</td>
<td>5.89</td>
<td>1.13</td>
<td>1.57</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H_t-p</td>
<td>100</td>
<td>7.74</td>
<td>6.88</td>
<td>1.35</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H_t-t</td>
<td>84</td>
<td>7.12</td>
<td>7.35</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LANE 2</th>
<th>Headway Type</th>
<th>Sample Size</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Error (95% confidence)</th>
<th>$P_T$</th>
<th>$H_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_p-p</td>
<td>857</td>
<td>3.17</td>
<td>3.82</td>
<td>0.256</td>
<td>1.14</td>
<td>0.04</td>
<td>3.19</td>
</tr>
<tr>
<td>H_p-t</td>
<td>35</td>
<td>3.63</td>
<td>3.45</td>
<td>1.14</td>
<td>0.92</td>
<td></td>
<td></td>
</tr>
<tr>
<td>H_t-p</td>
<td>100</td>
<td>3.32</td>
<td>2.54</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H_t-t</td>
<td>5</td>
<td>2.07</td>
<td>1.05</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LANE 3</th>
<th>Headway Type</th>
<th>Sample Size</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Error (95% confidence)</th>
<th>$P_T$</th>
<th>$H_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_p-p</td>
<td>1761</td>
<td>1.88</td>
<td>1.96</td>
<td>0.091</td>
<td>0.38</td>
<td>0.001</td>
<td>1.88</td>
</tr>
<tr>
<td>H_p-t</td>
<td>1</td>
<td>0.46</td>
<td>0.28</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H_t-p</td>
<td>2</td>
<td>0.81</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H_t-t</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>LANE 4</th>
<th>Headway Type</th>
<th>Sample Size</th>
<th>Mean</th>
<th>Standard deviation</th>
<th>Error (95% confidence)</th>
<th>$P_T$</th>
<th>$H_a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H_p-p</td>
<td>2558</td>
<td>1.37</td>
<td>1.69</td>
<td>0.066</td>
<td>0</td>
<td>0</td>
<td>1.37</td>
</tr>
<tr>
<td>H_p-t</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H_t-p</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>H_t-t</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.5 Passenger Car Equivalent (PCE) for heavy vehicles and/or buses

Table 4.4 shows the passenger car equivalent values for heavy vehicles and/or buses based on the Highway Design Manual, Highway Capacity Manual, and use of field data (equation 3.2) to calibrate the heavy vehicle factor in lanes 1 and 2. These values are based on the total length of the study. Since the percent of trucks in lanes 3 and 4 was almost zero, no attempt was made to calculate the PCE for heavy vehicles in these lanes. Since the percentage of heavy trucks in lane 1 was more than that of lane 2, the passenger car equivalent factor for heavy trucks and buses in that lane was higher than that of lane 2.

Table 4.4: Passenger Car Equivalent for heavy vehicles and or buses in lanes 1 and 2

<table>
<thead>
<tr>
<th>Method</th>
<th>LANE 1</th>
<th>LANE 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highway Design Manual</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Highway Capacity Manual</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Field Data (equation 3.2)</td>
<td>1.8</td>
<td>1.17</td>
</tr>
</tbody>
</table>

4.6 Free-flow Speed (FFS)

The free-flow speed is determined in all lanes with different criteria. Table 4.5(a) summarizes the results of the free-flow speed in lanes 1 and 2 with various situations while Table 4.5(b) shows the similar results in lanes 3 and 4. The daytime free-flow speed is representative for all lanes. The corresponding values are 91.7 kph, 92.4 kph, 111.5 kph, and 115.7 kph in lanes 1, 2, 3, and 4 respectively. In addition, the free-flow speed estimation by using the speed limit approach was determined as 118
kph. It is the highest value from both measured and estimated approaches. The average estimated free-flow speed in all lanes was 105.7 kph.

Table 4.5(a): Free-flow speed values at different criteria in lanes 1 and 2

<table>
<thead>
<tr>
<th>Criteria</th>
<th>LANE 1</th>
<th>LANE 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flow n (pcph)</td>
<td>Speed (kph)</td>
</tr>
<tr>
<td>Low Flow rate (200-300 pcph)</td>
<td>16</td>
<td>247</td>
</tr>
<tr>
<td>High Flow rate (800-900 pcph)</td>
<td>3</td>
<td>841</td>
</tr>
<tr>
<td>Day (8 am-6 pm)</td>
<td>40</td>
<td>418</td>
</tr>
<tr>
<td>Night (6 pm-6 am)</td>
<td>48</td>
<td>384</td>
</tr>
<tr>
<td>Estimated Free Flow Speed (kph)</td>
<td>98.5</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.5(b): Free-flow speed values at different criteria in lanes 3 and 4

<table>
<thead>
<tr>
<th>Criteria</th>
<th>LANE 3</th>
<th>LANE 4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flow n (pcph)</td>
<td>Speed (kph)</td>
</tr>
<tr>
<td>Low Flow rate (400-500 pcph)</td>
<td>3</td>
<td>449</td>
</tr>
<tr>
<td>High Flow rate (800-900 pcph)</td>
<td>6</td>
<td>859</td>
</tr>
<tr>
<td>Day (8 am-6 pm)</td>
<td>23</td>
<td>645</td>
</tr>
<tr>
<td>Night (6 pm-6 am)</td>
<td>13</td>
<td>841</td>
</tr>
<tr>
<td>Estimated Free Flow Speed (kph)</td>
<td>109.6</td>
<td></td>
</tr>
</tbody>
</table>
4.7 **Speed and volume distribution**

The speed distribution curve shows the percentage of the vehicles passing at a particular speed. The curve shows the relationship of the percentage of vehicles versus the speed. Figure 4.3 shows the speed distribution in each lane. Thirty percent and 27% of the vehicles in lanes 1 and 2 respectively were passing the section at 85 kph. The highest percentages of vehicles in lanes 3 and 4 tended to move at 102.5 kph and 117 kph respectively.

![Speed distribution curve by percentage in each lane](image)

**Figure 4.3:** Speed distribution curve by percentage in each lane

In addition, the cumulative speed distribution curve was plotted for each lane in order to get the 85th percentile speed. Figure 4.4 represents the cumulative speed distribution curve in each lane. The 85th percentile speed values obtained from Figure 4.4 were 101.5 kph, 101.5 kph, 112.5 kph, and 120 kph in lanes 1 through 4 respectively. The difference in speeds for each between the first two lanes and lane 4
is almost 20 kph. References cited in the literature review were consulted to judge the significance of this speed range compared to typical U.S. highways. However, only one study (Chin and May 1990) provided speed data by lane. For that research the average speed per lane for high flow rates on an 8-lane freeway varied by only 1-3 mph (2-5 kph).

![Cumulative speed distribution curve in each lane](image)

Figure 4.4: Cumulative speed distribution curve in each lane

The curves of speed distribution and cumulative for all lanes combined are shown in Figures 4.5 and 4.6 respectively. Figure 4.5 shows two different peak values of the frequency of the vehicles; they were at speeds of 85 kph and 107.5 kph. In fact,
the first portion of the curve before the drop shows the pattern of the speed distribution curve in lanes 1 and 2 combined, whereas the second curve after the drop shows this behavior in lanes 3 and 4 as an average.

Figure 4.5: Speed distribution curve for the entire roadway

Finally, the cumulative speed distribution curve for all lanes combined was generated and is shown in Figure 4.6. The 85th percentile speed in all lanes is 115 kph,
meaning that 85 percent of all the vehicles move at this speed, which is over the posted speed limit of 110 kph.

Figure 4.6: Cumulative speed distribution curve for the entire roadway

Figures 4.7 through 4.10 shows the volume distribution by percentage of the different vehicle types (passenger car, small bus, big bus, and truck) in lanes 1, 2, 3, and 4 respectively. These figures represent the comparison of the percentage between the peak hour period (4:00 pm-5:00 pm) and the total study length (8:00 am-6:00 am). In addition, the average of the percentages for each type in all lanes was determined. Table 4.6 shows the average percentage of vehicles by type for the entire roadway in both the peak period and the total study length. Figure 4.11 presents the results which appeared in Table 4.6.
Figure 4.7: Volume distribution in lane 1

Figure 4.8: Volume distribution in lane 2
Figure 4.9: Volume distribution in lane 3

Figure 4.10: Volume distribution in lane 4
Table 4.6: Percentage of vehicles by type in all lanes

<table>
<thead>
<tr>
<th>Vehicle Type</th>
<th>Peak Hour (%)</th>
<th>Entire Study Length (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passenger Car</td>
<td>92.6</td>
<td>78.1</td>
</tr>
<tr>
<td>Small Bus</td>
<td>3.4</td>
<td>11.5</td>
</tr>
<tr>
<td>Big Bus</td>
<td>3.2</td>
<td>3.2</td>
</tr>
<tr>
<td>Truck</td>
<td>0.8</td>
<td>7.2</td>
</tr>
</tbody>
</table>

Figure 4.1: Volume distribution in all lanes combined

The trucks are more prevalent in the off-peak period than in the congested hour (4:00 pm-5:00 pm). This fact reflects the very low percentage of trucks in the peak hour compared to the whole study length. The same is true for small bus users. The percentage of the big buses in both periods is the same. On the other hand, the
occurrence of passenger cars is very high within the peak hour but the overall percentage is reduced during the whole period of study. In both periods, passenger cars comprise more than 75% of the traffic fleet as an average in all lanes.

4.8 Accuracy of traffic counters and video recording methods

The difference in 5-minute volume counts from the obtained data from both traffic counters and video-recording methods is represented in Figure 4.12. Lane 1 was selected between the period of 6:15 pm and 6:20 pm. The data obtained from the video recording was aggregated from 1-minute intervals to 5-minute intervals in order to perform the comparison. The total number of vehicles obtained from the traffic counter data in the assigned period was 41 vehicles while this number was 42 vehicles through the video observation. Figure 4.12 also shows the number of vehicle types obtained by both types. No miscounts or additional counts took place in measuring the volume of both trucks and buses in the selected period (6:15-6:20 pm). The generated error of the volume counts of the passenger cars and small buses was 5% and 6% respectively; therefore the accuracy of both traffic counter and video recorder detection methods in measuring volume is almost 95%. Though the sample size is small, the generated error behaves like random error because it results in both over-counting (additional) and under-counting (miscount). Therefore, the mean error might be closer to zero.

In addition, the corresponding speed values were 98.7 kph and 97.9 kph, taken from the video recording and traffic counters respectively at the same time chosen in Figure 4.12. The difference in speed detection from both methods was 0.8%.
4.9 Speed-flow-density relationships

The total of 88 data points were obtained from the 15-minute intervals. Eleven points from the whole data were considered outliers. The outliers are usually the points out of data range. They occurred at various times throughout the day but there were none during the peak hour.

Figure 4.13 shows results of the regression analysis for the flow-density relationship based on the obtained data points. The generated equation of the linear mathematical model is:

\[
\text{Flow} = 99.343 \times \text{Density} + 15.486
\] (4.1)

Equation (4.1) shows that if the density is zero the flow will be very light (16 pcphpl). The coefficient of determination value ($R^2$) is 0.983. The model is deemed statistically reliable because the significance level is very close to zero with a very small value of $2.6\times10^{-68}$. The jam density ($D_j$) could not be reached because the situation of congested flow did not occur while collecting the data.
The speed-density relationship is shown in Figure 4.14. A linear model was selected to calibrate speed-density relationship because it fits the data with the highest $R^2$ obtained of 0.934 and followed the ideal pattern form of similar relationships. The data was also statistically significant and the obtained value was $5.3 \times 10^{-46}$. The calculated mathematical equation of the linear model is:

$$\text{Speed} = -0.5721 \times \text{Density} + 107.8$$  \hspace{1cm} (4.2)

According to equation (4.2), the free-flow speed is equal to 107.8 kph. The average measured free-flow speed was 105.7 kph so the difference of both values is small, which is another evidence that the selective model is the best fit of the scatter plot of the data.
The speed-flow relationship was plotted and is shown in Figure 4.15. Several mathematical models were attempted to achieve the highest value of R^2. However, the regression analysis of the speed-flow relationship was not reliable statistically because the highest R^2 obtained was 0.0224. The generated equation of the linear form of the speed-flow relationship is:

\[
\text{Speed} = -0.0027 \times \text{Flow} + 104.08
\]

Not only is the curve unreliable but it has limited use since measured flows were not high enough to provide data near the end of Region 2 (see Figure 2.2). Further, there is no data to plot Regions 3 and 4. The lack of data is a direct result of the fact that this highway is not congested even during the peak hour flow for the year.
4.10 Developing level of service criteria

The level of service criteria on the Jeddah-Makkah freeway was developed. Table 4.7 shows the HCM density boundary marks used to define levels of service A through E. The corresponding HCM speeds and flow rates are also presented in Table 4.7 for comparison with the results of the Jeddah-Makkah freeway. The values of maximum service flows at levels of service A through C in this research were smaller than that in the HCM while they were higher than the HCM at levels of service D and E. The corresponding minimum speed values at all break points (boundary marks)
between levels of service were less than the HCM. The obtained results in Table 4.7 of both minimum speed and maximum service flow were obtained from the speed-density and flow-density relationships (Figure 4.13 and Figure 4.14) respectively by entering the curves with the fixed values of the maximum density (breakpoints) and getting the corresponding values of flow and speed respectively. The difference in minimum speeds at levels of service A and B between the two criteria was 0.1 mph. The corresponding values obtained from the results of the maximum flow at the same levels were also less than that of the HCM. Level of service C has the same pattern as levels of service A and B. Levels of service D and E, which represent congested flow, had flows greater than that of the HCM even though the corresponding values of speeds were less than the HCM criteria. This means at high flow rates the speeds were reduced compared to the HCM standards. The average estimated free-flow speed is 105.7 kph (66 mph). It was used as 55 mph in Table 4.7 to perform the comparison.

Table 4.7: Level of service criteria developed from Jeddah-Makkah freeway data compared to HCM (1985) level of service criteria

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>Maximum Density (pc/mi/ln)</th>
<th>HCM</th>
<th>Jeddah-Makkah freeway</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum Speed (mph)</td>
<td>Maximum Service Flow (pcphpl)</td>
<td>Minimum Speed (mph)</td>
</tr>
<tr>
<td>A</td>
<td>10</td>
<td>65</td>
<td>650</td>
</tr>
<tr>
<td>B</td>
<td>16</td>
<td>65</td>
<td>1040</td>
</tr>
<tr>
<td>C</td>
<td>24</td>
<td>64.5</td>
<td>1548</td>
</tr>
<tr>
<td>D</td>
<td>32</td>
<td>62</td>
<td>1984</td>
</tr>
<tr>
<td>E</td>
<td>45</td>
<td>52</td>
<td>2350</td>
</tr>
<tr>
<td>F</td>
<td>var</td>
<td>var</td>
<td>var</td>
</tr>
</tbody>
</table>
CHAPTER 5
Conclusions and Recommendations

5.1 Conclusions

The peak hour for the year 2001 occurred at the same time of the previous data (2000). The Jeddah-Makkah freeway did not reach capacity during this study and therefore does not reach capacity anytime of the year. Headways decreased as flow increased. Headways were significantly smaller between passengers cars in lanes with no trucks (Lanes 3 and 4) compared to lanes with trucks (Lanes 1 and 2). Passenger car equivalents for heavy trucks based on actual data were 60 to 90% smaller than PCE’s based on Highway Design Manual (HDM) or HCM recommendations. Both average values of the measured and estimated free-flow speeds were 7% and 4% smaller than the speed limit. In addition, the average estimated free flow speed was 8% smaller than the 85th percentile speed. Based on comparison with limited U.S. data it appears that the speed difference between lanes is greater for the Jeddah-Makkah freeway. The criteria for level of service, which were obtained from the results of this research, show a slight difference from that mentioned in the Highway Capacity Manual (HCM). The minimum speed values of the results were less than the HCM criteria for levels of service A through F. From the results of the speed distribution, the vehicles in both lanes 1 and 2 were more restricted with the speed limit compared to lanes 3 and 4, which is due to the existence of the trucks which make the movement slow and the fact that the speed occurs in a limited range (75 kph-95 kph). Vehicles in
lanes 3 and 4 that represent the fast lanes attempt to move at a wider range of speeds, causing larger standard deviation in speeds. This is due to the aggressive behavior of the drivers who try to move as fast as they can and pass from either the left or right of the front vehicles (even though regulations prohibit passing from the right). The flow-density curve of this research was developed in a linear form. Always, when the density increases, the corresponding flow increases for all the data points. However, U.S. research assumes that the flow increases if the density increases until the density reaches the critical density (when the capacity occurs). After that, the relationship of the flow-density will take the opposite pattern (the flow decreases when the density increases). Similar speed-density relationships were achieved in this research project. The speed-flow curve generated from the data indicated that speeds and flows were not well correlated for the limited range of acquired data.

5.2 Recommendations

There are several recommendations from this project that should be taken into account in the future in order to achieve detailed traffic characteristics on the Jeddah-Makkah freeway. The three major recommendations are as follows:

1. Develop mathematical models in each lane to identify the differences of the speed-flow-density relationships for individual lanes.

2. Use the video recording method in freeways to detect speeds and volumes rather than the traffic counters due to the ease of installation and prevention of stopping the movement of traffic during the installation process.

3. Study driver behaviors, which influence the traffic characteristics.


APPENDIX A

List of equipment used

1. HI-STAR traffic counter, model number (NC-90A)

2. Panasonic video camera (VHS), model number NV-M3000 EM

3. Panasonic video camera (8 mm), model number VX33 with digital zooms of X250 and optical zoom of X21

4. Video vision machine software: Ulead Media Studio Pro 6, Video Editor 6 (2)
APPENDIX B

Speed-density relationship

Figures B.1 through B.3 show the mathematical models of the speed-density relationship for logarithmic, exponential, and power forms with the corresponding $R^2$. In addition, the arithmetic equation is provided within each plot; the jam density was assumed to be 40 pc/km/lane.

![Graph showing speed-density relationship for logarithmic form. The equation is $y = -3.6144\ln(x) + 110.45$ with $R^2 = 0.76$.]

Figure B.1: Logarithmic form
Figure B.2: Exponential form

Figure B.3: Power form