EVALUATION OF APPROACH LEG CAPACITIES AT A SIGNALIZED URBAN ROUNDABOUT

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

EGE CEM SALTIK

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER SCIENCE
IN
CIVIL ENGINEERING DEPARTMENT

JANUARY 2018

Approval of the Thesis:

EVALUATION OF APPROACH LEG CAPACITIES AT A SIGNALIZED URBAN ROUNDABOUT

Submitted by **EGE CEM SALTIK** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University, by**

Prof. Dr. Gülbin Dural Ünver _________________
Dean, Graduate School of **Natural and Applied Sciences**

Prof. Dr. İsmail Özgür Yaman _________________
Head of Department, **Civil Engineering Department, METU**

Assoc. Prof. Dr. Hediye Tüydeş Yaman _________________
Supervisor, **Civil Engineering Department, METU**

Examining Committee Members:

Prof. Dr. Murat Güler _________________
Civil Engineering Department, METU

Assoc. Prof. Dr. Hediye Tüydeş Yaman _________________
Civil Engineering Department, METU

Asst. Prof. Dr. Hikmet Bayırtepe _________________
Civil Engineering Department, Gazi University

Asst. Prof. Dr. Hatice Meriç Gökdalay _________________
Aviation Management Department, UTAA

Asst. Prof. Dr. Hande Işık Öztürk _________________
Civil Engineering Department, METU

Date: 25.01.2018

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last Name: Ege Cem Saltık

Signature:

ABSTRACT

EVALUATION OF APPROACH LEG CAPACITIES AT A SIGNALIZED URBAN ROUNDABOUT

Saltık, Ege Cem

M.S., Department of Civil Engineering
Supervisor: Assoc. Prof. Dr. Hediye Tüydeş Yaman

January 2018, 128 pages

Intersections are vital components of road networks, which provide ability to change directions serving many conflicting movements. When the traffic volumes are high, intersection control is necessary via either signalization or by design such as roundabouts. However, the even latter is mostly signalized in Turkey due to improper design aspects and driver behavior issues. When signalized, approach capacity at an intersection/roundabout is governed by the saturation flow rate, S, as well as the allocated effective green time, g. For the determination of S, the literature has some formulae using design characteristics or site measurements, which are based on ideal and simple conditions, whereas on-site situations can be very complex and different. This study focused on evaluation of nature of traffic, more specifically saturation flow and inefficiency during green time on the approach legs of a signalized roundabout. The numerical results were obtained from digitization of time headways, h, from fisheye camera recordings at a roundabout connecting a major and a minor arterial in Konya during morning (AM) and evening (PM) peak periods (including 115 cycles) as well as noon off-peak period (48 cycles). Headways are indexed for each lane, l, on each approach leg a during

each cycle i separately as $h_i^{a,l}$, which enabled estimation of corresponding saturation flow rates of $S_i^{a,l}$ based on on Highway Capacity Manual (HCM) and German Highway Capacity Manual (*HBS* in German) methodologies. Analysis $S_i^{a,l}$ values from 163 cycles were used to develop a practical saturation flow rate, $S_{85}^{a,l}$, which was later used in the development of inefficiency because of the fluctuations in the flow rates. During the green time on a lane of an approach $\phi_i^{a,l}$,3-moving average method was used to determine instantaneous flow rates, $\tilde{Q}$.

The statistical analysis of the 19057 time headway data for 19057 vehicles passing through study intersection showed a lognormal distribution with an interquartile range (IQR) of [1.6s;3s] on major approaches during the peak periods, while IQR on minor legs were [2s;4s]. Time headways were more scattered during the off-peak noon with IQR values of [2s;3s] and [2s;5s] for major and minor approaches, respectively. Regression of headways with vehicular or flow characteristics (vehicle type, cycle length, green time, etc.) did not produce any statistically significant factor effect. Saturation flows based on design characteristics using HCM and HBS methods suggested values in the range of 1900 – 2000 veh/h/ln for different lanes in the major approaches. Estimation of HCM site-based observation method produced higher values reaching up to 2631 veh/h/ln. The practical S values, S_{85}, were found within the range of 2400 veh/h/ln, which were reached mostly during the peak hour cycles, but also, some of the cycles in the noon, as well. 3-moving average flow rates showed values as high as 2900 veh/h/ln and as low as 600 veh/h/ln, even sometimes in the same cycle. Lane 2 and Lane 3 (the outermost lane) on all approaches generally experienced higher flows regardless of the time or assumed methodology. Average inefficiency during green periods was found as 30 percent even on the major approaches, which were due to traffic break in the upstream flow conditions.

Keywords: Time Headway, Saturation Flow Rate, Approach Leg Capacity, Moving Average, Inefficiency in Signalized Intersections

ÖZ

KENT İÇİ SİNYALİZE DÖNEL KAVŞAK KOLLARINDA KAPASİTE DEĞERLENDİRİLMESİ

Saltık, Ege Cem

Yüksek Lisans, İnşaat Mühendisliği
Tez Yöneticisi: Doç. Dr. Hediye Tüydeş Yaman

Ocak 2018, 128 sayfa

Kavşaklar, yön değiştirme sebebiyle çatışan hareketlere hizmet veren, ulaşım ağının önemli bileşenleridir. Trafik hacimleri yüksek olduğu zaman, kavşak kontrolleri sinyalizasyon veya dönel kavşaklar gibi tasarım elemanları ile sağlanmaktadır; ancak tasarım yönünden uygunsuz uygulamalar ve sürücü davranışları sebebiyle Türkiye'deki çoğu dönel kavşakta trafik sinyalizasyonu uygulanmaktadır. Sinyalize olduğunda, kavşak / dönel kavşak yaklaşımın kapasitesi, efektif yeşil süresi, g, boyunca belirlenen doygun akım oranı , S,ile belirlenmektedir. Bu çalışma, sinyalize dönel kavşak yaklaşım kolundaki yeşil süresi boyunca, özellikle trafiğin doygun akımı ve verimsizliğinin değerlendirmesine odaklanmıştır. Sayısal sonuçlar Konya'da ana ve tali yolların kesişmesi ile oluşan dönel kavşaktaki balıkgözü kamera kaydı ile sabah ve akşam zirve saatleri (115 sinyal devre süresini içermektedir) ve aynı zamanda öğlen zirve dönemindeki (48 sinyal devre süresini içermektedir) zamana bağlı takip aralığı, h_i, değeriyle elde edilmiştir. Takip aralığı her bir devre süresi boyunca, i, her bir yaklaşım kolu a ve her şerit için l ile gösterilmiştir, bu Highway Capacity Manual (HCM) ve German Highway Capacity Manual (Almanya'da HBS) metotlarına dayanan doygun akım oranını $S_i^{a,l}$ tahmin

etmeyi mümkün kılmaktadır. 163 devre süresinden elde edilen $S_i^{a,l}$ değerleri pratik bir doygun akım oranı $S_{85}^{a,l}$ belirlemek için kullanılmıştır, daha sonra bu değer doygun akımdaki oranındaki dalgalanma yüzünden, verimsizlik değerlendirmelerinde de kullanılmıştır. Efektif yeşil süresi boyunca yaklaşımdaki bir şeritte verimsizlik, $\phi_i^{a,l}$, anlık akım oranını, $\widetilde{Q}$, belirlemek için 3'lü hareketli ortalama yöntemi kullanılmıştır.

Çalışılan alandan geçen 19057 araç için zamana bağlı 19057 istatistiksel analiz çeyrekler açıklığı (IQR) zirve dönemlerinde ana yaklaşımlarda [1.6s;3s], tali yaklaşımlarda ise [2s;4s] log-normal bir dağılım göstermiştir. Öğle saatlerinde ana ve tali kollarındaki yaklaşımlardaki zamana bağlı takip aralığı, çeyrekler açıklığı [2s;3s] ve [2s;5s] sırasıyla, daha dağınıktır. Takip mesafesinin taşıt ve trafik özelliklerine bağlı (araç tipi, sinyal devre süresi, yeşil süresi, vb.) regresyon analizi istatistiki olarak önemli bir değer göstermemiştir. Kapasite değerlendirmelerinde, ana yaklaşım kollarında şerit bazlı 1900-200 araç/saat/şerit değerleri öneren, HCM ve HBS yöntemleri kullanılarak tasarım durumuna bağlı doygun akım hesaplanmıştır. HCM saha gözlemlerine dayanan yöntemi 2631 araç/saat/şerit değerine kadar çıkan daha yüksek değeler üretmiştir. Zirve saatlerindeki sinyal devre süreleri hem de öğle zamanı çoğu devre süreleri boyunca, pratik S değerleri , S_{85}, 2400 araç/saat/şerit aralığında bulunmuştur. 3'lü hareketli ortalama akım oranları en yüksek 2900 araç/saat/şerit ve en düşük 600 araç/saat/şerit, bazen aynı devre sürelerinde de göstermiştir. Gözlem zaman ve yöntem gözetmeksizin, en yüksek akım Şerit 2 ve Şerit 3 (dış şerit) gözlemlenmiştir. Ana yaklaşım kollarda bile yeşil süresi boyunca ortalama verimsizlik değerleri trafik akışı sırasındaki kopmalardan dolayı yüzde 30 olarak bulunmuştur.

Anahtar Kelimeler: Zamana Bağlı Takip Aralığı, Doygun Akım Oranı, Yaklaşım Kolu Kapasitesi, Hareketli Ortalama, Sinyalize Kavşaklarda Verimsizlik

To my beloved family

ACKNOWLEDGEMENTS

My sincere thank goes to my remarkable supervisor Assoc. Prof. Dr. Hediye Tüydeş Yaman for her continuous encouragement to begin my graduate study. I am indebted to her for leading me to the field of research and shaping my academic vision and background with her endless academic experience. Her guidance helped me in all the time of my research.

I would like to tell my sincere gratitude to ISSD Inc. for their support to use their systems during study and video recording data. I would like to thank to Oruç Altıntaşı for his guidance, support throughout the whole time of this thesis study.

I express my sincere regards to my workmates from Yüksel Proje A.Ş.

I am grateful to all my friends at Transportation Research Center, Ali Asghar Kazemi Afshar, Beyhan İpekyuz and Funda Türe Kibar and Pınar Karataş Sevinen. I would like to give special thanks to Gülçin Dalkıç and Berhan Melek, who have been patient and hospitable. I appreciate my friends Ali Yaşar and İlim Karaçal for endless support, helps and guidance.

I would like also to express my deepest appreciation to my family, my mother Birsen Saltık for her endless support, patient and encouragement, my father Salman Saltık for his confidence and contribution to my education life, my sister Esin Bircan Saltık for support through thesis period.

TABLE OF CONTENTS

LIST OF FIGURES

FIGURES

LIST OF TABLES

TABLES

CHAPTER 1

INTRODUCTION

An intersection is an area, shared by two or more roads, which has a main function of providing for the change of the route directions (Garber & Hoel, 2009). It composes of different direction of roads; each of which serves entrance or exit of the traffic. Intersections provide easy access, safety and comfort movement of vehicles and pedestrians, which further brings continuity of the traffic. Roads connecting to the intersection area are called approach legs.

When the traffic volumes are high, intersection control is necessary via either signalization or by design such as roundabouts. Traffic signalization is an effective way to alert driver about traffic regulations. It is a commonly accepted and implemented a way of control; although it has a significant cost attached. As an alternative but cheaper-in-operation solution is the use of roundabouts to manage high traffic volumes with traffic signs and road markings while posing fewer conflict points.

A major principle in the success of roundabouts is the priority of circulating flows over the entering ones at the approach legs, which have to yield and wait for an adequate gap. Secondly, the radius of the roundabout is crucial to store and serve traffic flows adequately. Last, but not the least, issue is the traffic culture. While there are almost universal limits for the design of a roundabout, in urban applications in Turkey, such standards are not often followed resulting in capacity loss of a roundabout serving the flows. Secondly, traffic enforcement and education vary from country to country, the success of roundabout in controlling traffic flows also vary. Thus, in some of countries such as Turkey, some of Asian countries, traffic at roundabout is management with full signalization at not only the approach leg but also in the circulating area (Yang, Li, & Xue, 2004). In such cases, the signalized roundabout starts acting closer to signalized intersections, for which approach capacity concepts can be borrowed from the latter.

1.1 Scope of the Study

This study mainly aims to evaluate of approach leg capacities at a signalized urban roundabout. Signalized intersection/roundabout legs capacities are mostly governed by the saturation flow and the green time it has allocated, as well as the exit capacities defined by Federal Highway U.S. Department of Transportation Federal Highway Administration. (2000). While it is possible to obtain the latter much easily, it is more challenging the estimate the former, thus, constitutes the focus of this study. The literature includes simple and commonly used formulae for the estimation of saturation flow rates. They rely mostly on design and traffic flow characteristics (such as number of lanes, traffic composition, heavy vehicle percentages, etc.); they make an assumption about a base saturation flow value and uses correction/adjustment factors, as in the cases of on Highway Capacity Manual (HCM) and German Highway Capacity Manual (*Handbuch fuer die Bemessung von Strassenverkehrsanlagen*, *HBS,* in German) methodologies. However, for site-based observations, different approaches were proposed, which can produce estimation results differentiating greatly from simplistic evaluation of literature.

This study focused on determination of approach leg capacities, for both major and minor ones, at a signalized urban roundabout based on not only the design characteristics but also site investigations, using full fisheye camera video data at an intersection, which was controlled dynamically. The analyses were repeated for morning (AM) and evening (PM) peaks, as well the noon off-peak, to see the variations within the day. The site-observation based saturation flow estimations were performed based on HCM approach, S, but also were compared against a new interpretation of flow conditions during cycle using moving average technique to estimate instantaneous saturation flow levels, $\tilde{Q}$. The latter also analyzed to test the level of fluctuations during green period against the simplistic assumption of constant saturation flow in the literature. Finally, choosing the 85 percentile values of instantaneous flow levels, S_{85}, as a practical threshold limit, inefficiency in a lane, ϕ_i, is defined as the ratio of flow not served in the duration $\tilde{Q}$ stays below the practical threshold, S_{85}.

As a case study, fisheye camera-based video data at a roundabout in Konya (which was taken from the dynamic traffic signal control unit installed) was digitized to obtain vehicle time headways, h_i, for each lane on each leg from fisheye camera recordings of a signalized urban roundabout in Konya. The video data was digitized for approximately 90-minute intervals during the Data digitization was performed manually for each approach leg as well as each lane separately. Statistical characteristics of time headways were firstly considered to draw a conclusion about traffic flow conditions.

One limitation of the study was the lack of real-time signal cycle times and green periods during the video recording, which were estimated from the digitized video data instead. As a result, the effective green or time loss during start or end of green times were not determined directly, but "used green times" were estimated from start and end-of vehicle moves in all lanes jointly as a surrogate measure. Analysis of time headways by vehicle type did not suggest any statistically significant passenger car equivalency (pce) rates for minibuses or buses; as an alternative, literature values were assumed for pce conversions; thus, some of the analyses were repeated in units of veh/h/ln as well as pce/h/ln to keep a better judgment on the matter.

1.2 Layout of the Thesis

The layout of this thesis is organized as follows: In Chapter 2, the literature on intersection, signalized intersection and roundabout, saturation flow calculation methods depending on some resources and statistical behavior of the site based observations, collection of the data from site and determination of saturation flow is summarized. Chapter 3 presents methodology of the study. Results of headway and saturation flow studies at each approach legs are given at Chapter 4. Finally, Chapter 5 includes a conclusion, recommendation, and a future research part.

CHAPTER 2

LITERATURE REVIEW

2.1 Intersection Control Guideline

While determining types of intersection, different types of components are taking into account. Balance between these factors should be provided to obtain optimum performance of the intersection. Safety is an important component of the intersection design. Not only vehicles but also pedestrians are utilizing intersection. Before taking a decision, intersection area should be observed closely. Basic safety requirements such as expected number of accidents or injuries of intersections should be determined. While making these, previous site based observation results should be dealt. Which type of accidents was occurred, how many of them have fatal results are taken into account. These affect the design of the intersection. Also, both security level at the intersection and different intersection capacities depends on speed limit of approach legs. Location of the intersection like on highway, urban arterial or suburban regions affect the speed of the approach legs. Moreover, capacity of the intersection is another factor to guide type of intersection. This depends on the approach legs capacity.

Capacity of the approach leg leads the definition about the intersection. In this respect, Institution of Highways and Transportation (IHT) prepare Figure 2.1 below. According to figure, major and minor approach leg flows determine the type of intersection. If the major approach leg capacity is much higher than the minor approach, there is no need to do about intersection. It is shown as the priority regimes at Figure 2.1. If the major and minor approach legs have some flow capacity, conflicted points could be seen in the traffic. To regulate the conflicted area, roundabout and signalized intersections could be implemented as stated green part in Figure 2.1. Also, beltways, highways have regular high density flow capacity and speed in these areas is high. In this respect, turning activities or connection of the minor roads to these could not be done at grade level. These conditions require grade separated intersection design requirements. There is no need either for junctions or roundabouts to be on motorways nor for signalized to be on rural roads as stated at European Commission (2017).

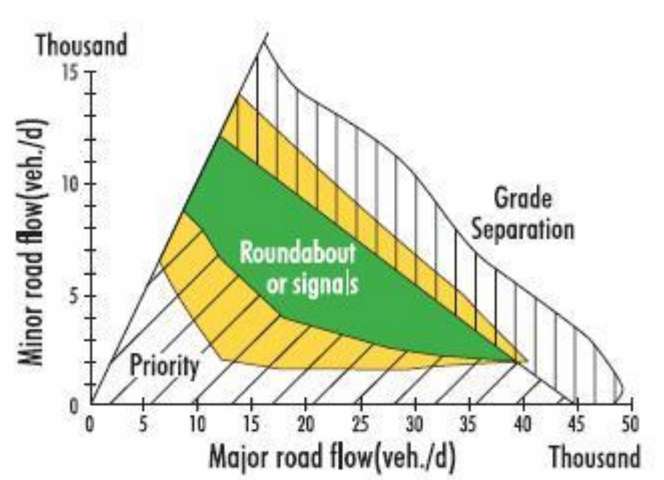

Figure 2.1: Type of junction based on traffic flows (https://ec.europa.eu)

2.2 Roundabouts

Roundabout is the one of the types of the intersection. According to Highway Capacity Manual (2010), roundabouts are intersections with a generally circular shape, characterized by yield on entry and circulation around a central island. Inner traffic flow direction at roundabout is counterclockwise where traffic continues from right side; whereas, clockwise traffic flow is seen at roundabout at the countries drive left policy. While decision-making process of the intersection, if the geometric conditions are available, roundabout is preferred due to security reasons because by its nature and geometric design, roundabouts reduce conflict points tremendously. Diverging and merging traffic at intersection is softly distributed at inner of roundabout (Crockett, 2015). These geometric design priorities decrease connection of vehicles that cause some conflict points at roundabouts. Reduction of the conflict points gives an opportunity to the reduction of the accidents.

Design of roundabout has been dealt with closely over the last few years. Implementation of roundabouts was firstly seen in Europe. Developments in USA followed the European countries implementation, so there are no specifics publications accepted as a nationwide design guide. HCM does not include a method for estimating performance measure for roundabouts. Therefore, developments at USA followed the European countries implementation. In this respect, to idealize EU implementation and give an idea about design of them, FHWA (Federal Highway Administration) published Roundabout: An Informational Guide in 2000. Moreover, Australian design guideline was published as Roundabouts-A Design Guide, issued by the National Association of Australian State Road Authority in 1986. Then, AUSTROADS Guide to Traffic Engineering Practice gives some information about roundabout at Part 6 at 1993. In this guideline, lane marking and signing implementation for urban and rural sites of the cities was presented. Also, British design manual is Geometric Design of Roundabouts, known as TD 16/93 provides geometric design standards for roundabouts with respect to operation and safety. Two design guidelines were published at France in 1988. One of them was for urban conditions by the Centre d'Etudes des Transports Urbains (CETUR, now known as CERTU), and another was for rural conditions by the Service d'Etudes Techniques des Routes et Autoroutes (SETRA). This guide indicates geometric criteria with respect to landscaping features. It offers outward slope of the roadway to increase the visibility of the central island, to facilitate the connection to the other roadways, and to simplify the drainage. At 1996, SETRA was updated. Also, Swiss Roundabout Guide was Guide Suisse des Giratoires, which was developed in 1991 by the Transportation and Planning Institute of the Federal Polytechnic School of Lausanne, Switzerland. This guideline explains integration of the roundabout into the urban space or general environment. The German Highway Capacity Manual was published in 2000. It presents some of the information about traffic criteria and urban design criteria for the installation of roundabouts. Table 2.1 shows design guidelines of roundabout according to different countries design guideline.

Table 2.1: Design Elements Stated by Three Guidelines

Description	British	Australian	French
Central Island diameter (at the non-mountable curbs)	Min. 4m	Min. 5m, Recommend 10m. Typical 20-30m	Min. 7m
Width of circulatory travel-way (curb to curb)	Max. 15m	---	Min. 6.5 - 8.5m Max. 9m
Inscribed circle diameter	Min. 15m Max. 100m	---	---
Entry width (Curb to Curb)	Min. 4m Max. 15m	Min. 5m	Recommend 5m for 1-In approach 8m for 2-In approach
Entry Radius	Min. 6m Recommend 20m	---	Recommend 10-15m entry radius<= inscribed radius
Exit width (curb to curb)	Recommend 7-7.5m	Min. 5m	Recommend 5-6m for 1-In, 8m for 2-In
Exit Radius	Min.20m, desirable 40m	---	Min. 15m, Max. 30m Exit rad. > Central island radius
Lighting	Required	Required	1. Required if approach is already lighted 2. Otherwise not required in rural areas

Source: Bared et al, 1997.

Operational variation, cultural diversity and driver perception could be different for each country but also for different areas at same country. That is why, there are no generally accepted geometric design requirements of roundabouts. According to the Traffic Safety Project done by General Directorate of Highways (KGM), SWEROAD, a review of guidelines and practices for highway design was made and proposals for amendments and changes were proposed in 2000. It stated that the roundabout central island has around 10 to 25 meters central island radius. For two lanes roundabouts, it should be at least 15 meters. Moreover, main determination of the circulatory roadway width is a number of the lane of widest entry. SWEROAD (2000) report also states that 33.5 meters is used at double lane roundabout inscribed circle diameter.

Operation of the roundabout depends on major design components of it. Each of components tries to utilize the roundabout design and reduce crashes, traffic delays, fuel consumption and air pollution. As stated at WSDOT Design Manual (2017), good design is a process of creating the smooth curvature, channelization and deflection required to achieve consistent speeds, well-marked lane paths and appropriate sight distance. However, site-specific details and requirements have some effects on components. Figure 2.2 shows some design components of the roundabout. These are main features of the roundabout design. All of the components of the roundabout design is explained in Table 2.2.

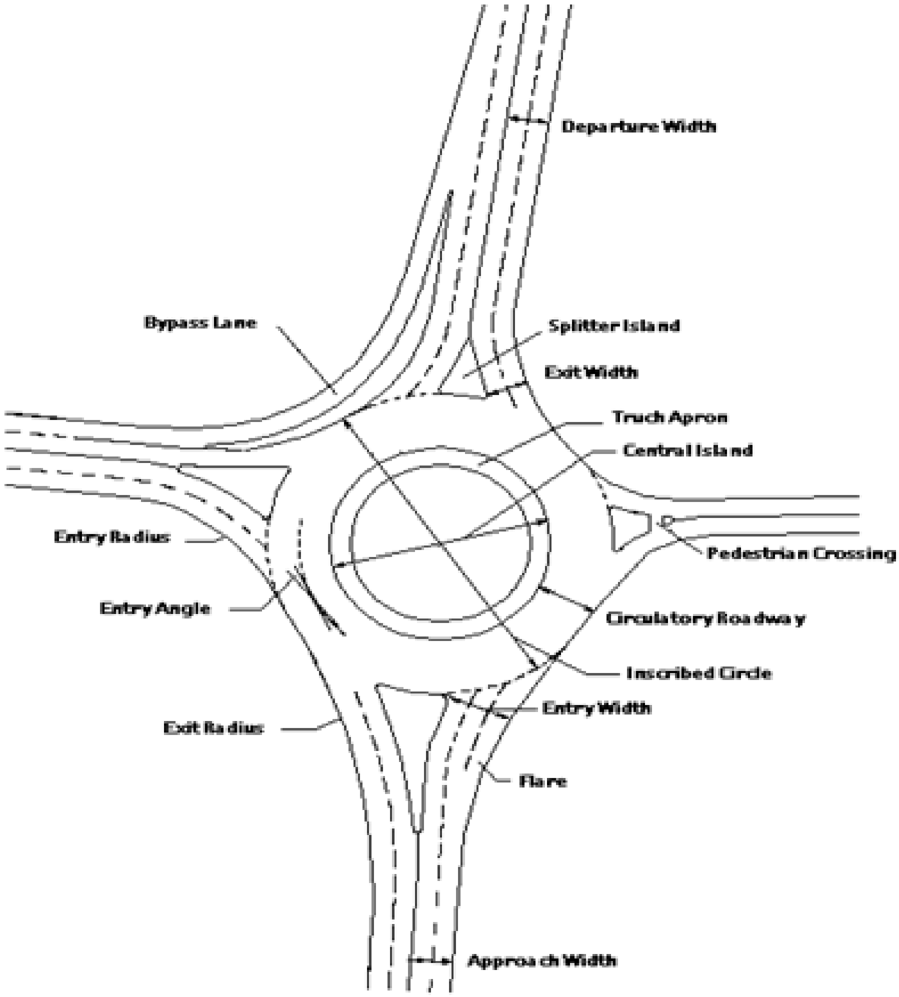

Figure 2.2: Design elements of roundabouts (https://www.fhwa.dot.gov)

Table 2.2: Design components of roundabouts

Inscribed Circle:	The diameter of the inscribed circle may not be bigger than 100 m. A minimum diameter of 30 m is required for roundabouts on the State highway system because smaller circles do not adequately allow truck movements.
Circulatory Roadway:	The width of the circulatory roadway depends mainly on a maximum of the entry lanes. The roadway must be at least as wide as or 1.2 times of the maximum entry width. Also, lane lines within the circle should not delineate. The pavement may be either crowned or sloped to one side, depending on the need to facilitate drainage or minimize adverse cross-falls for vehicle paths.
Central Island:	Inscribed circle and the width of circulatory roadway was used to identify.
Splitter Island	It is preferred to distribute entry and exit traffic in the circle and it locates at approach legs while designing, the raised curb is preferred not only to deflect the traffic but also to provide a pedestrian crossing.
Truck Apron	It is a component of the outer part of the central island. While oversized vehicles turn, it provides an area for their wheels.
Bypass Lane	It is preferred for right turn movements instead of inner of roundabout
Pedestrian Crossing	It is not allowed at inner of roundabout but approach legs provide area for pedestrian crossings. This helps to decrease the speed of the vehicle before entering the roundabout.
Approach Width	It is a width of the lane that approaches the roundabout.
Departure Width	It is a width of the lane that exits the roundabout.
Entry Width	It is a distance between the right curb line of the entry and the intersection of the left edge line and the inscribed circle perpendicularly
Exit Width	It is a distance between the right curb line of the exit and the intersection of the left edge line and the inscribed circle perpendicularly.
Flare	It provides an additional lane to increase the capacity of a roundabout at approach legs.
Entry Angle	It is important for the driver because smaller angle affects the visibility of the driver; whereas higher angles cause excessive braking on entry, so this may cause a decrease in the capacity.
Entry Radius	It is a radius of curvature measured along the right curb at entry.
Exit Radius	It is a radius of curvature measured along the right curb at the exit.

Source: https://www.fhwa.dot.gov

2.2.1 Properties of Roundabouts

Roundabout has been decreasing the conflict points at intersection. This makes driver and pedestrian movements easy at roundabout and makes them take and accord traffic decision process. Also, to make an intersection as a signalized type, operational and maintenance priorities of the signal control systems should be taken into account. With changing the traffic flows, traffic signals need to be retimed regularly, so this requires maintenance and operation systems renewed. That is why, while taking a decision about intersection type, cost of the signalized systems become an important factor. Although roundabout is thought as an alternative to signalized intersections, roundabout in some countries has been implemented with signal systems. In addition, flow is led at roundabout. Vehicular movements and their waiting time at intersections are decreased this means decreasing the pollution. Decreasing in the number of the conflict points at roundabout cause less interaction between vehicles, so vehicle at the act and stop activities, pollutions at intersections are reduced at roundabouts. Brılion & Vandehey (1998) mentioned that German and other countries indicate that roundabouts account for a reduction in noise levels.

On the other hand, roundabout could be classified at the at grade level intersection. It requires the flat area to locate. That is why, roundabouts can be taken into account for areas that allow acceptable outside diameter and other appropriate geometric design elements. In order to provide other design requirements of the roundabout should be located appropriately at the site. According to NCHRP (1998), it needs grade larger surface and maintenance of traffic during construction. Also, pedestrian activities at inner side of the roundabout are not allowed. Although some of the pedestrian crossing requirements and design factors are dealt closely with the approach legs of the roundabouts, their movement at the inner site of roundabout is restricted to other intersections. Priority of this is to increase mobility at the inner side. They do not care about the pedestrian activities, so pedestrian safety at inner of the roundabout less care about the issue.

Table 2.3: Advantages and disadvantages of roundabouts

Advantages	Disadvantages
Less Serious Accidents Cost Self-Regulating Less Pollution	Flat Area Pedestrians Safety Construction

2.3 Signalized Intersection Control

Two or more roads are sharing the same site at intersection. These sites give an opportunity to change directions. These types of sites create conflict areas. In order to decrease these types of conflicts and hotspot points, some of precautions have to be taken to control intersections. There is a different way to control intersection. Using traffic signals, signs or marking may regulate, warn and lead traffic. Garber & Hoel (2009) stated that aim of the traffic control is to help drivers, so this increases highway safety by ensuring the orderly and predictable movement of all traffic on roads. More complex and complicated areas for driver and pedestrian movements require properly designed traffic-control signalization systems. Therefore, placement of the traffic signals and their design properties like size, color and shape are important. Not only these but also recognition and understanding of signals at all locations for similar traffic and geometric characteristics of intersections are important for driver and pedestrians. It is possible to occur conflicts when traffic streams moving in different directions interfere with each other. There are three types of conflicts as merging, diverging and crossing. The number of conflict points at an intersection could be changed according to number of approaches, turning movements and type of traffic control at the intersection. Main aim of the traffic signals is to reduce the number of conflict points at intersections. That is why, while implementing to signalized systems to the areas, traffic flow characteristics, turning activities, number of vehicles, pedestrian activities and another type of concerns related to traffic flow are taken into account closely.

Traffic signals have been implemented throughout the cities in the world because most of the conflicts arise from movements of traffic in different directions is addressed by time-sharing principle. While implementing the systems to the intersection, orderly movement of the traffic and capacity increasing are aimed. Garber & Hoel (2009) stated that proper timing of the different color indications makes operation of the signal more efficient. That is why, some of the design terminologies have to be defined below to understand this system.

Table 2.4: Signal time terminology

Cycle	A signal cycle is one complete rotation through all of the indications provided.
Cycle Length (***C***)	Indicating the time interval between the starting of green for one approach until the next time the green starts.
Interval	Any part of the cycle length when signal indications do not change
Phase (***P***)	Nonconflicting movements to flow any safely stop the flow before another movement starts
Lost Time (***L***)	Time during which the intersection is not effectively utilized for any movement
All Red Interval (***AR***)	Red indication for all approaches
Effective Green Time (***g***)	Time for vehicle ability to cross intersection
Lane Capacity **(*Cap*)**	Ratio of green time to cycle length
Saturation Flow Rate (***S***)	Flow rate in lane if green indication is continues

Bajad & Ali (2016) stated that signal design procedure involves six major steps. These are

- Phase design
- Determination of amber time and clearance
- Determination of cycle length
- Apportioning of green time
- Pedestrian crossing requirements
- Performance evaluation of the design

Phase design is important to separate conflicting movements in an intersection into various phases. This creates much probability for phase design. Precise method for phase design could not be defined. As stated Mathew & Bombay (2014) studies, this is a trial and error process because system is led by the geometry of the intersection, the flow pattern especially the turning movements, and the relative magnitudes of flow. This is important further design procedures to determine cycle time and green time for flow pattern of the cycle. In addition, traffic flow at approach legs is controlled with signals. Movement of traffic could be allowed in opposite directions or each leg are moved independently to others. That is why, a different type of the signal control is taken into account into phase design of intersections.

Main traffic parameters at a signalized intersection include headway, saturation flow and capacity. Headway is represented as a time to measure the distance between two vehicles. According to Salim, Vanajakshi, & Subramanian (2010), headway is the time that difference between the arrival of the same point of the leading and the following vehicle at the observed area. Headway is used a variable of saturation flow rate, so statistical characteristic of this variable, such as probability distribution, means, medians, as well as detecting extreme values, gains more importance to determine saturation flow rates. Liu & Shao (2012) states that to measure the intersection capacity and time traffic signals, saturation flow has a fundamental role. Saturation flow calculations have been implemented if signs are all green at all times. HCM defines saturation flow rate like the equivalent hourly flow rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions, assuming that the green signal is available at all times and no lost times are experienced, in vehicles per hour of green or vehicles per hour of green per lane.

2.3.1 Determination of Saturation Flow at Signalized Intersections

Saturation flow is a type of flow at the full-time green. Most of the saturation flow observation is idealized this situation as Figure 2.3. The signal display is shown on the horizontal axis and the instantaneous flow of vehicles is presented on the vertical axis. According to Figure 2.3, some of the time is lost due to driver's perception to react. This is shown in the graph as start loss time. Then after a few seconds, the vehicle in the cycle reaches constant level until the traffic signals turn. This is known as yellow extension. The maximum rate of discharge, which is called at the Figure 2.3 Part an as saturation flow, is sustained in this period. When the amber signal phase occurs, it affects driver behavior. That is why, when the lights are amber, driver break their speed and take a position to wait but others do not disturb their activities until the light turns red. The usable amount of green time occurs between the end of the start-up delay and the end of the yellow extension. This period is accepted as effective green time.

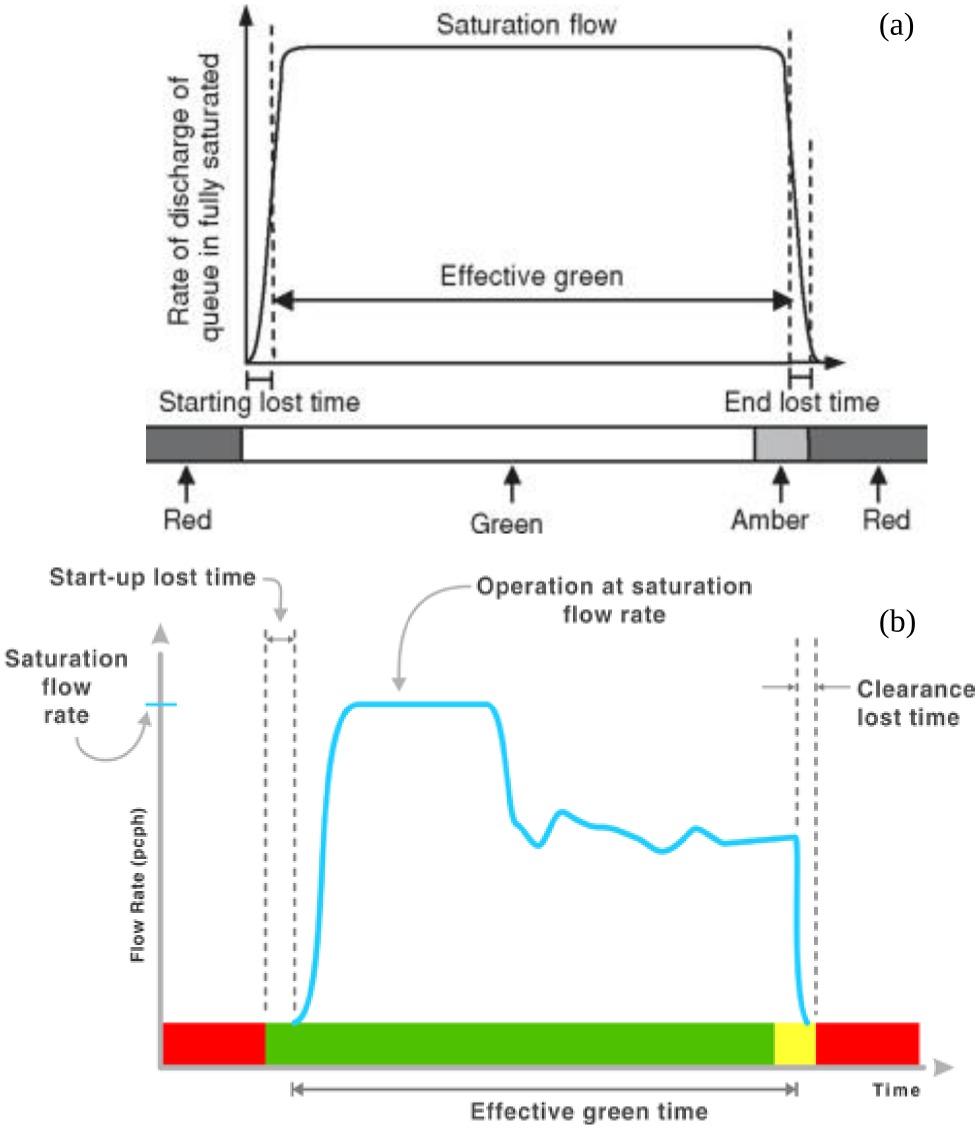

Figure 2.3. Saturation flow (a) idealistic case (Saha et al, 2009) (b) realistic case (https://www.fhwa.dot.gov)

According to idealized model, flow is constant at the effective green period. However, site-based saturation flow observations are a bit different from idealized one. Many components of traffic may affect the flow, so constant level flow rate could not be each for each cycle. Also, the idealized model implies that if the green time is long enough, this gives a number of the vehicles to discharge. However, operation flow could not be constant as offered an idealized case. During the effective green, there could be fluctuation in flow. In addition, saturation flow calculations have been done with respect to an effective green period, so idealize the effective period of the cycle, lost time determinations gains more importance. Like saturation flow, lost time at traffic flow is accepted as constant. Driver behavior to react to sign when it turns green is approximately is same between idealized graph. Boumediene, Brahimi, Belguesmia, & Bouakkaz (2009) determined that when lost times remains constant, the proportion of it becomes smaller at longer cycle time. Lost time is affected by the driver perception and reaction to signal a turn. While observing the traffic flow in a cycle, end lost time is as important as start-up lost time to define traffic flow behavior within a cycle. In this respect, Agent & Crabtree (1983) made a study and clarified end lost time as approximately 2.5 seconds at central business district locations. Also, reaction time to traffic signals, driver behavior of different age groups could be differentiated. According to Lu & Pernia (2000) studies, presence of the older drivers significantly affects intersection capacity

Saturation flow rate observations depend on time headway, measures the temporal space between vehicles with respect to time. Salim, Vanajakshi, & Subramanian (2010) stated that It is usually measured as the time lapse between the front bumpers of the first vehicle crossing over the same point. According to Highway Capacity Manual (HCM), saturation flow rate is calculated as

$$S = \frac{3600}{h} \quad \text{(2.1)}$$

where, S : Saturation flow rate (veh/h); Headway (s)

Site-based observations about time headway HCM clarify as shown in Figure 2.4. After the fourth vehicle, average headway becomes constant. This is the start-up lost time for traffic flow. Therefore, the fifth vehicle is acceptable as a starting point for saturation flow measurements while making a site based observations. According to number of vehicle in cycle, average headway could change. In this respect, most of the observations are done up to first 10 or 12 vehicles to generalize time headway at cycle. HCM while calculating the saturation flow observations about site-specific situations, HCM recommends a saturation flow rate of 1900 veh/h/ln, corresponding to a saturation flow headway of 1.9 second.

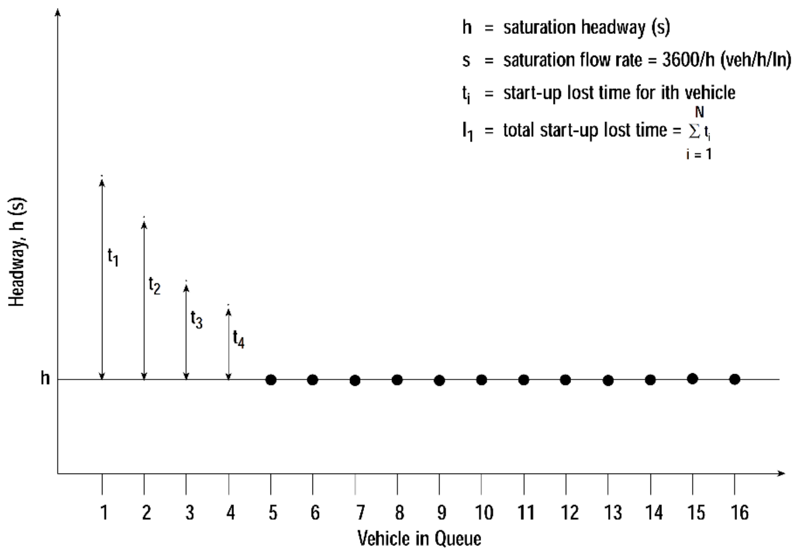

Figure 2.4. Concept of Saturation Flow Rate and Lost Time (HCM, 2010)

Saturation flow rate is an importance factor for measuring traffic performance. This value could be observed from the site. Many factors have some effects on saturation flow estimations. According to Stanic, Tubic, & Celar (2011), these are,

- Lane type
- Number and type of conflicts in the signal plan
- Regularity of the intersection geometric layout
- Traffic composition
- Discipline, aggressively and skillfulness of drivers
- Type of vehicle approach to an intersection
- Size of the settlement
- Vehicle characteristic

- Nonstandard conditions (weather, carriageway markings, pedestrian discipline…)

In traffic, through, left or right turns are made on lanes. With markings and signs on lanes, driver takes a position where they will go. This situation and its undesirable results as conflict points are taken into account at lane type. Turning traffic movements and degree of conflict points. Observations on the through traffic and some of the traffic situations including turning activities could be differentiated. According to Bang (1978), base saturation flow value for through traffic was 1700 vph; whereas, saturation flow for turning traffic only was 1500 vph.

Vehicle types in the traffic is an important factor affecting flow. Driver behavior, speed of the flow could be differentiated whether leading vehicle is a bus, minibus or truck. Heavy vehicle has an effect on a headway of the cycle around 3 to 21 percent (Nııttymaki & Pursula, 1997). This has some effects on saturation flow estimation. Nııttymaki & Pursula (1997) studies showed that 1 percent change in saturation flow is about 25 to 40 vph. On this issue, studies put some evidence about differentiated of pce for signalized intersections and roundabout. According to gap acceptance of the vehicle at circulating flow of inner of the roundabout, Tanyel (2005) stated that pce value for minibus is 1.15 pce/vehicle and 1.50~1.65 pce/vehicle for buses at roundabouts. In this respect, pce values for signalized intersections and roundabouts calculations done by Turkish Standards Institute (TSE) is shown below. Moreover, lane width is differentiated 2.4 to 4.1m. Less than 2.4-meter lane width is not implemented as width is taken as 2.4 meters at HCM calculations. As stated at Nııttymaki & Pursula (1997) observation, with increasing the lane width, saturation flow is increasing as it is shown in Figure 2.5.

Table 2.5: pce values in Turkish Standards TS 6407

Vehicle Type	Urban Roads	Traffic Circles	Signalized Intersections
Passenger car, taxi, motorcycle with sidecar, Light van up to 1500 kg unloaded	1.00	1.00	1.00
Minibus, Mini-van, taxi	1.15	1.30	1.27
Unloaded heavy commercial trucks weighing over 1500 kg, Horse carriage.	2.00	2.80	1.75
Urban and intercity passenger bus (including articulated), service bus, trolley bus, tram	3.00	2.80	2.25
Motorcycle, scooter	0.75	0.75	0.33
Bicycle	0.33	0.50	0.20

Source: Tanyel, 2005.

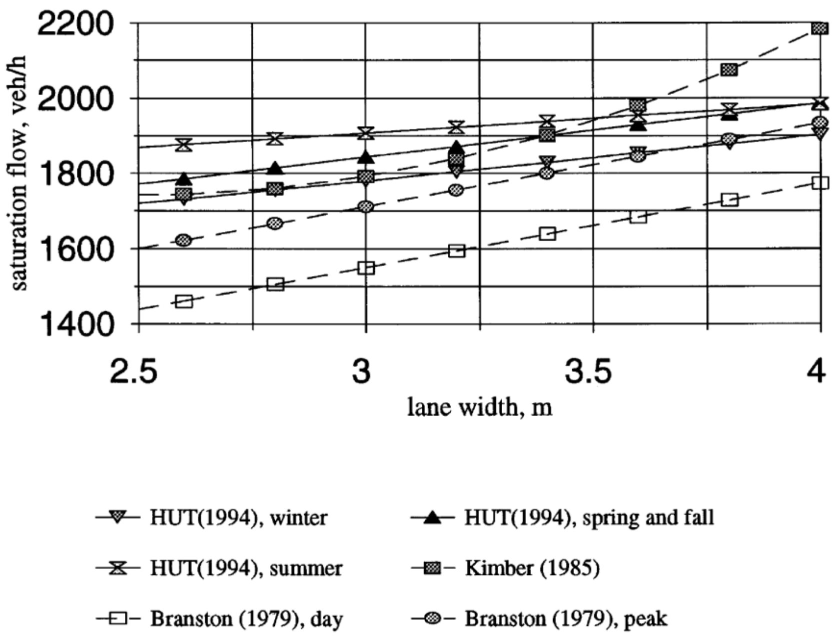

Figure 2.5. Effect of lane width on saturation flow values (Nıttymaki & Pursula, 1997)

Changing the environmental conditions could affect the saturation flow rate. Driver behavior at summer and winter could not be same. Therefore, lost times at winter are slightly higher with comparing the other seasons. According to Nııttymaki & Pursula, (1997), saturation flow value drops by 12 percent if the weather is rainy and 20 percent smaller under snowy conditions. Therefore, this requires a different signal program for extremely bad and traffic conditions.

While calculating the saturation flow rate at different sites or different countries, characteristics of the site could be taken into account. If the site is mostly used heavy vehicles, heavy vehicle adjustment factors should be dealt to idealize that site or compare here with others. Also, different countries could be implemented saturation flow observations according to their rules and cultures. That is why it is not easy to generalize saturation flow calculation for the whole of the world. However, some of the basic observations or some rules could be used and according to site behavior, saturation behavior calculations could be adjusted. According to Highway Capacity Manual, 2010 saturation flow observations could be done according to site situations as

$$S = S_o \cdot N \cdot f_w \cdot f_{HV} \cdot f_g \cdot f_p \cdot f_{bb} \cdot f_a \cdot f_{LU} \cdot f_{RT} \cdot f_{LT} \qquad (2.2)$$

where the adjustment factors are shown in Table 2.6. Some countries have slightly different methods in use, as in the case of Germany, which used the formula of

$$S = \frac{3600}{t_H} = \frac{3600}{f_{HV} \cdot f_1 \cdot f_2 . t_{H,0}} \qquad (2.3)$$

t_H, stands for adjusted saturation headway (see Table 2.6 for adjustment factors). In the revised edition of HBS in 2015 (Wu, 2017), base saturation flow was accepted as 2000 veh/h/ln corresponding to a base headway ($t_{H,0}$) of 1.8 seconds.

Table 2.6: Definitions for saturation flow adjustment factors in HCM and HBS

S	Saturation flow rate for all lanes in lane group(pce/h)
S_o	Base saturation flow rate per lane (veh/h/l, mostly accepted as 1900 pcphpl)
N	Number of lanes in lane group
f_w, f_b	Adjustment factor for lane width
f_{HV}	Adjustment for heavy vehicles in traffic stream
f_g, f_s	Adjustment factor for approach grade
f_p	Adjustment factor for existence of a parking lane and parking activity adjacent to lane group
f_{bb}	Adjustment factor for blocking effect of local buses that stop within intersection area
f_a	Adjustment factor for area type
f_{LU}	Adjustment factor for lane utilization
f_{LT}	Adjustment factor for left turns in lane group
f_{RT}	Adjustment factor for right turns in lane group
f_{Lpb}	Pedestrian adjustment factor for left-turn movements
f_{Rpb}	Pedestrian -bicycle adjustment factor for right-turn movements.
f_1	max (f_b, f_r f_s), -
f_2	min (1, f_s), -
f_r	Adjustment factor for turning radius *R*,
f_s	Adjustment factor for approach grade *s*,

The adjustment factor for heavy vehicles in traffic movement in HBS is calculated as

HBS, $f_{HV} = \dfrac{q_L + 1.75.q_{truck+bus} + 2.5q_{trailer}}{q_{veh}}$ …………………………………….….... (2.4)

where,

q_L : Flow rate of light vehicles (cars, vans, and motorcycles), veh/h

$q_{truck+bus}$: Flow rate of trucks and buses, veh/h

$q_{trailer}$: Flow rate of trailers, veh/h

q_{veh} : Total flow rate, veh/h= q_L + $q_{truck+bus}$ + $q_{trailer}$

On the other hand, f_{HV} in HCM is determined by

HCM, $f_{HV} = \dfrac{100}{100 + \%HV(E_T - 1)}$ …….…………………...……………... (2.5)

where,

E_T: Passenger car equivalent, for each heavy vehicle, E_T is 2.0 passenger-car. $\%HV$: % heavy vehicles for lane group volume. However, in Turkey, TS6407 reported passenger car equivalency factors (pce) for buses and minibuses as 2.25 and 1.27 respectively.

2.4 Statistical Background

2.4.1 Probability Distributions

It is also important to analyze the probability distribution of headway and determine average values accordingly. As headway is a real value variable, two selected distributions to explain the variability in the headways are selected as normal and log-normal distributions. Both of the distribution is used to simplify data of the headway. Distribution of the data is could be idealized better in one distribution. At normal distribution, an average of the data is in the middle of other data is distributed around middle value. Mean of the data is reflecting the extreme value and other data in the sample is distributed mean value. However, at lognormal distribution, an average of data returns will not be extreme value and extreme values could locate below and above of the average. Location of the mean value does not mean to reflect extreme value, so data in the sample will not be differentiated similarly around the mean data, unlike normal distribution.

Both normal and log-normal distributions are simplifying data distribution. Figure 2.6 explains how to compare normal and log-normal distribution of the data. In this respect, data is similarly located around mean value at normal distribution line of the figure; whereas, data distribution is not similar to a log-normal distribution.

Mean data on normal distribution is also accepted as the median of the sample. Data is also distributed around the median. However, it does not behave similarly at log-normal distribution. Mean of log-normal distribution is not necessary to be highest in the sample.

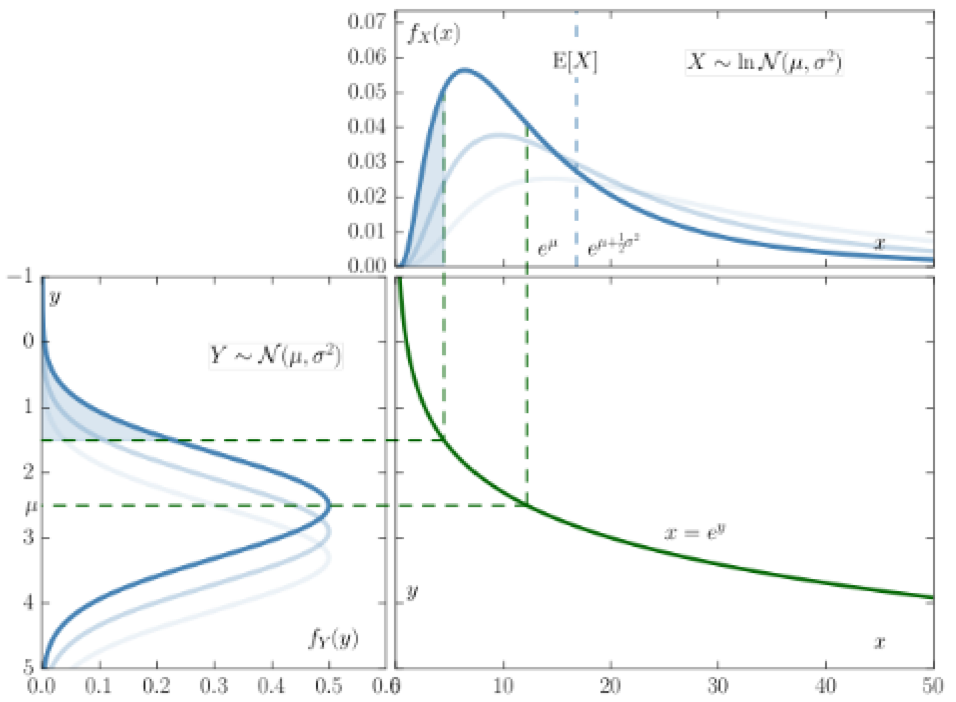

Figure 2.6: Normal & log-normal distribution of data (http://www.wikizero.org)

Normal and log-normal distributions are used to describe the probabilistic distribution of the data statistically. Prediction of data distribution is uncertain because observed data series may not be totally representative of the real probability of occurrence of the phenomena due to random error. Standart deviation could be used to predict a range of values of the data. This helps to idealize data within the range. In Figure 2.7, expected mean value and standard deviation of data are shown. Data distribution within the range gives a confidence interval. It is used estimate population of the data. Therefore, confidence interval gives upper and lower limit of the population. In most of the statistical observations, 95% confidence interval is used. In this observation, to set the upper boundary of the data, confidence interval is used at observations. Determination 95% confidence interval with is done with respect to equations shown in below.also, There are some of the jumpings of observed data. To determine the upper limit of data, the confidence interval is one of the methods of analysis.

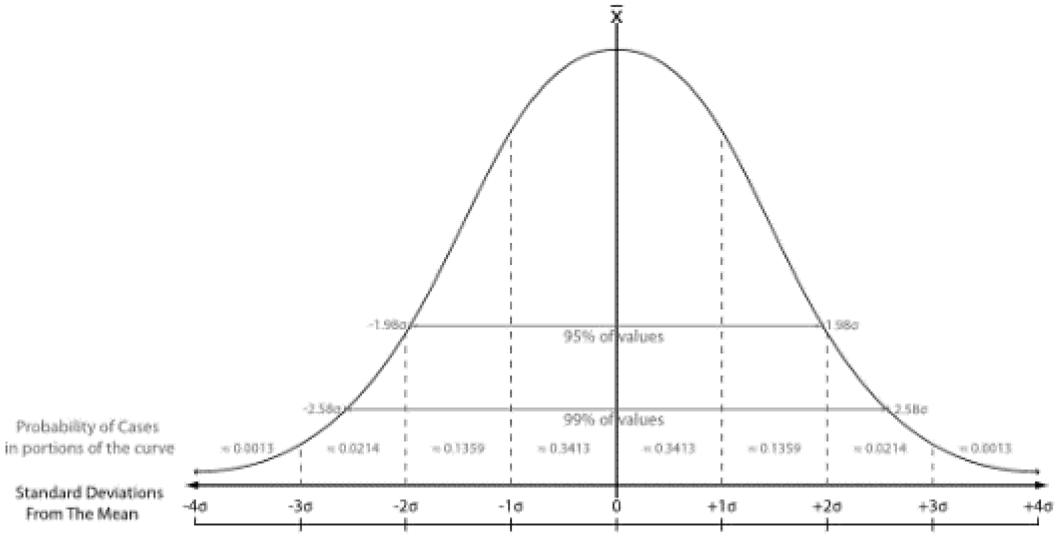

Figure 2.7: Confidence interval at normal distribution (www.liregentsprep.com)

It can also be shown as LU3 95% confidence interval and exponential value of the 95% of the data is shown as LU3a while analyzing the data.

$$CI = \bar{x} \mp z\sigma \quad \text{...} \quad (2.6)$$

where,

CI :Confidence interval; $\bar{x}$ Mean value; z : Probabilistic distribution value according to a percentile of range, σ : Standart deviation

For a 2-sided 95% confidence interval, $z = 1.96$, which makes the upper limit (LU)

$$LU = \bar{x} + 1.96\sigma \quad \text{...} \quad (2.7)$$

However, in analyses where the lower limit is not critical from the point of view, it is also possible to assume 1-sided confidence interval, where 95% confidence will correspond to $z = 1.64$, providing an upper limit of LU'.

$$LU' = \bar{x} + 1.64\sigma \quad \text{..} \quad (2.7a)$$

To check normality of the data, a Q-Q plot is used, which is also called a quantile-quantile plot. It is created by plotting two sets of quantiles against one another. Both quantiles come from the same set of normal distributed data. Therefore, observed data distribution along the diagonal line is shown in this observation. The diagonal line is showing expected and observed data similarity. The 45-degree reference line is plotted. Q-Q plot provides a visual comparison of the sample quantiles to the corresponding expected quantiles. If two sets of data come from a population with the same distribution, data should fall approximately along this reference line. To analyze the distribution of data around the reference line, it does not necessarily to fit data. In order to compare data distribution, how it is located around the reference line is important. Moreover, to compare of the distribution of different peak periods observation, the main advantage of Q-Q plot is that sample size does not to be equal. In this respect, IBM SPSS Statistics V20.0.0 program allows a test of the data within exploring command for each observation period.

2.4.2 Extreme Values Determination

In random variable data collection, it is important to detect the extreme values which can affect the average value estimations drastically. This is also called "data cleanup" and is a common practice. It is important and necessary to set thresholds for the detection of outliers that had to be eliminated before analysis in order to find average headway. The range of data set was greatly influenced by the presence of just one unusually large or small value. Therefore, outliers should be determined to eliminate points affecting the result of the distribution in the sample. Different techniques were implemented data to fit best outliers to the data. One way was to use interquartile range, IQR as a guide. It measured the variability of data. In this respect, quartiles were values that divide data set into four equal parts containing an approximately equal number of observations. Each of them represented 25th percentile of the sample. In addition to the minimum and maximum values (also shown as Q0 and Q4), each 25-percentile limit was represented with Q1, Q2 (also called median) and Q3 respectively. In this evaluation, a critical measure was the variability of the middle 50% of the data. It was called IQR which measured as the difference between Q3 and Q1. Moreover, these quartiles, percentile distribution of the data, were clearly seen at box plot. Box plot is based on percentages of the sample size, not the sample size itself, so five values are arranged on the box plot as a minimum, first quartile, median, third quartile and maximum respectively. Each section of the boxplot contains 25% of the data. At observation of the data, 6 box plots have been placed on the same graph because they show how data evolves with time and how it is differentiated data at different observation period. In this respect, some of the data at the observed data set displays high maximum or low minimums. To define these outliers, some of the upper limit constraints had been determined. Outliers are defined as those values, which are 1.5 times IQR above or below the third quartile, producing an alternative upper limit (LU_a).

$$LU_a = Q3 + 1.5IQR \quad \text{(2.8)}$$

Sometimes, assuming a 3*IQR* tolerance, extreme outliers are detected by an upper limit LU_a'

$$LU_a' = Q3 + 3IQR \quad \text{(2.8a)}$$

2.4.3 Moving Average Methods

Idealistic and realistic observations can conflict each other to define traffic flow conditions for each cycle. To see a general tendency in the cycle, moving average is accepted as a method to get an idea of the trend in a data set (Droke, 2001). The averaging "moves" over time, in that each data point of the series is sequentially included in the averaging, while the oldest data point in the span of the average is removed (Ward, 2006). Therefore, the average is measured at several times for several subsets of data. Moving average is calculated in the sample as shown in the formula below.

$$x_i = \frac{x_i + x_{i-1} + x_{i-2}}{3} \qquad (2.9)$$

CHAPTER 3

METHODOLOGY

3.1 Study Area

Observations were done on the signalized roundabout at Konya, Turkey. It is a junction of four urbanized street. At northeast and southwest direction of the roundabout, Çevre Yolu Caddesi has been locating. Roundabout has been operated with traffic signals not only at the approach legs but also inner of the roundabout. Traffic flow is higher in a north-south direction. Northeast part of this direction gives a chance to arrive in Medicine Faculty of the Selcuk University. Southwest part is also to the Sille. It is the one of the important settlement of the Konya due to historical and environmental properties. The vehicle in this direction moves through the city center and it makes easy to reach industrial part of the city. East part of the roundabout is Sefik Can Caddesi. PTT and some high-density residential areas are locating here. Also, west part of the roundabout is locating the Genc Osman Caddesi. Aksemsettin mosque on the Genc Osman Cadde is 150 meters away from the roundabout. There is also some of the middle scale shopping mall is 150 far away from the site and high-density residential areas are locating this part of the roundabout.

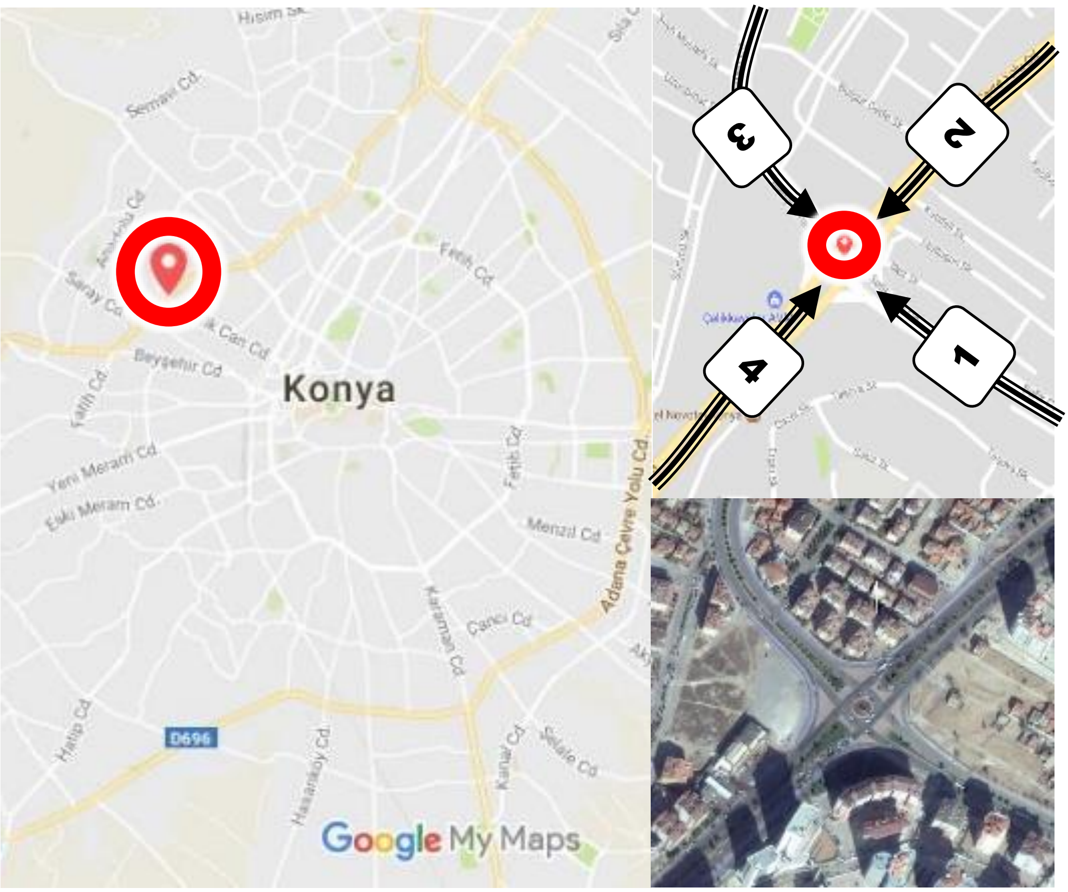

Figure 3.1: Location of the study area (Google Map/Earth, 2017)

The study area is managed as dynamically. Unlike fixed time plans, series of logic statements and their replacements are chosen from a matrix holding a series of plans relating to differing traffic flow conditions. Dynamic intersection control systems have been used to control traffic signalization. While making these, main purpose is to optimize signal timing according to number of the vehicle. Cycle length is not constant. According to traffic density with respect to many factors in the approach and connected road conditions, cycle length duration become accordable. This helps to reduce waiting of the vehicle in the traffic and reduction in the queue of the traffic. In this respect, some of the electronic management devices are located to intersection control systems to accord green lights automatically. Therefore, by using the cameras, data related to traffic density is recorded real-timely. This helps system to analyze in time and manage the traffic signals, so waiting in the red light in traffic could be reduced for vehicles. Moreover, system could be worked with other intersection situation. This provides integration of the other intersections. According to their density, systems could reduce traffic jams at others by according to signalization time. This increases both intersection and roads between intersection traffic flow fluently. Moreover, waiting time at traffic is reduced. Therefore, carbon monoxide and sound pollution could be minimized and fuel consumption of vehicles could be decreased. Moreover, according to ISSD Inc (2016), safe period's production of the system decrease the red light violations, so traffic accidents could be minimized.

Vehicle density of each approach is differentiated between them. Main traffic flow is at Cevre Yolu Caddesi direction (Approach 2, A2, and Approach 4, A4) as shown in Figure 3.2. These directions were accepted as major direction of the observed area. Sefik Can Caddesi and Genc Osman Caddesi are donated as Approach 3 and 4, A1 and A3, respectively. Also, each approach leg has three lanes while making lane based observations, the innermost lane was donated as Lane 1 and outermost was demonstrated as Lane 3. Lane 2 was accepted as a lane between them. These are shown in Figure 3.2. Each of approach legs site view is shown in Figure 3.2. Inner and approach leg of the roundabout signalization, physically design properties of roundabout is seen in this figure. Also, the figure shows a number of the lane at inner and approach of roundabout. Both of them have been signalized with traffic signs.

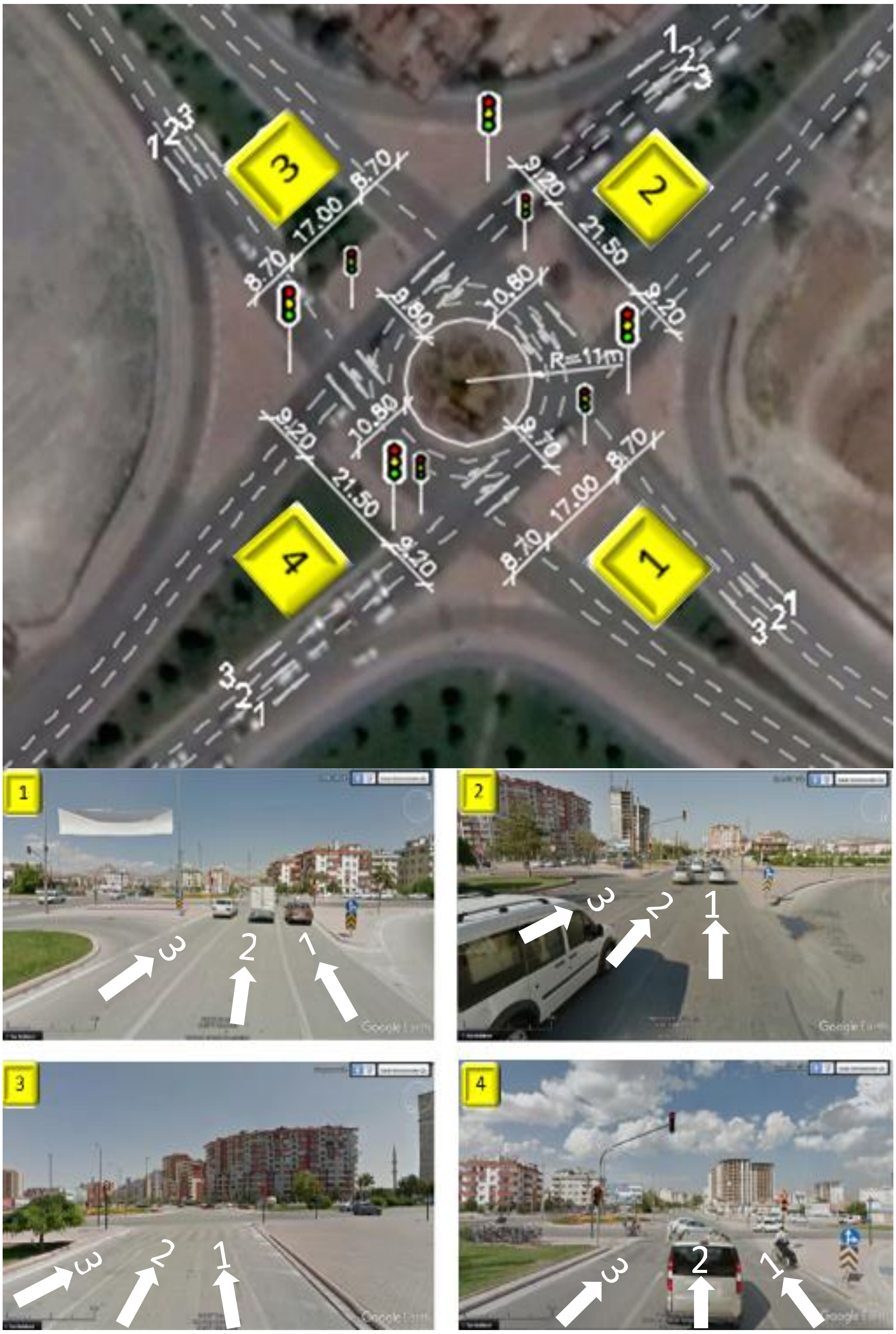

Figure 3.2: Intersection geometry and views of approach legs with lane numbers

Figure 3.3 shows technical equipment locations of the site. In this respect, three cameras are located at the site for observing vehicular and pedestrian movement. ISSD Inc. dynamically controls area and signal timing is accorded with respect the pedestrian and vehicular movement at the site. Each of the number at the figure indicates the traffic signals at the area. Numbers from 1 to 9 are referring the traffic signs for vehicular movement; whereas, numbers from 21 to 28 denote signalization for pedestrian activities.

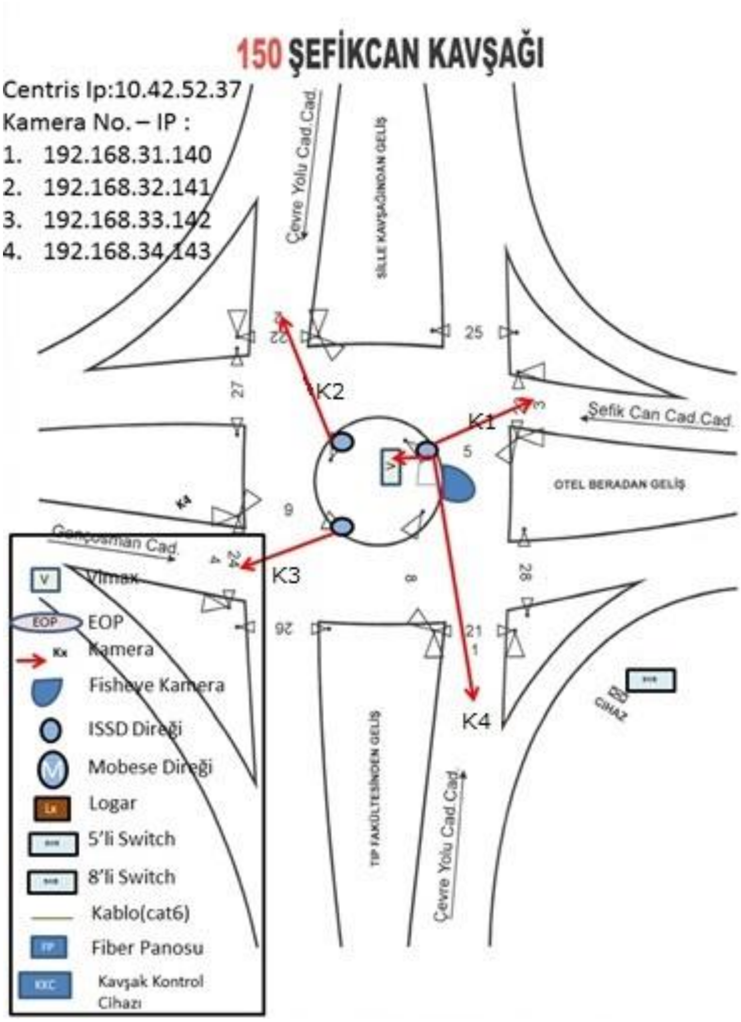

Figure 3.3: Technical equipment location at dynamically managed roundabout

Table 3.1: Characteristic of roundabout

Description	Study area
Central island diameter (at the non-mountable curbs)	22m
Width of circulatory travel-way (curb to curb)	10m
Inscribed circle diameter	45m
Entry width (Curb to Curb)	9m
Entry Radius	39m
Exit width (curb to curb)	9m
Exit Radius	39m
Lane width	2,9m
Inner lane width	3,6m
Lighting	Inner & approach legs

Area is controlled with respect to four phases signal control systems for both inner and approach leg of the roundabout. The first movement of vehicle at inner of roundabout is allowed at this system. After leaving the car inner side, approach leg movement is supplied, so traffic flow at Approach 2 and Approach 4 are allowed at the same time. This means that green times for opposing approach legs is same. Although cycle duration for whole approach legs at that time is same for them, green times are identical for opposing legs. To idealize the signal durations of each cycle, Figure 3.4 is given. It represents the light sequence of the observed day of the study area. Because of the dynamically managed roundabout, Figure 3.4 represents the cycle duration of the observed area for each observation period of the day. The figure is prepared for vehicular and pedestrian movement. Therefore, as given in Figure 3.4, the signal duration for pedestrian activities could be observed from 9th to 17th column; on the other hand, first to 8th column is indicating the vehicular activities. Cycle duration of each observed period is the difference between them.

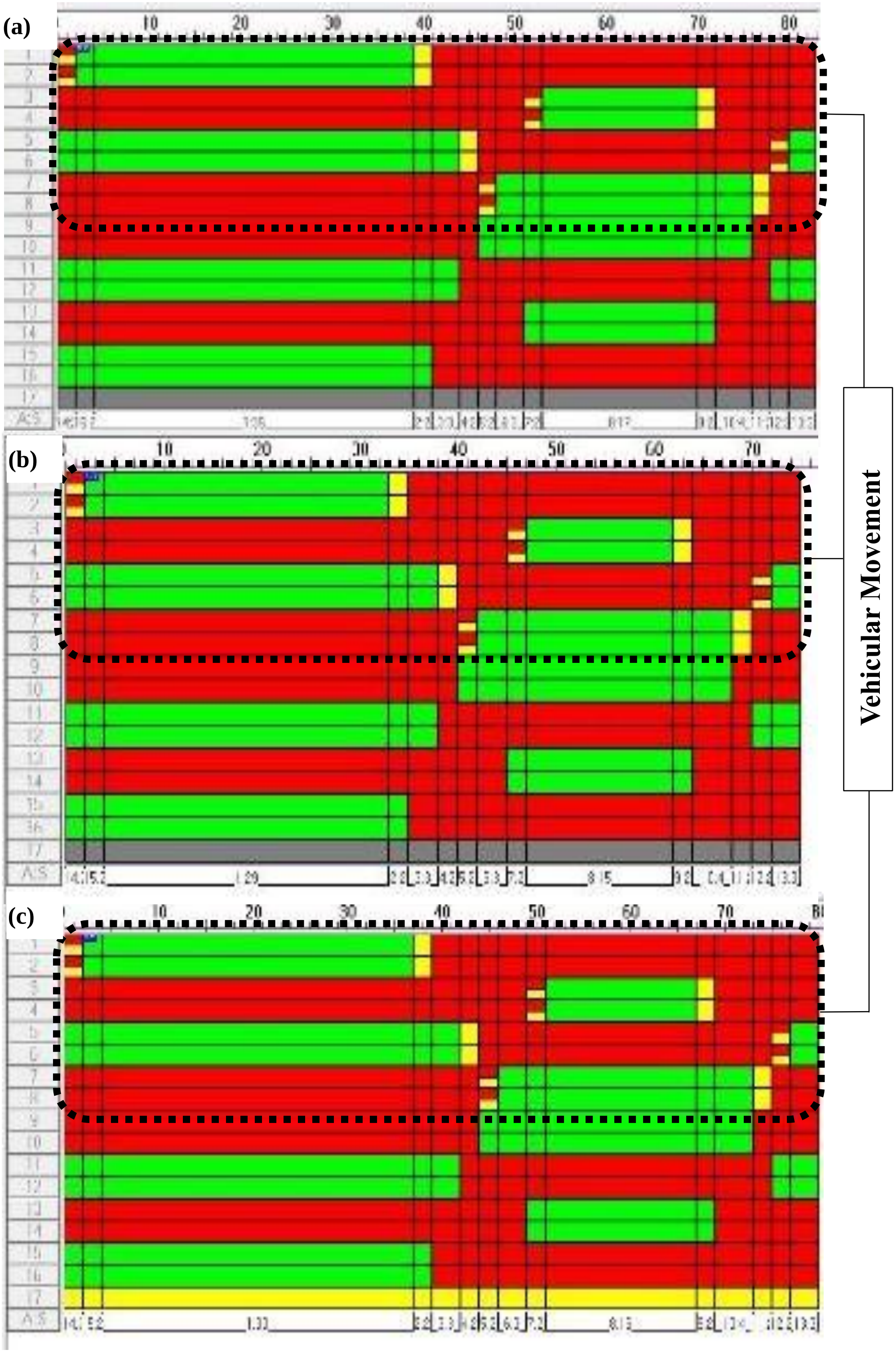

Figure 3.4: Light sequence of the peak period of study area (a) AM (b) NOON (c) PM

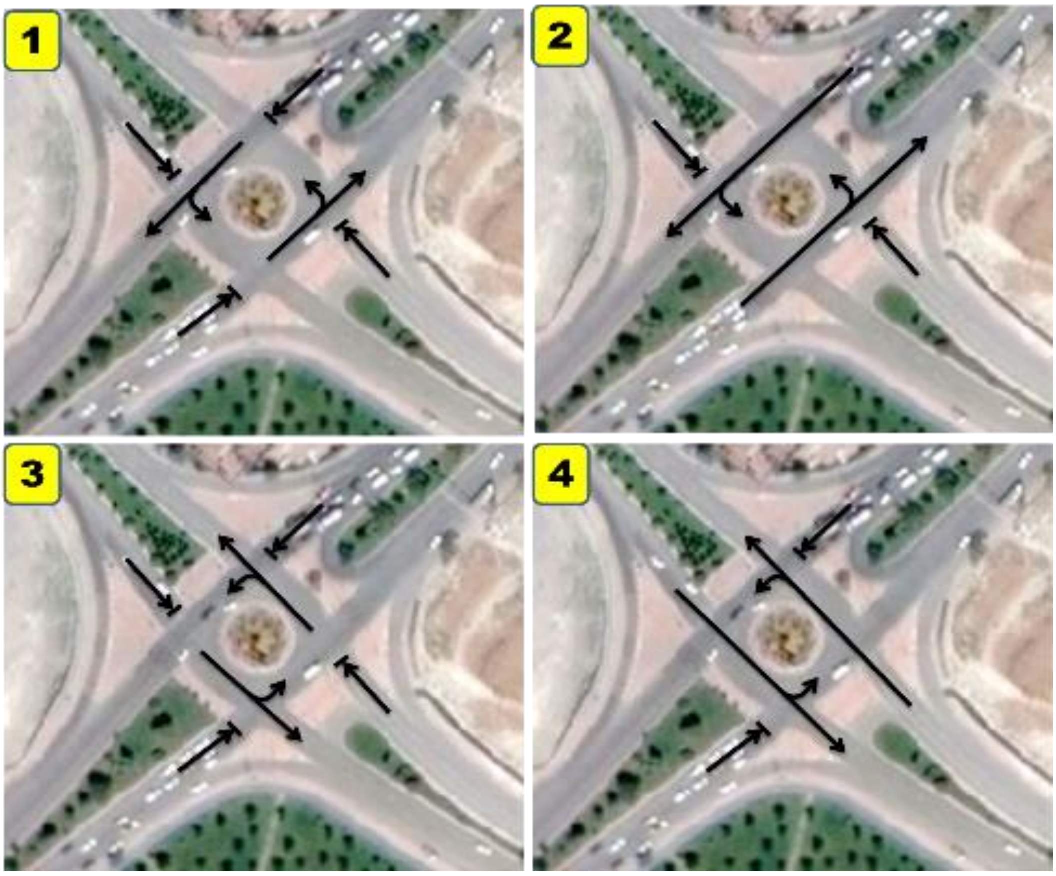

Figure 3.5: Phase diagram of study area

Inner and approach leg vehicular movements at study area are seen in Figure 3.5. In this respect, the study area has four phases. Before movement of vehicles at approach legs, vehicles at inner of roundabout is allowed to move. This help to discharge of traffic at inner of roundabout before reaching vehicles at approach legs. Part 1 represents movements at inner of roundabout. After a few seconds, vehicles at approach legs are allowed to do their activities. This is shown in Part 2. After completing movements in one direction, other direction which is seen at the figure east-west direction, complete their movements. Phase diagrams of other direction are seen as Part 3 & 4 in Figure 3.5. Moreover, through and left turn movements are seen in the study area. Right turn flow is always supplied at bypass lanes at each approach legs. Due to no traffic signs, right turn movements are done without any interruptions.

3.1.1 Design Characteristics of the Roundabout

General design components of the roundabout are shown in Table 3.1. Central island in the study area is 22 meters appropriate with the SWEROAD guideline (report prepared for KGM. Moreover, main determination of the circulatory roadway width is a number of the lane of widest entry. The total width of the lane at the inner side of roundabout is 10 meter. Although this is a bit higher than the French standards, width of roundabout is suitable with other standards. SWEROAD (2000) report states that 33.5 meters is used at double lane roundabout inscribed circle diameter; whereas, study area is proper with the English standards. The main restriction at this point is optimal size of location of roundabout. As stated at FHWA (2000), the capacity of an approach is dependent on not only the number of entering lanes but also the total width of the entry. The width of the observed area is around 8.70 to 9 meters for not only entry but also exit width. This is suitable for British and Australian design guidelines. Report prepared for KGM states that main radius of entry and exit should normally be around 15 meters. At observed area, entry and exit radius is 39 meter, which is higher than offered. Width is also at approach legs and inner side of the roundabout is approximately 3.5 meter. Due to signalizations at inner and approach legs, study area is differentiated of the implication of roundabout design from USA and European countries because most of the roundabout in those countries have been non-signalized. Some of the examples include approach leg signalization but none of them has any signalization at inner of roundabout.

3.2 Data Collection

Roundabout data is taken away from the site. To reach data of study area, ISSD Inc. that is the dynamically managing roundabout allows using their systems to take a record of the site. This system provides us to view of the roundabout and its inner and approach legs situation. Fisheye camera can be organized to see each approach to the roundabout. While making this, systems offer 2, 4 or 8 camera side view. In order to see details of every approach and, catch to movements of the vehicles at every approaches and lane, camera is chosen 4 sides. Figure 3.7 in the below shows how fisheye camera was located at roundabout and distribution of the technical equipment there.

Video recorded program is used to take a record. Data of the thesis is related to 24.02.2016 dated. That day was weekday and schools were opened. So traffic flow situation is reflected the real daily traffic condition. Moreover, observations were conducted at the morning, noon and evening peak periods. Morning observation was done between 7:29-8:54, noon data collection was done at 10:59-12:30 and evening observation was done at observation was 16:59-18:30. Each of the peak periods is approximately 90 minutes.

Figure 3.6: Site view from data observed fisheye camera

Video recorded program, CamStudio was used while accessing the dynamically control study location remotely. To improve data quality, each camera view was accorded to see not only approach legs but also inner site of the roundabout. Traffic signals of the each approach legs was accepted as a node mark for every lane to determine vehicle headway and their lanes. While observing the data, these nodes were marked by rope. This provided determination of the vehicle headway. With help of this, passing time of each vehicles was recorded at each lane. Some of the programs based on the pixels timing observation has been used to check results of the data with respect time recording data. In this respect, errors on reaction time was minimized

3.2.1 Data Digitization with MATLAB R2010a

Observations of the roundabout was depending on lane-based, so each of the vehicle type, headway, total number of the vehicle, cycle length, used green and others were taken into account. That is why, MATLAB R2010a program was used to determine them. MATLAB code was written as shown at Figure 3.8. This program was used to analyze the data and provided raw data to observe. Some of the special codes written to decipher the data. First, ISSD systems was opened and connected to roundabout lively. Also, Cam recorded program was started to take a view to the computer. ISSD systems and camera recorded had been worked together. The camera-recorded program to the computer recorded view of the roundabout. In order to see the results taken from the camera recorder program, MATLAB code was used to digitize the data. This code is depended to time and vehicle classification. Lane-based observations of the raw data taken by the cam-recorded program was done at MATLAB Program. While entering the code, it was asking firstly name of the study area then starting time of the recording data. Recording time was seen at the left side of the ISSD system and manually time was written to the code. Also, direction of the flow and lane based observation data was written to the system. All of these provided general information about study area. Some of the shortcuts were used to determine vehicle types. Car was defined as 1, minibus was called as 2, bus was coded as 3 and truck was determined as 6 to the program. This gave a chance to record both type of vehicle and headway of vehicle and helped further observations.

```
Command Window
Enter intersection name:sefikcan
Enter video start time (ayracı nokta olarak giriniz):07.00
Enter the direction :3
Enter the observation lane:3
Enter p_car(1), midibus(2), bus(3),truck(6)(to exit press x):1
Enter p_car(1), midibus(2), bus(3),truck(6)(to exit press x):2
Enter p_car(1), midibus(2), bus(3),truck(6)(to exit press x):3
Enter p_car(1), midibus(2), bus(3),truck(6)(to exit press x):6
Enter p_car(1), midibus(2), bus(3),truck(6)(to exit press x):x
Enter video end time (ayracı nokta olarak giriniz):07.10
fx >>
```

Figure 3.7: Observation of the data by using MATLAB R2010a program

3.2.2 Data Analysis with IBM SPSS Statistics V20.0.0

All of data taken from site were organized at Microsoft Excel program and imported to IBM SPSS Statistics V20.0.0 program. SPSS (Statistical Package for the Social Sciences) was preferred to adapt present data in research work. The uses of SPSS helped to make the research work more scientific and reliable by using the some of the statistical approaches. Statistical data distribution of headway was analyzed with this program. Probability distribution of the data and normality curves to interpret the distribution of the data were done at the study. While observing time headway of the data distribution, Q-Q plot was used. Percentile distribution of the data was also shown with box plots. This helped data to cluster within a percentile range. Effects of each observed data on saturation flow were explained with regression analysis. It was done to explain of significantly important of data.

3.3 Signal Data Preparation

Data taken from video record program was analyzed at the MATLAB program firstly. This analysis period at program gave some details about vehicle type and vehicle passing time for each lane at the roundabout. Then, analysis began to take time differences between vehicles. This time difference between vehicles gave headway of each vehicle at each lane of the approach leg. Observations were done on camera recorded program. Therefore, traffic signs could not be determined. Green time of each cycle was observed from starting a movement of vehicle. This means that driver firstly perceives green sign and reacts moving of vehicle. Observed value of the green from the site was differences between the first vehicle movements to last vehicle passing time in the cycle. Therefore, green time calculated from the observation was called used green time instead of effective green time. Study area approach legs have four signalization phase. Traffic signs behavior is same at the two opposing directions. Like observation of the used green time, red time observation was done according to movement of the vehicle. Differences between first vehicle movement and movement of last vehicle at previous cycle gave used red time of the former cycle. Cycle time duration was considered by summation of used green and used red time of the cycle. While observing, cycle duration of all approach legs at that time was same. If the cycle had no vehicle, calculation of the green time and re time could be a failure. That is why, each duration of cycle length observations was done with respect to first and last movement of vehicle and their relationships with former cycle was taken into account to calculate cycle length.

While taking results from MATLAB code, it gave some information about vehicle type. In this respect, how many vehıcles used the cycle at that approach leg based on the lane, which type of vehicle came on each lane was answered. Also, position of the vehicle not only ın cycle but also in lane was observed. This gave a chance to evaluate effects of previous vehicle effect on the former vehicle. In order to analyze headway, saturation flow rate and capacity of the intersection, all these parameters became variable to find them. Both saturation flow and capacity calculation were done with respect HCM (2010). Notation of variables in data processing and some of the formulas are shown below.

Table 3.2: Table of notation

a	Approach direction
i	Cycle
l	Lane
$h_{i,j}^{a,l}$	Headway of vehicle j on lane l in cycle i of a approach
$t_{i,j}^{a,l}$	Observation time of vehicle j, on lane l in cycle i
$g_{used,i}^{a}$	Used green time of a approach in cycle i
$r_{used,i}^{a}$	Used red time of a approach in cycle i
$\hat{C}_{i}^{a}$	Signal cycle time in cycle i of a approach
$V_{A,j}^{i,l}$	Vehicle type of j on the lane l in cycle i of a approach
$S_{i}^{a,l}$	Saturation flow on lane l in cycle i of a approach
$N_{i}^{a,l}$	Number of the vehicle on lane l of a approach in cycle i
$N_{veh,i}^{a,l}$	Number of the vehicle type on lane l of a approach in cycle i
$V_{pre,i}^{a,l}$	Previous vehicle type j on the lane l in cycle i of a approach
$T_{s,i}^{a}$	Cycle start time of i^{th} cycle of a approach
$T_{e,i}^{a}$	Cycle end time of i^{th} cycle of a approach
Q_{avg}^{a}	Average flow rate a approach (veh/h)
$Q_{avg}^{a,l}$	Average flow rate for lane(veh/h/ln)

$$h_{i,j}^{a,l} = t_{i,j}^{a,l} + t_{i,j-1}^{a,l} \quad \text{......} (3.1)$$

$$\hat{C}_{i}^{a,l} = g_{used,i}^{a,l} + r_{used,i}^{a,l} \quad \text{......} (3.2)$$

$$N_{i}^{a,l} = \sum_{veh} N_{i}^{a,l} \quad \text{......} (3.3)$$

In order to eliminate video recording error, first cycles were eliminated because data was recorded while the traffic is fluent. Also, first vehicle headway of each lane was disregarded because up to traffic signs turn green, first vehicle was waiting due to driver perception and reaction to lights. That is why, first vehicles of each lane was eliminated. To idealize the data and get more accurate from observation data, some statistical analysis was done. This gave some thresholds for data, so some of the observations were done once again at further observations.

Observed headways were further coded into two categories, major and minor approaches, represented by type of 100 and 200, respectively. These variables were created to investigate headway characteristics of different regimes on different approach legs. Also, there were three observations period at morning, noon and evening period of a day. Each of them was coded as at starting of the observation period like 730 for morning, 1100 for noon and 1700 for evening analyzed period. That is why, each of cycle observation period was donated in this respect. In addition, cycle ID was determined by summation of cycle number and analyzed period.

3.4 Statistical Characteristics of Headway Data

Useful and meaningful characteristics of the traffic flow can be obtained by studying the headway. Headway is an important parameter for planning, analyzing, designing and operating roadway systems (Jakimavicius & Burinskiene, 2009). In order to investigate headways at the approaches of an urban roundabout statistically, observed headway data was first explored using IBM SPSS statistics V20.0.0 program to get its descriptive statistics, such as minimum values, maximum values, quartiles and interquartile range (IQR). Additionally, 85 percentile value for the time headways was also monitored and shown as $h_{0.85}$. As different traffic regimes on different approach legs may represent different characteristics, these statistics were prepared for three study periods and two approach directions (majors versus minors) separately.

3.4.1 *Time Headway Data Clean-up*

All of the observations about 85 percentile, outlier, an extreme outlier, and 95% confidence interval were done for determining the threshold of the data. First, using IQR based upper and lower limits defined in Equations 2.8 and 2.9, two upper limit threshold values for time headway data, LU1 and LU2, to be included in the averaging process were calculated. A third limit was determined to assume log-normal distribution for time headways and using a 95% confidence interval, that produced LU3a. Truncation of the data after all of the observations done for determining the outlier was shown at analyzed process. Studies were maintained by using truncated data. Also, upper limit constraints for truncated data was shown

LU3b. Like other observations, this gave a chance to compare the outlier limit while determining the truncated value.

Vehicle Type Effects on Headway:

Effects of the vehicle type on the headway data calculations became a questionable issue for analysis period. Different vehicle interaction on traffic causes a difference in headway of the vehicles. Also, reaction time and perception of the driver behavior to other vehicle is changing according to the vehicle type. Observation done by Kumar, Parida, & Saleh (2014) for Silchar City stated that time headway differences between leading and following vehicle is around 1.3 to 2.4 seconds and for bus, headway is approximately is 2.6 to 6.6 second. However, Sumner & Baguley (1978) stated that there are no results according to the type of vehicle actually being followed while measuring differences in how car and truck drivers follow lead vehicles; whereas, Evans & Rothery (1976) found that changing the lead vehicle size affects the driver decisions about time and space headway. Sayer & Mefford (2003) stated that if lead car is a truck, this reduces the driver concentration because driver takes care of the only leading car, so it has a reduction in driving workload. In this respect, direct statistic observations about headway analysis were deeply analyzed about effects of the vehicle type on headway. In this respect, observations did about vehicle type. This gave an opportunity to evaluate the effect of vehicle type on headway calculations.

Many observations have been closely dealing with the effect of the vehicle types. That is why, they have been trying to generalize all type of vehicle in the same category by using the passenger car equivalency factors. However, different observed area makes it difficult. At study area, different observations have been done to find constants for buses and minibusses to convert vehicles. However, the statistical significance of them was high. This made a questionable thing for those constants. That is why, constants at TS6407 was used while using the passenger equivalency factors.

Lane Effects on Headway:

Observation site has three lanes at all approach legs and inner side of the roundabout. Effects of the approach leg and each approach legs lanes headway capacity became a questionable issue for comprehension of effects of lane headway on headway of the roundabout approach legs. According to HCM 2010, exclusive right turn lane should be considered if the right turn volume exceeds 300 veh/h and adjacent mainline volume exceeds 300 veh/h/ln. Right turning activities are allowed with bypass lanes before entering the roundabout signalized area. At observation site, right turning activities were done with bypass lane without any signal control. Therefore, through and left turning activities was allowed at the observation area. Bonneson (1992) found that through movements have larger headways than those of the left turn movements. At this point, turning activities cause some interruptions for vehicles. Lin (1992) stated that movement of the vehicle could be interrupted due to left turn vehicles. While observing, due to not physically separated lane for left turn movement, lane-based observations about headway are affected. This causes differentiated in capacity-based calculations for each lane. At this point, Levinson (1992) observed that sixty percent of the through vehicles in the lane is blocked due to two left turn per cycle.

3.5 Saturation Flow Estimation

Due to critical role of the saturation flow for intersection capacity, different techniques have been tried to calculate saturation flow for each country. As mentioned Chapter 2, there are different methods implemented in different areas but signalized intersection saturation flow estimation is mostly done with respect to HCM. While observing, different correction factors have been implemented in each country of the world. Calculations of saturation flow are done regarding these corrections. Most of the researchers have been concerning on fundamental adjustment factors by comparing the most widely accepted document HCM. Highway capacity manual is analyzing the saturation flow in a different way. One of them is that if site-specific saturation flow observations are not done, base saturation flow is offered by HCM as 1900 veh/h/ln. However, base saturation should be done according to site requirements. In this respect, site-specific situations are dealt and saturation flow calculation is done in this manner. Moreover, there is no need to use base saturation flow if site-specific studies are done. This is preferable for HCM because base saturation acceptance could be differentiated in many cases. There is an important point that is how to generalize headway. In this respect, HCM is offering the after 4th vehicle, headway of the vehicle reaches a constant level. Therefore, headway of cycle could be generalized by taking an average of the headway after 4th vehicle. Also, HBS (German Highway Capacity Manual) studies about the estimation of saturation flow are dealt. Unlike HCM, HBS is accepted base saturation flow as 2000 veh/h/ln and base headway as 1.8 second. Heavy vehicle correction factors are also different from HCM. All of the saturation flow estimations have been taken into account and results of them are compared with each other while analyzing observed area at Konya.

3.6 Inefficiency Estimation

HCM 2010 states that saturation flow rate can be determined directly from field measurement. This does not require any adjustment factors. Instead of factors, headway distributions of each cycle is taken into account. In this respect, observations for each lane cycle by cycle should be done as stated HCM (2010); however, time headway is corrected for each lane. While analyzing the data, headway of the fourth vehicle was subtracted from the last recorded vehicle in the cycle and average headway distribution of the cycle was observed for each cycle. Saturation flow calculations for each lane of the approach legs depending on the site observations was done as stated Equation 3.4 and 3.5

$$h_{avg,i}^{a,l} = \frac{h_i^{a,l} - h_{i,4}^{a,l}}{N_i^{a,l} - 4} \quad \text{......................................} \quad (3.4)$$

$$S_{S,veh} = \frac{3600}{h_{avg,i}^{a}}, \text{ (in veh/h/ln)} \quad \text{......................................} \quad (3.5)$$

where,

$S_{S,veh}$: Site based saturation flow rate (veh/h/ln)

$h^{a,l}_{avg,i}$: Average headway of the ith cycle at at *a* approach leg for lane *l*,

$h^{a,l}_{i,last}$: Last vehicle headway of the ith cycle at at *a* approach leg for lane *l*,

$h^{a,l}_{i,4}$: Fourth vehicle headway of the ith cycle at at *a* approach leg for lane *l*,

$N^{a,l}_{i}$: Number of the vehicle at ith cycle at *a* approach leg for lane *l*,

To compare all type of the saturation flow calculations, passenger car equivalency factors (pce) had been used as stated at TS6407. These factors were 2.25 for buses and 1.27 for minibusses. This had some effects the site based observations. Conversion of the vehicles had some effects on the site based headway calculations. Although the headway of the different type of vehicle did not change, it had some effects on a total number of the vehicles at cycle. Therefore, the increment of total number of the vehicle caused decreasing headway calculations of the cycle as stated Chapter 2, so increment in the saturation flow was observed. Cycle-based studies will be analyzed and comparison of the veh/h/ln and pce/h/ln observations will be shown to observe differences in the cycle. Studies were widened to all observed period for comparison of the veh/h/ln and pce/h/ln. Observation done by HCM and HBS used passenger equivalency factors. That is why, rest of the analysis maintained using equivalency factors to compare results of others. In this respect, site-based observations were converted to veh/h/ln to pce/h/ln. Calculations were done for each observed period and for each cycle of that period, so headway of vehicle was taken into account. This helped to find saturation flow for that vehicle at the cycle. However, if the vehicle type was different from passenger car, saturation flow estimations were done by using the equivalency factors as

$$S_{S,pce} = \frac{3600}{h^{a}_{avg,i}} f_{HV} . = S_{S,veh} . f_{HV} \text{, (in pce/hr/ln)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.5a)}$$

Site-based saturation flow calculations had been revised by making the moving average for each cycle. This helped to analyze the trend of the traffic flow each cycle, so general behavior at the cycle could be idealized in this way. Each cycle had been composed of the different type of the vehicle. Effects of one vehicle could be seen in the lead and former vehicles. Therefore, to evolve three vehicles effects on others, moving average had been implemented at the analyzed period. According to HCM, first four vehicles were eliminated at the site based observations. While calculating the headway calculations, observations were done based on the equations below. Average of the three vehicles was observed and the saturation flow of the three vehicles was stated for each lane as a veh/h/ln.

$$\hat{h}_i = \frac{h_i + h_{i-1} + h_{i-2}}{3}$$, i > 4, rank of the vehicle in the cycle…………………………… (3.6)

$$\tilde{Q}_{veh} = \frac{3600}{\hat{h}_i}$$, (in veh/h/ln) ….……..………………...………………..……..…...… (3.7)

where,

$\tilde{Q}_{veh}$: Saturation flow rate depending on moving average method (veh/h/ln)

$\hat{h}_i$: Moving average of headways of i^{th} rank of the cycle

In order to use moving average analysis, each of the vehicle types has been equivalent. That is why, passenger car equivalency (pce) has been stated at the TS6407 was used. Firstly, each of the vehicle types has been converted by using passenger car equivalency factors. Each three of the vehicle type equivalency factors was determined an average of them was taken. In this respect, saturation flow observation had been revised for making moving average with respect to passenger car equivalency factors. Calculations were done with respect to Equation 3.7a.

$$\tilde{Q}_{pce} = \frac{3600}{\hat{h}_i} f_{HV} = \tilde{Q}_{veh} \cdot f_{HV}$$ (in pce/h/ln) ……....…..….……..………………… (3.7a)

where,

$\tilde{Q}_{pce}$:Adjusted saturation flow rate depending on moving average method (pce/h/ln)

Although traffic flow at the effective green period is constant as stated in HCM, site-based observations demonstrate how it is fluctuating at each cycle period. In our observations supports this fluctuation at the each cycle period. However, this cycle based fluctuation at each observed period causes inefficiency at each cycle. Calculations depending on different methods are offering the saturation flow capacity of each lane for each cycle. Lane-based observations were done in the light of these guides. In our site-based observations, distribution of the saturation flows from highest to lowest was determined for each lane. Their percentile distributions were also done to compare with each other. In this respect, instead of using the calculations offered by guidelines, site-based most likely thresholds was tried to determine. Taking largest flow may cause overestimations for saturation flow, so 85 percentile of the each lane was taken into account because general distribution of the data was close to this range at each cycle period. In addition, this highest value and traffic flow at the cycle was studied closely. Comparison of the flow and 85 percentile of the cycle was done to analyze changing in the flow at cycle at used green period. This gave efficiency of the flow; however, efficiency of the flow could be differentiated for each lane. That is why, 85 percentile was changing due to lane based observations at site.

Inefficiency during the effective green in a phase in every approach was defined as “the total number of vehicles that could have passed, if flow were observed at the practical saturation rate, but did not”. The effective green period in a phase can be started at the fourth vehicle, $t^{l}_{i,4^{th}}$, and ended by the beginning of the clearance lost time at the end of the used green, $t^{l}_{i,e}$, in this study. The practical saturation flow rate is assumed as S_{85}, as defined above for each lane. End lost time for urbanized intersections was taken 2.5 seconds Section 2.3.1, which is also assumed for this analysis. To represent fluctuations during the used green, instantaneous saturation flow rates, $\tilde{Q}^{l}_{i\,j}$, were used with 3-moving average method for each vehicle j passing the intersection at time $t^{l}_{i,j}$.

Total inefficiency in a cycle *i* on a lane *l*, ϕ^{l}_{i}, can be defined as the summation of marginal inefficiency $\Delta\phi^{l}_{i,j}$ between the two consecutive instantaneous flow conditions, $\tilde{Q}^{l}_{i,j}$ and $\tilde{Q}^{l}_{i,j+1}$, as

$$\phi^{l}_{i} = \sum \Delta\phi^{l}_{i,j} \text{ , } j>3 \text{ ………………………………………………… (3.8)}$$

Marginal inefficiencies (see Figures 3.8) can be calculated based on the levels of $\tilde{Q}^{l}_{i,j}$ and $\tilde{Q}^{l}_{i,j+1}$ as follows:

Case 1: if $\tilde{Q}^{l}_{i,j+1}, \tilde{Q}^{l}_{i\,j} \leq S_{85}$,

$$\Delta\phi^{l}_{i,j} = \frac{1}{2}(\tilde{Q}^{l}_{i,j} + \tilde{Q}^{l}_{i,j+1} - 2S_{85})(t^{l}_{i,j+1} - t^{l}_{i,j}), \text{ ………………………………… (3.9a)}$$

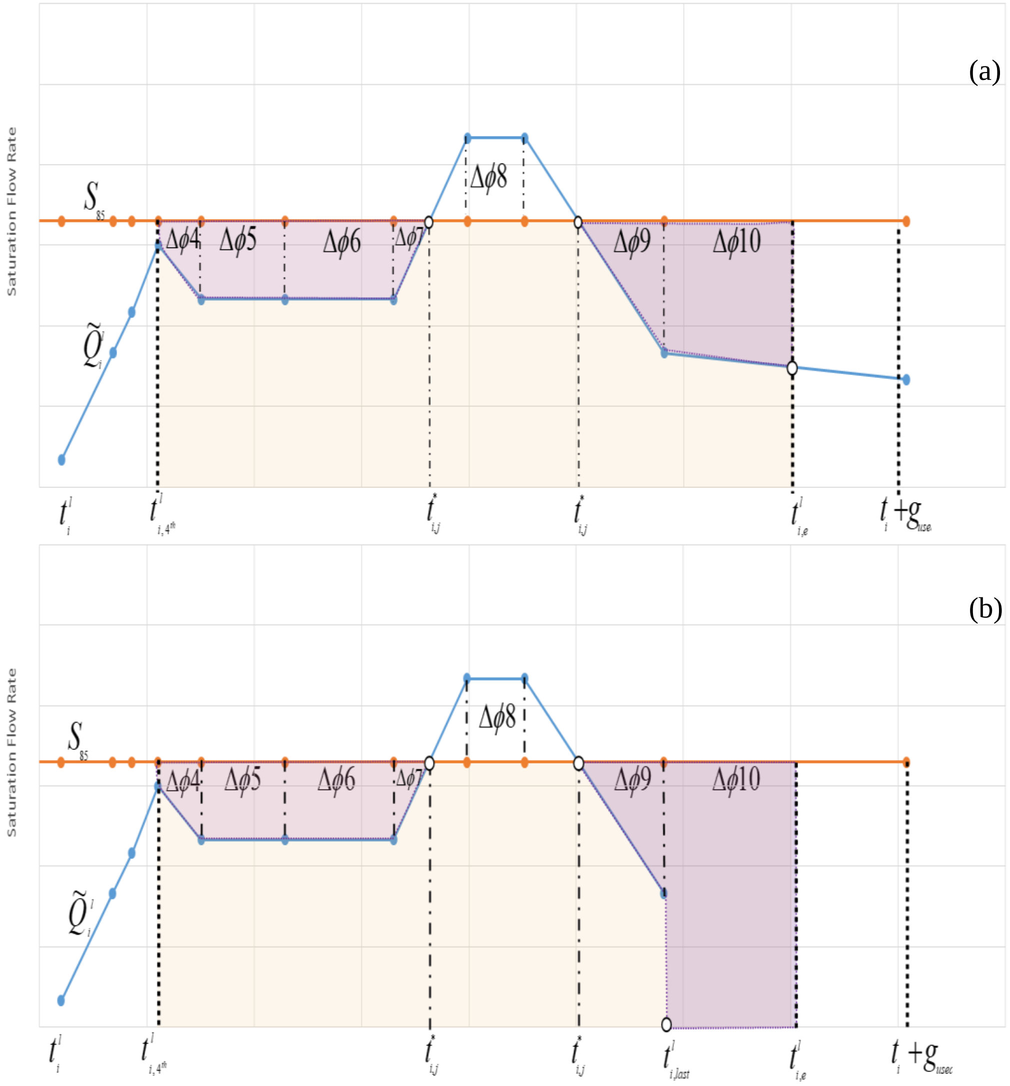

Figure 3. 8: Inefficiency during a cycle for a given lane and S_{85} limit

Case 2: If one of the instantaneous flow exceeds S_{85}, $\tilde{Q}^l_{i\,j+1}$ or $\tilde{Q}^l_{i\,j} > S_{85}$,

if $\tilde{Q}^l_{i\,j} < S_{85}$, $\Delta\phi^l_{i,j} = \frac{1}{2}(\tilde{Q}^l_{i\,j} - S^l_{85})(t^*_{i,j} - t^l_{i,j})$, ……………...…………………….. (3.9b)

if $\tilde{Q}^l_{i\,j+1} < S_{85}$, $\Delta\phi^l_{i,j} = \frac{1}{2}(\tilde{Q}^l_{i\,j+1} - S^l_{85})(t^*_j - t^l_{i,j+1})$, ….....……...…………………….. (3.9c)

Case 3: If $\tilde{Q}^l_{i\,j}, \tilde{Q}^l_{i\,j+1} > S^l_{85}$, no inefficiency is assumed.

There are special end conditions to consider.

Case 4: If end of effective green happens while the flow continues on a lane, $t^{l}_{i,e} < t^{l}_{i,last}$

$\Delta\phi^{l}_{i,j} = \frac{1}{2}(\tilde{Q}^{\,l}_{i,j} + Q^{\,l}_{i,e} - 2S^{l}_{85})(t^{l}_{j+1} - t^{l}_{j})$ → to be interpolated

Case 5: If flow of a lane ends earlier than the end of the used green, $t^{l}_{i,e}$ $(t^{l}_{i,last} < t^{l}_{i,e})$

$$\Delta\phi^{l}_{i} = S^{l}_{85}(t^{l}_{i,e} - t^{l}_{i,last}) \quad \text{(3.10)}$$

as shown in Figure 3.8b.

Major approach legs inefficiency calculations was done with respect to green used period and the results will be given at the Chapter 4. Due to breaking in the flow and not continuity of the flow for cycle, minor approach legs inefficiency was clearly determined, so there is no need to define inefficiency for minor approach legs.

CHAPTER 4

CAPACITY ANALYSIS OF APPROACH LEG

4.1 Evaluation of Capacity by Design Elements

Highway Capacity Manual (2010) uses saturation flow rate as 1900 veh/h/ln. If approach legs indicate lower speeds (less than 50 km/h), base saturation flow rates could be taken as 1,800 veh/h/ln. Although HCM states that saturation flow may be increased or decreased on the basis of local field measurements, calculations were done with respect to HCM base saturation flow rate as 1900 veh/h/ln. Moreover, HCM focuses on mean departure headway only. Departure headway at each lane demonstrates log-normal distribution (Xuexiang, et al., 2009). Also, Majeed, Zephaniah, Mehta, & Jones (2014) stated that 1.9 seconds recommended headway of HCM might not be the mean value at all intersections. Due to differences between driver behavior and vehicle types, headways are usually not constant. As stated observations of Teply, Allingham, Richardson, & Stephenson (2008), due to differences in those things and localities, it is useful for comparisons among regions and cities. That is why, most of the observations about intersection and roundabouts capacity have been based on method at HCM.

4.2 Signal Control Properties

Data was collected from study area at 24.02.2016. Surveys were conducted at morning, noon and evening peak periods (07.29 to 08:54, 10:59 to 12:30 and 16:59 to 18:30). Cycle length of observation was approximately 91 minutes at noon and evening period but morning observation period was 85 minutes. In order to analysis capacity of observed area, signal control properties of it are analyzed in Table 4.1. In this respect, 17709 vehicles was observed and most of them were seen at morning period as 6633 vehicles. 6067 of them is a private car and there were 441 minibuses and 125 buses at the observed area. At evening period, 5903 vehicles was using the roundabout. There were 5376 private cars; whereas, 441 minibuses and 86 bus were watched in the area. Also, 5173 vehicles used roundabout at noon period. 4812 private car, 291 minibuses and 70 buses used area at noon. Figure 4.2 and 4.3 are showing the total number of the vehicle based on lanes at major and minor approach legs. Number of the vehicle at noon observation period is lower than others. Also, minor approach legs have lower number of vehicle and it has no vehicle at Lane1. The main reason is that number vehicle using Lane 1 at minor approach legs turn right by using the bypass lane. Most of the car at observed area have taken a position at Lane 2 and Lane 3 but some of noon observed data includes vehicle coming at observed area on Lane 1.

Table 4.1: Descriptive statistics of intersection count data

	AM		NOON		PM	
	Major A2 & A4	Minor A1 & A3	Major A2 & A4	Minor A1 & A3	Major A2 & A4	Minor A1 & A3
Observation Period	07:29-08:54	07:29-08:54	10:59-12:30	10:59-12:30	16:59-18:30	16:59-18:30
Total Observation Duration	85 min	85 min	91 min	91 min	91 min	91 min
No. of vehicles	5638	995	4484	689	5097	806
No. of PC	*5166*	*901*	*4183*	*629*	*4639*	*737*
No. of MB	*363*	*78*	*239*	*52*	*378*	*63*
No. of B	*109*	*16*	*62*	*8*	*80*	*6*
No. of vehicles by Lane (veh)						
Ln 1 (A1/A2)	*794*	---	*590*	---	*1074*	---
Ln 1 (A3/A4)	*810*	---	*524*	*79*	*762*	---
Ln 2 (A1/A2)	*904*	*271*	*773*	*152*	*770*	*303*
Ln 2 (A3/A4)	*1228*	*278*	*948*	*152*	*1108*	*163*
Ln 3 (A1/A2)	*730*	*193*	*722*	*185*	*593*	*209*
Ln 3 (A3/A4)	*1172*	*253*	*927*	*121*	*790*	*131*
Avg. Flow Rates (veh/h)						
Q_{avg}	3979.8	702.4	2758.5	454.3	3360.7	531.4
Avg. Flow Rates by Lane (veh/h/ln)						
$Q_{avg}^{A,1}$ *(A1/A2)*	*560.5*	---	*389.0*	---	*708.1*	---
$Q_{avg}^{A,1}$ *(A3/A4)*	*571.8*	---	*345.5*	*52.1*	*502.4*	---
$Q_{avg}^{A,2}$ *(A1/A2)*	*638.1*	*191.3*	*509.7*	*100.2*	*507.7*	*199.8*
$Q_{avg}^{A,2}$ *(A3/A4)*	*866.8*	*196.2*	*625.1*	*100.2*	*730.5*	*107.5*
$Q_{avg}^{A,3}$ *(A1/A2)*	*515.3*	*136.2*	*476.0*	*122.0*	*391.0*	*137.8*
$Q_{avg}^{A,3}$ *(A3/A4)*	*827.3*	*178.6*	*611.2*	*79.8*	*520.9*	*86.4*

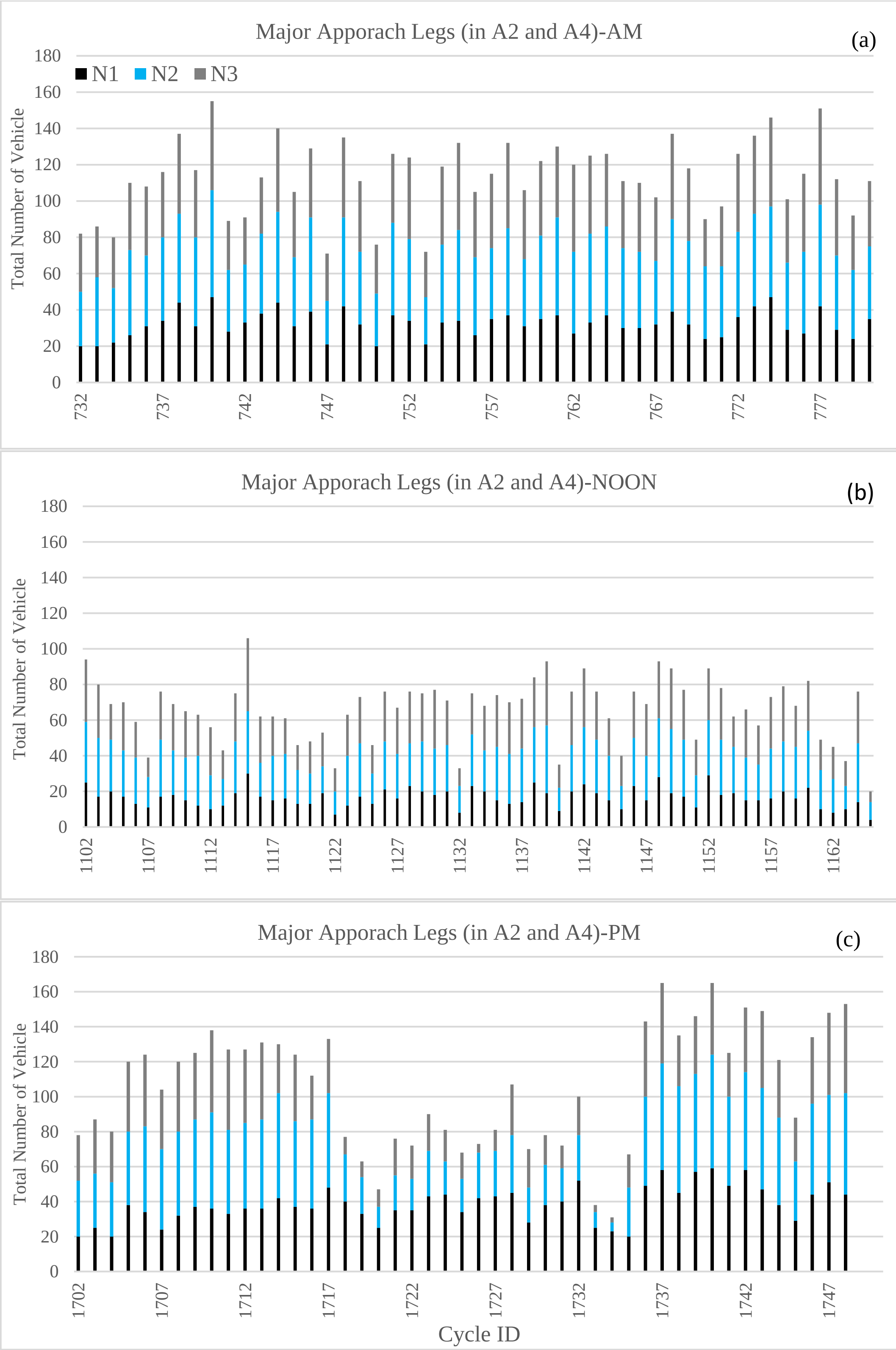

Figure 4.1: Total number of vehicle & Cycle ID at major approach legs (a) AM (b) NOON (c) PM

Figure 4.2: Total number of vehicle & Cycle ID at minor approach legs (a) AM (b) NOON (c) PM

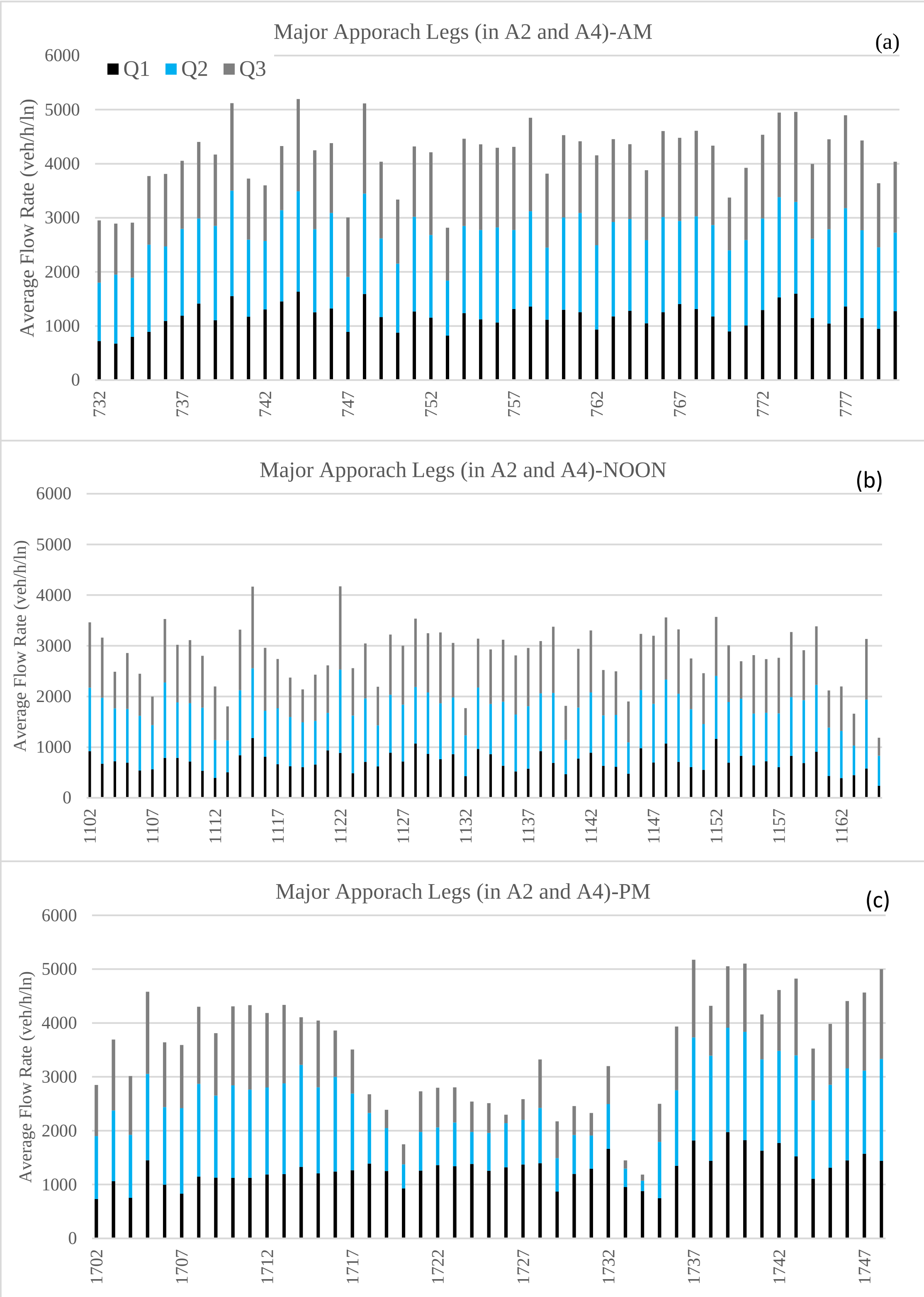

Figure 4.3: Average flow rate & Cycle ID at major approach legs (a) AM (b) NOON (c) PM

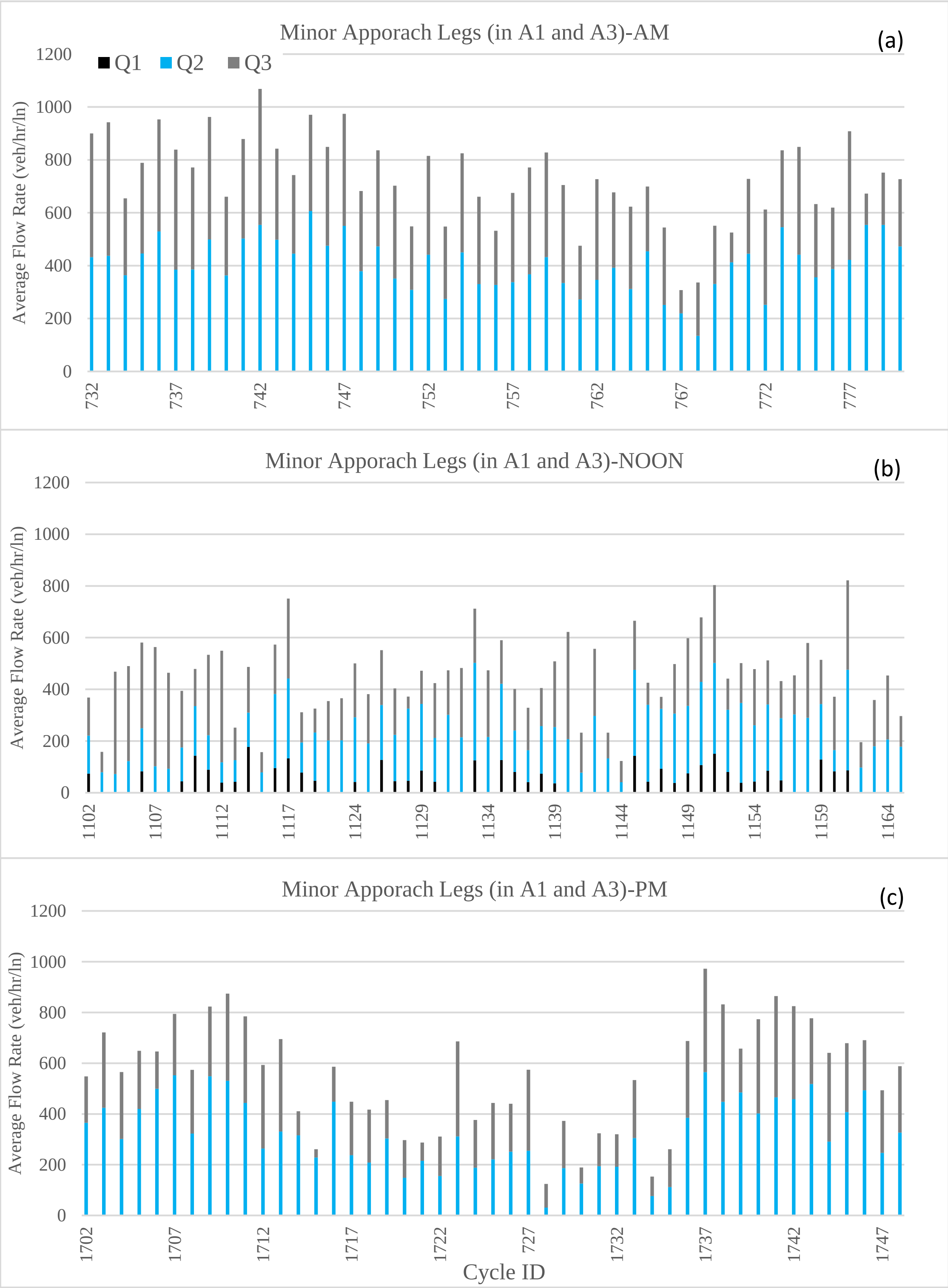

Figure 4.4: Average flow rate & Cycle ID at minor approach legs a) AM b) NOON c) PM

4.3 Probability Distribution of Headway

As different traffic regimes on different approach legs may represent different characteristics, these statistics were prepared for three study periods and two approach directions (majors versus minors) separately as shown in Table 4.2. Percentile distribution of the data is shown in box plot at Figure 4.5. In this figure, data distribution with respect to major and minor sites is shown a different color. Green on the box plot shows minor site approach legs; whereas; red indicates major legs. Major sites data distribution is mostly close together. This means data more condensed than minor. However, minor sites approach legs indicates the wider range in the values. This puts some evidence about how data spread out.

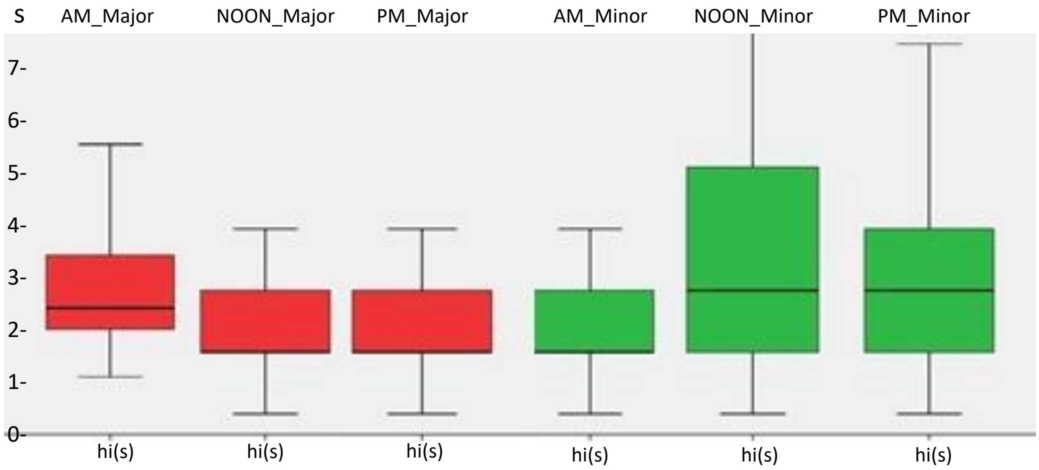

Figure 4.5: Box plot for headway of approach legs

In all time periods, the minimum observed headway was around 1 second, while the maximum headway values ranged between 17-30 seconds during morning and evening peaks. However, a very long headway of 63 seconds was observed only on the minor approach legs during the noon off-peak period only. The median value of the headways as shown as Q2 is differentiated between 2 to 3 second. This means data 50^{th}-percentile of the data is less than 3 second and is locating between in this range. Moreover, 75^{th}-percentile of the data is approximately close 3 seconds. Although minor approach leg observation of noon and pm observation is higher than 3 seconds (values are 5 and 4 second respectively), most of the data consisting

75th-percentile of the headways values is taking position 3 to 5 second. Also, difference between 75th and 25th-percentile of the data called interquartile range is differentiated 1 to 2 second. Like in the Q3 and median values of the headway, IQR is higher at noon and pm peak period observation. Low value of the IQR is showing low dispersion of the data. In this respect, data is differentiated is in low range generally; otherwise, IQR value became higher in noon and pm minor site observation. That is why, finding the identical headway value in small range is a bit easy while comparing the higher range.

Table 4.2: Descriptive statistics of headway, h_i, data for different time periods and approach legs

h_i (s)	AM		NOON		PM	
	Major	Minor	Major	Minor	Major	Minor
Q0 (minimum)	0.7	1.0	1.0	1.0	1.0	1.0
Q1 (25-percentile)	1.6	2.0	2.0	2.0	2.0	2.0
Q2 (median)	2.6	2.0	2.0	3.0	2.0	3.0
Q3 (75-percentile)	3.0	3.0	3.0	5.0	3.0	4.0
$h_{0.85}$ (85-percentile)	4.0	4.0	5.0	7.0	3.0	5.0
Q4 (maximum)	29	22	29	63	30	17
IQR= Q3-Q1	1.4	1.0	1.0	3.0	1.0	2.0
LU1 (Outlier limit)	5.1	4.5	4.5	9.5	4.5	7.0
LU2 (Extreme outlier limit)	7.2	6.0	6.0	14.0	6.0	10.0

Both normal and log normal distributions are simplifying data distribution. Neither of them is better than others. They only describe statistical behavior of the data. Table 4.3 compares normal and log-normal distribution of the data. According to Shao & Liu (2012) analysis, they stated that according to the statistical analysis result of the surveyed queue discharge headway data, it is revealed that the sample average value and queue discharge headway distribution is likely unsymmetrical. Like their observations, study area headway distribution has lognormal statistical distribution. The headway variable show more log-normal distribution properties than a normal distribution. In order to analyze the results of the statistical data distribution Q-Q plot distributions are shown in the Appendix B. Each of the Q-Q plot observation is done for each peak period observation at each approach directions. Headway distribution of all major and minor directions is a bit far away from the diagonal line in normal statistical distribution. However, lognormal distribution of the headway is closer to the straight line than a normal distribution. Distribution of the data idealize that headway data at each peak periods and each direction of the approach has statistically lognormal distribution. In this respect, statistical distribution of the data was accepted as log-normal, so data was truncated by using the log normal distribution priorities. However, while making this, some of the extreme data was eliminated with respect to upper limit observations as mentioned at Section 3.4.1. Therefore, data was evolved again as shown the Table 4.3 at the truncated data results section. This data clean-up process affects result of the headway. Maximum headway data was measured as 3.2 seconds; whereas, it was 2.6 second at same major approach leg and observation period. When truncating the data, headway was observed as 2.5 seconds. Also, the result of the minor approach legs was reached 4.2 seconds at normal distribution and 3.4 was observed at log normal distribution of the data. However, data clean-up process leads headway data to decrease to 3.3 seconds. Truncation results are less than other distributions. The main point is that some of the extreme data at the observation, which are affecting the results, were eliminated. In this respect, average flow calculation results were shown in the table below with respect to truncated data.

Table 4.3: Descriptive statistics behavior of headway, hi, data for different time periods and approach legs

	AM		NOON		PM	
	Major	Minor	Major	Minor	Major	Minor
Assuming normal distribution						
$\bar{h}_i$ (s)	2.7	3.0	3.2	4.2	2.6	3.4
σ_h (s)	2.2	2.0	2.8	4.2	2.0	2.3
LU3 (95% upper confidence limit)	7.0	6.9	8.7	12.4	6.5	7.9
Assuming log-normal distribution $\ln(h_i)$ Raw data results						
$\bar{h}_i$ (s)	2.2	2.6	2.6	3.4	2.2	2.9
σ_h (s)	1.7	1.6	1.9	1.8	1.7	1.7
LU3a (95% upper confidence limit)	6.5	6.8	8.7	10.9	5.9	8.0
Truncated data results						
$\bar{h}_i$ (s)	2.2	2.6	2.5	3.3	2.2	2.8
σ_h (s)	1.7	1.6	1.8	1.8	1.6	1.6
LU3b (95% upper confidence limit)	6.1	6.6	7.9	9.8	5.6	7.5
Average Flow, $\hat{Q}$ (veh/h/ln)	1636	1385	1440	1090	1636	1286

4.3.1 Rank Effect on Headway Distribution

Most of the observations have been accepting constant headway distribution after the fourth queuing vehicle in the cycle as stated in HCM. However, observation done by Lin & Thomas (2005) provided that after the fifteenth vehicle in cycle, it is not reached the constant headway. In order to idealize study area, box plot was used to display the distribution of data based on the five number summary: minimum, first quartile, median, third quartile, and maximum. Figure 4.6 ~Figure 4.9 show percentile distribution of the data for each approach leg at each observation periods. While showing the data extreme outliers were eliminated. Headway distribution up to the fourteenth vehicle in the queue is shown in the figures. In this respect, major approach legs (A2 and A4) headway data distribution was so compensated than minors. The range of each quartile at minor legs is higher, so this shows how time headway was bigger than major legs. According to results, noon period observations is different from others. Quartiles are far away from them.

Although data distribution estimation at major legs was estimated easy, this was not mentioned at minor approach legs. That is why, different box plot range and distributions could be seen at minor legs. Some of the cycles does not contain one sample, so the line of relevant figures at minor legs is due to sample size.

Although HCM is accepting that the headway distribution is constant after the fourth vehicle, site-based observations did not support this idea in this study. To analyze this, two samples from different observation period was chosen at major approach legs. Observations on this topic were mostly done at major approach legs because there were many outliers and failures of headway at minor sites. Studies were done at each lane to reflect the observations at headway with respecting queue position of the vehicle at cycle. While generating the results, the first vehicle at each lane was eliminated. Result given in Figure 4.6 about A2 was chosen from evening observed cycle 1705 and cycle observation period was 17:06:43-17:07:29. The figure shows that Lane 2 and Lane 3 headway distribution have a tendency to become a constant after 4^{th} vehicle but all of the lane at evening observed period cycle, 1705. Like HCM, headway distribution was decreased after the first vehicle at the noon period of the 1111 cycle of A4. Although each headway of the lane does not reach the constant level, 8^{th} position of vehicle at lane 2 and 3 indicate similar data distribution at 1111 cycle headway. Whereas, this types of the results could not be reached at follower cycles. In this respect, 1110 cycle was specially chosen to reflect how headway could be change at cycle based on the rank of vehicle after observing 1111 cycle. There were many fluctuations at the headway at each lane. Like noon observation, evening observation was chosen with respect to former observations. However, headway distribution according to rank based become hard to accept as former observations. There were many outliers and jumping of the data make difficult data to behave constantly. Results of them are shown in Figure 4.10 and 4.11.

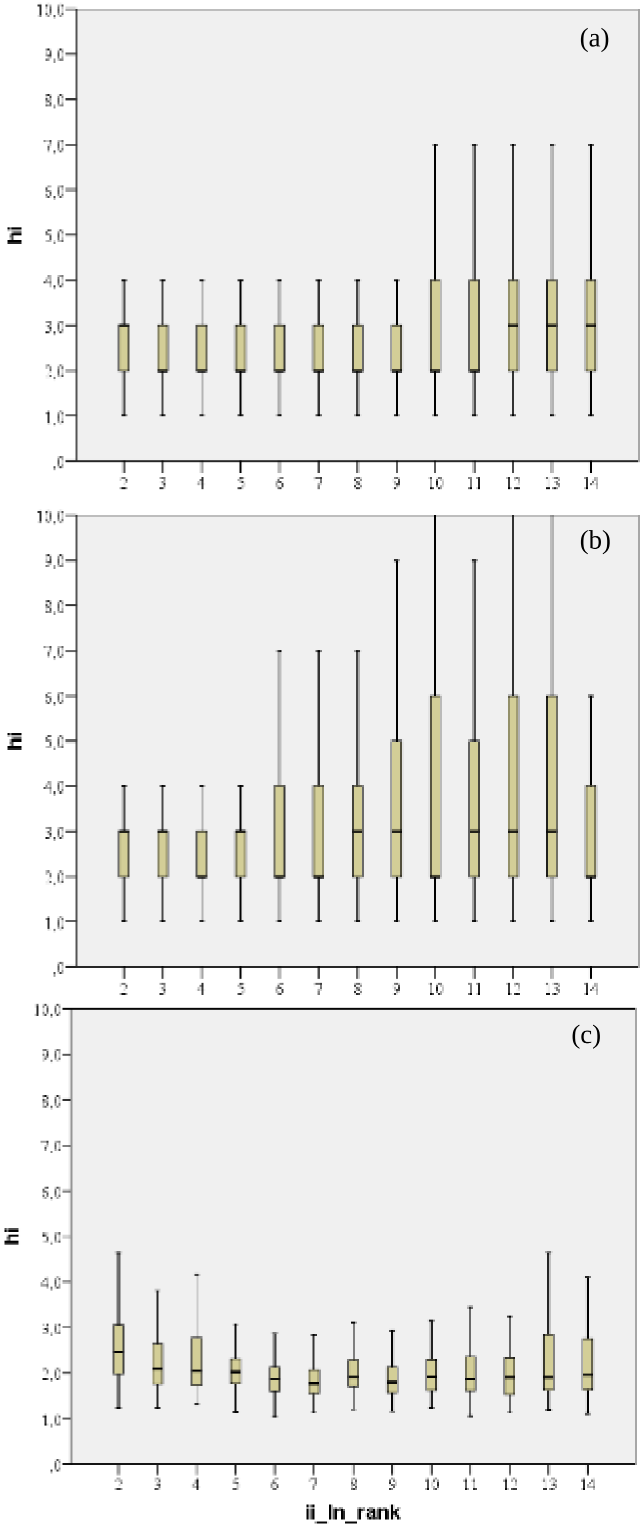

Figure 4.6: Rank & Headway distribution of A2 (a) AM (b) NOON (c) PM

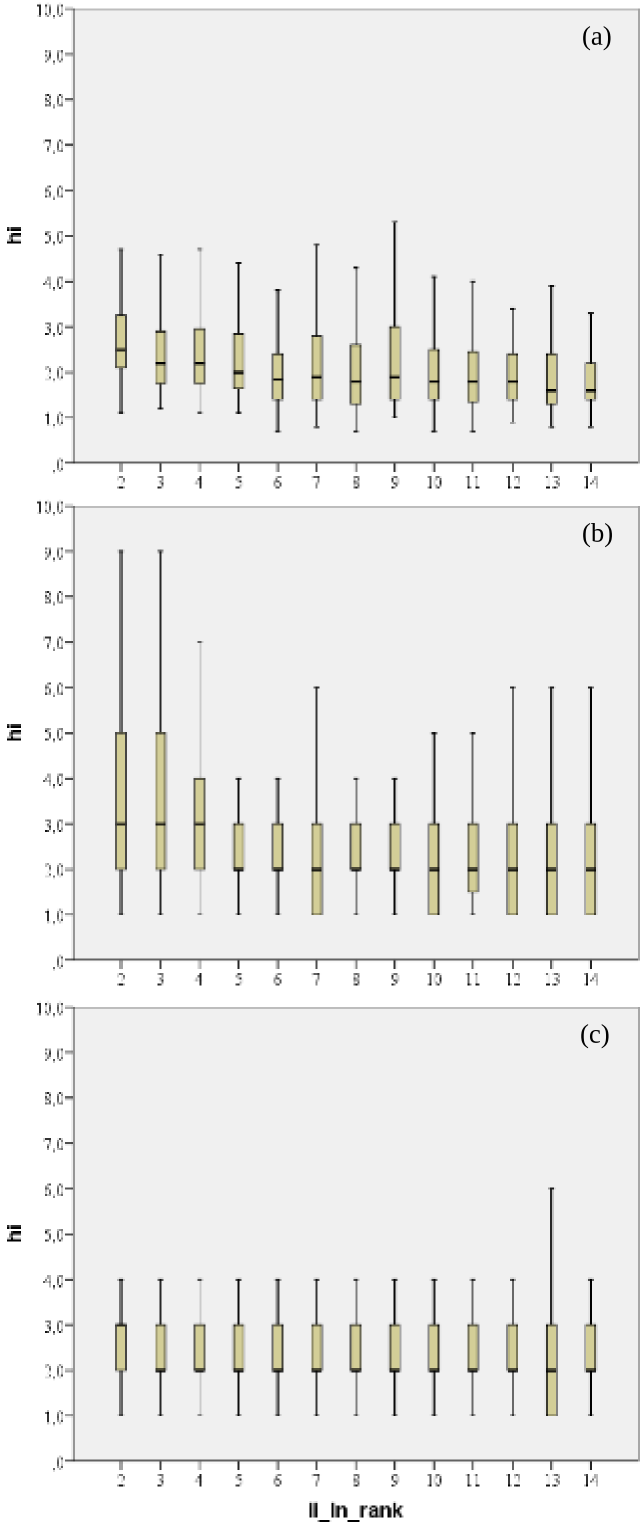

Figure 4.7: Rank & Headway distribution of A4 (a) AM (b) NOON (c) PM

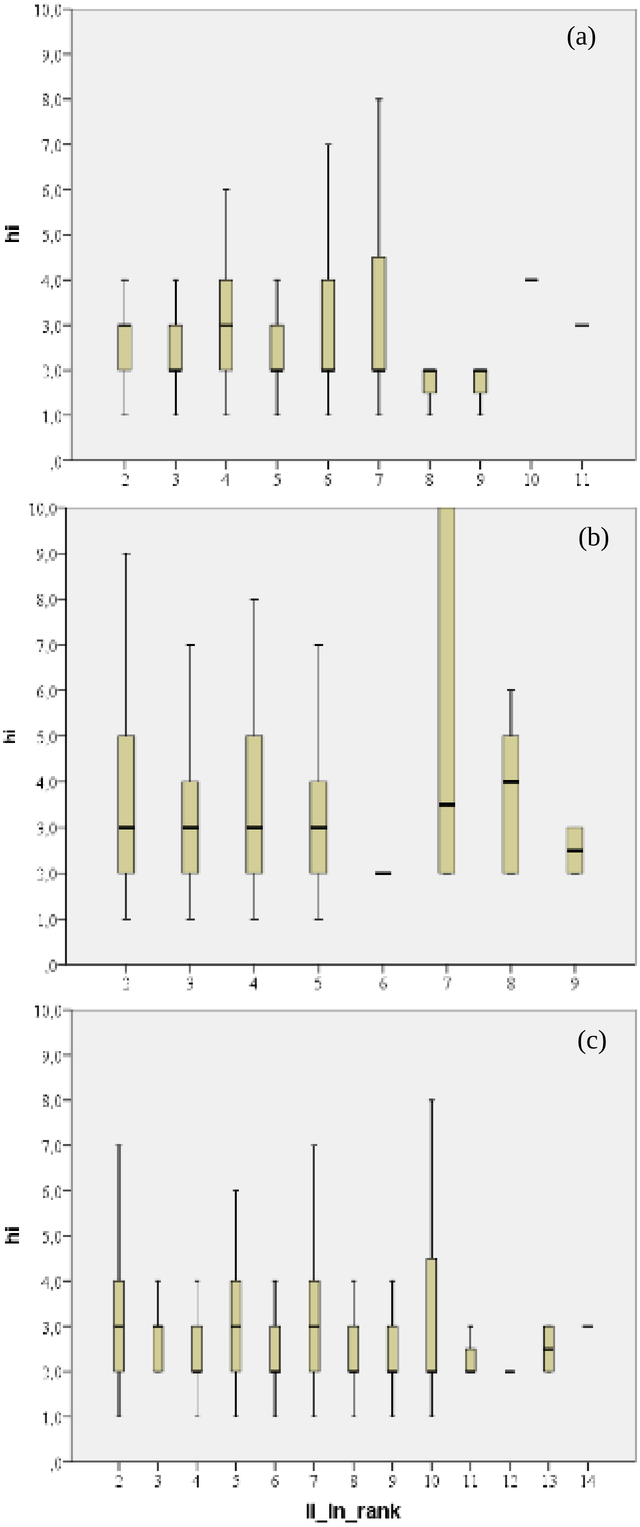

Figure 4.8: Rank & Headway distribution of A1 (a) AM (b) NOON (c) PM

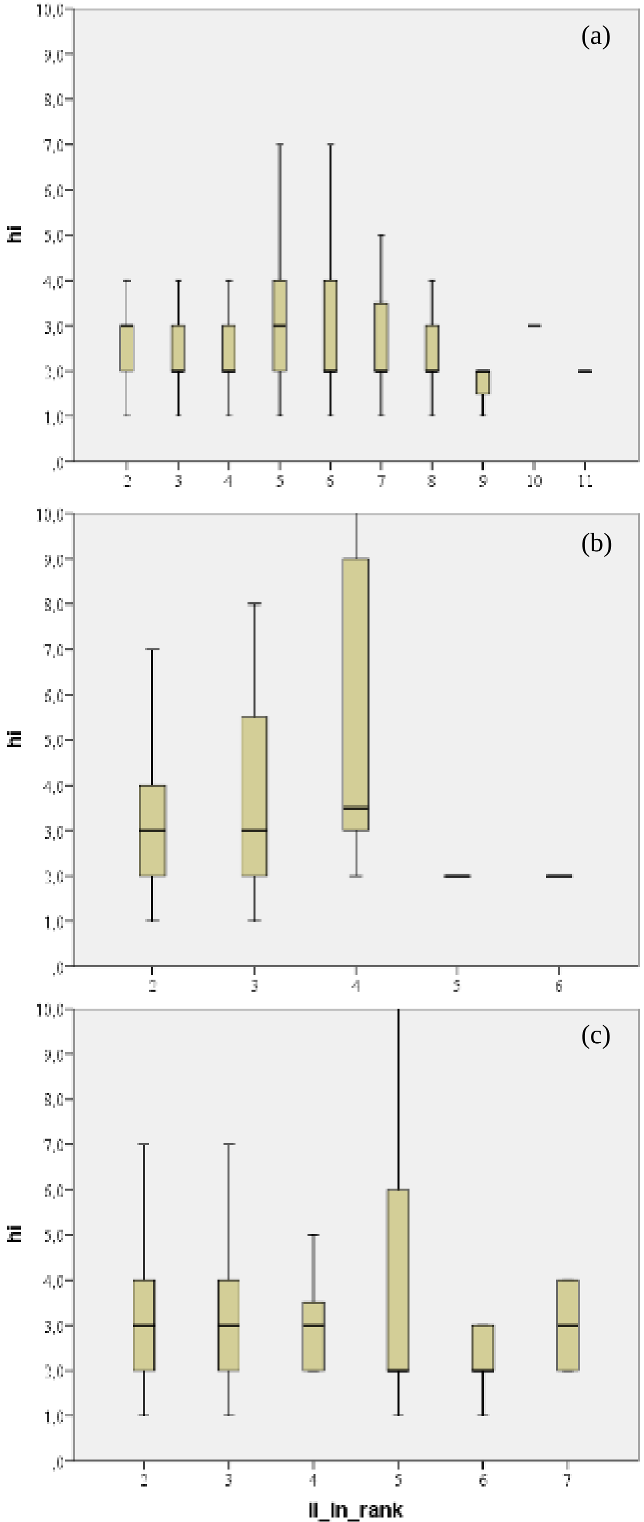

Figure 4.9: Rank & Headway distribution of A3 (a) AM (b) NOON (c) PM

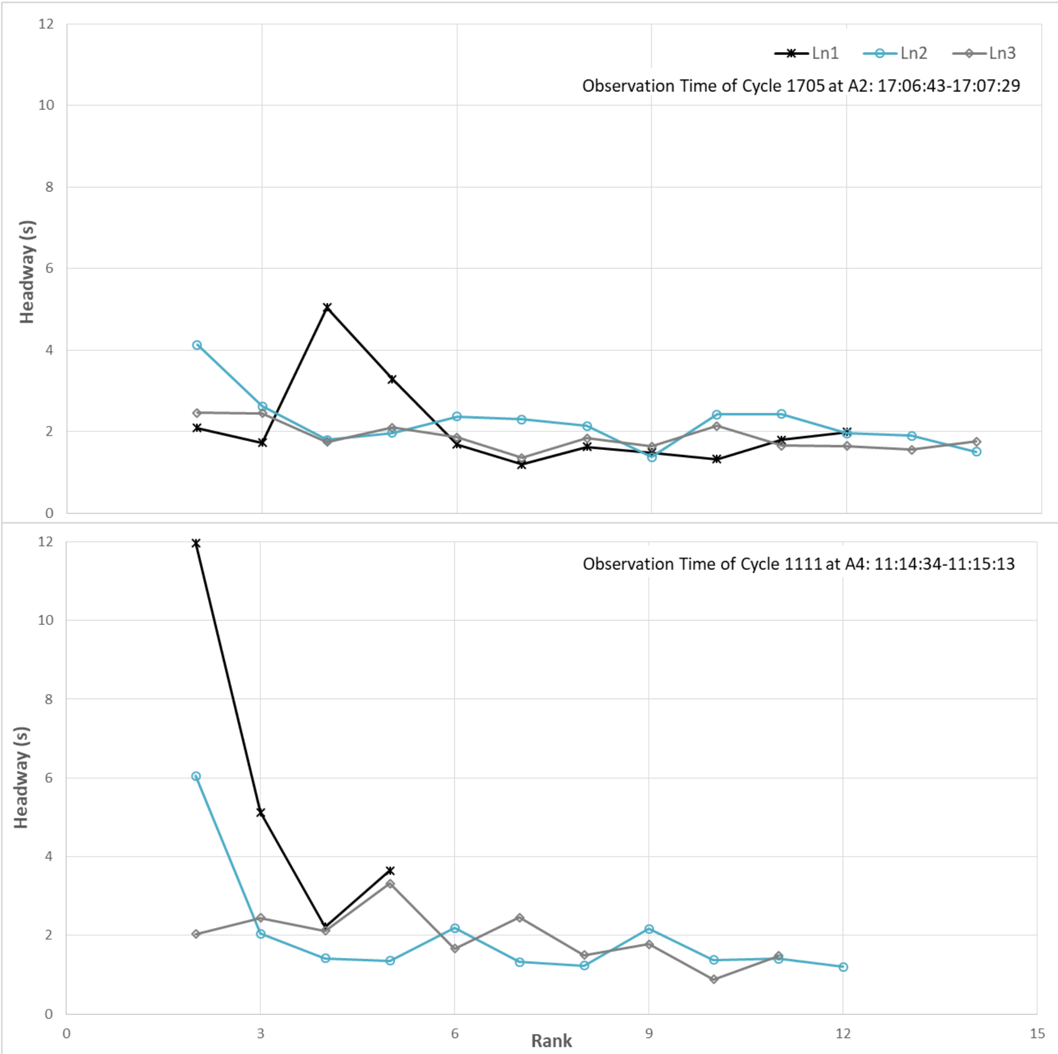

Figure 4.10: Headway & rank in cycle similar to HCM

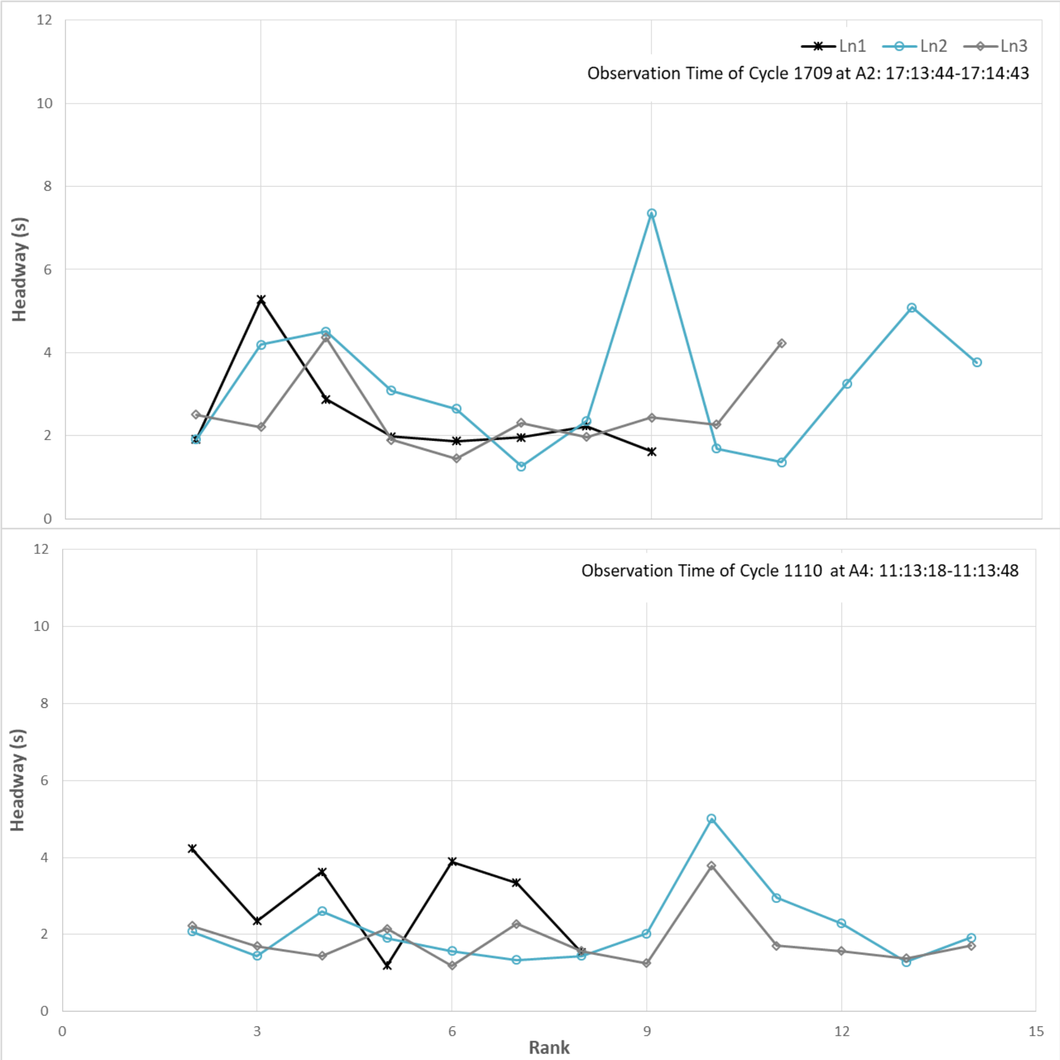

Figure 4.11: Headway & rank in cycle non-similar with HCM

4.3.2 Vehicle Type Effect on Headway Distribution

Headway distribution was analyzed with respect to type of vehicle. While observing its effects on headway distribution, log-normal studies had been done. Table 4.4 shows the result of this study. Percentile distribution and interquartile range are shown at the table. In this respect, results of the private car are so dominant that result of it is close to results given in Table 4.2. Also, minimum observed values for minibus was mostly around 1.0; however, minimum values for the bus was differentiated 0.8 to 2 second and it reached 3.0 second at minor legs pm peak period observation. Whereas, the maximum value for minibus was ranged between 7 and 29 second and for bus, maximum value differentiated 7 to 20 second.

Table 4.4: Summary of headways by vehicle types (Private car, minibus and bus) for different approach legs and periods

	Major	Minor
	AM	
Q0 (s)	(0.7;0.7;0.8)	(1.0;1.0;1.0)
Q1 (s)	(1.6;1.6;1.7)	(2.0;2.0;1.8)
Q2 (s)	(2.0;2.0;2.1)	(2.0;3.0;2.5)
Q3 (s)	(3.0;3.0;3.0)	(3.0;3.3;4.5)
Q4 (s)	(25.0;29.0;20.0)	(22.0;12.0;12.0)
IQR (s)	(1.4;1.4;1.3)	(1.0;1.3;2.8)
$h_{0.85}$	(4.0;4.0;4.0)	(4.0;4.6;6.8)
	NOON	
Q0 (s)	(1.0;1.0;1.0)	(1.0;1.0;2.0)
Q1 (s)	(2.0;2.0;2.0)	(2.0;2.0;2.5)
Q2 (s)	(2.0;2.0;2.0)	(3.0;3.0;3.0)
Q3 (s)	(3.0;4.0;3.0)	(5.0;6.5;7.0)
Q4 (s)	(29.0;21.0;12.0)	(63.0;.14.0;10.0)
IQR (s)	(1.0;2.0;1.0)	(3.0;4.5;4.5)
$h_{0.85}$	(5.0;5.0;4.0)	(6.0;10.0;---)
	PM	
Q0 (s)	(1.0;1.0;1.0)	(1.0;1.0;3.0)
Q1 (s)	(2.0;2.0;2.0)	(2.0;2.0;4.0)
Q2 (s)	(2.0;2.0;2.0)	(3.0;2.0;6.0)
Q3 (s)	(3.0;3.0;3.0)	(4.0;4.0;6.5)
Q4 (s)	(30.0;16.0;9.0)	(17.0;7.0;7.0)
IQR (s)	(1.0;1.0;1.0)	(2.0;2.0;2.5)
$h_{0.85}$	(3.0;3.0;3.0)	(5.0;4.0;---)

Median values of the minibus were 2.0 but at AM and NOON minor approach legs observations, the median of the minibus was 3.0 second. Q2 median value for bus differentiated from 2.0 to 3.0 second but at evening minor approach leg observation was reached to 6.0 second. 75^{th}-percentile of data distribution of both minibus and bus was similar and values were ranged between 3.0 and 6.5 seconds. Data of the bus was reached to 7.0 second at NOON period on minor legs. IQR of the minibus was ranged 1.3 to 2.0 seconds like IQR of bus but both of them was reached 4.5 seconds at minor approach legs observation at noon. Moreover, to idealize the distribution of the data, 85 percentile of the sample given at the table. In this respect, there was no data for bus in that range.

Table 4.5 gives an idea about the headway distributions regarding vehicle types. Mean values of the headway data and their standard deviations were observed. In this respect, average headway of private car was 2.2 second and at NOON it was 2.6 second. Value of it at minor approach legs was ranged between 2.6 to 3.3 second. The average flow rate is 1636 veh/h at morning and evening major approach legs. Also, minibus average headway was differentiated from 2.2 to 2.8 second at major legs, but it was around 2.7 to 3.6 second at minor sites. In this respect, critical average flow density was observed at evening period as 1636 veh/h. Bus average headway distribution, which is seen in Table 4.5, was also around 2.1 to 2.3 second. Minor sites observation was reached 5.2 seconds for bus. Average flow was critical at evening period observations which were 1714 veh/h. According to results of observations, private car average headway was closer to roundabout headway characteristics and critical density could be reached at pm major approach leg from if vehicle type was a bus.

Table 4.5: Comparison of general headway regarding vehicle types

		AM		NOON		PM	
		Major	Minor	Major	Minor	Major	Minor
Private Car	$\bar{h}_{PC}$ (s)	2.2	2.6	2.6	3.3	2.2	2.9
	σ_{PC} (s)	1.7	1.6	1.9	1.8	1.7	1.7
	$\hat{Q}_{PC}$ (veh/h)	1636	1385	1385	1091	1636	1241
Minibus	$\bar{h}_{MB}$ (s)	2.3	2.7	2.8	3.6	2.2	2.7
	σ_{MB} (s)	1.8	1.7	1.9	2.0	1.6	1.5
	$\hat{Q}_{MB}$ (veh/h)	1565	1333	1286	1000	1636	1333
	On-site PCE rates	1.05	1.04	1.08	1.09	1.00	0.93
Bus	$\bar{h}_{B}$ (s)	2.3	2.7	2.3	3.7	2.1	5.2
	σ_{B} (s)	1.7	2.1	1.8	1.8	1.6	1.4
	$\hat{Q}_{B}$ (veh/h)	1565	1333	1565	973	1714	693
	On-site PCE rates	1.05	1.04	0.88	1.12	0.95	1.79

Data distribution of the headway of each vehicle types was listed. Average flow distribution was calculated and results tabulated in Table 4.5. Although there is some difference about average headway of private car, minibus and bus, it could be accepted as same at major approach legs. Like major legs, private car and minibus have lower flow results at noon observation period of minor approach legs. Also, higher flow capacity for was reached at am minor observation period for each vehicle types. However, bus has lower capacity shows as 693 veh/h; whereas, minibus and bus were 1333 and 1241 veh/h respectively. In the light of this information, there is no significant effect on urban flow between private car, minibus and bus.

4.3.3 Lane Effects on Headway

Lane-based average flow calculations for each lane was done after truncated observations on headway. Although Lane 1 was mostly preferred for right turning activities, lane 2 and 3 was chosen through and left turning movement. The average flow rate of each lane was shown in the Table 4.6. This affected flow of each lane, so lowest values were observed at Lane 1 for all approach legs. While analyzing the major approach legs A2 and A4, highest average flow results were seen at Lane 2 at evening period as 1800 veh/h/ln; whereas, the flow rate at Lane 1 and Lane 3 was 1565 veh/h/ln at this period. Also, the flow rate at NOON is lower than other period and maximum flow was observed at Lane 3 as 1586 veh/h/ln. The lower flow rate was observed at lane 2 and 1 respectively as 1500 and 1186 veh/h/ln. Although each lane demonstrated highest flow rate at evening period, lane 3 reached at highest at AM as 1714 veh/h/ln which Lane 2 average flow rate was. However, as it is expected, flow at minor legs was lower than majors. The highest average flow rate for minor approach legs was observed at Lane 2 and Lane 3 at AM as 1385 veh/h/ln. During PM, Lane 2 average flow rate was 1286; whereas, flow at Lane 3 was 1241 veh/h/ln. Although flow at Lane 1 at minor legs was not observed at PM periods, it had 1059 veh/h/ln at NOON. This level of flow was higher from flow at Lane 2, 1029 veh/h/ln. The highest value was observed at NOON at Lane 3 as 1200 veh/h/ln.

Table 4.6: Headway characteristics by lane

		AM		NOON		PM	
		Major	Minor	Major	Minor	Major	Minor
Lane 1	$\bar{h}_1$ (s)	2.6	---	3.1	3.4	2.3	---
	σ_1 (s)	1.7	---	1.8	1.6	1.6	---
	$\hat{Q}_1$ (veh/h/ln)	1385	---	1161	1059	1565	---
Lane 2	$\bar{h}_2$ (s)	2.1	2.6	2.4	3.5	2.0	2.8
	σ_2 (s)	1.7	1.6	1.8	1.8	1.6	1.6
	$\hat{Q}_2$ (veh/h/ln)	1714	1385	1500	1029	1800	1286
Lane 3	$\bar{h}_3$ (s)	2.1	2.6	2.3	3.0	2.3	2.9
	σ_3 (s)	1.6	1.5	1.8	1.7	1.6	1.6
	$\hat{Q}_3$ (veh/h/ln)	1714	1385	1565	1200	1565	1241

4.4 Capacity Estimation of Roundabout Approaches

Saturation flow rate estimation was done with respect to HCM method as mentioned in Chapter 2. Measurements have been done cycle by cycle. In this respect, one of the saturation flow estimation with respect to time of the cycle duration is shown in the Figure 4.12. It represents observation at morning period at Approach 4 and cycle number is 739. Calculations were done for each lane to see the how saturation flow is differentiated on the cycle for each lane. The first four vehicles at the cycle was eliminated while calculations site based saturation flow calculations as mentioned at HCM. The point on each lane shows observed data at the cycle. Point on the Lane 2 is so close than other lanes. A number of the vehicle was higher at lane 2 and 3 than Lane1. However, others lanes did not reach the same capacity as Lane 2. Some of the headway of the data is less than 2 seconds push saturation flow rate to reach to 2500 veh/h/ln at that lane. There were some gaps between vehicles at lane 1 and 3, so saturation flow at lanes closed to 500 veh/h/ln. In order to analyze the effect of the vehicle types on saturation flow distribution, passenger car equivalency was implemented.

Passenger car equivalency factors were accepted as stated TS6407. To see its effects, Figure 4.12 also revised as pce/h/ln. General distribution of the saturation flow at cycle did not change because cycle mostly is consisting of private car. This situation is acceptable for the whole of the observation period and each approach legs. Figure 4.12 represents a comparison of the veh/h/ln and pce/h/ln. This gives an idea of that saturation flow rate of the private vehicle leads the general of the cycle. Traffic flow at major approach legs was continued; whereas, there was a gap at minor approaches. This caused lack of continuity at the graphics. Also, a number of the vehicles at minors was not as same as major legs. After eliminating the first four vehicle, some of the cycle at minor legs were not evaluated at graphics due to not enough number of the vehicle.

Data distribution based on each cycle was shown within the observed period duration. In the light of them, Figure 4.13 represents the arrangement of the saturation flow rate for each approach legs from highest to lowest. A number of the cycle is higher than another observed period. Cycle duration at noon is higher than other periods. At Approach 2, saturation flow was close to 2500 pce/h/ln but this threshold was exceeded at Approach 4. Top twenty of saturation flow rank was around 1000 pce/h/ln. Also, lowest of saturation flow was reached at Lane 1 at Approach 4. This is commonly possible for Approach 2 but NOON period observations at Lane1 was higher than other Lane 1 observations at Approach 2. Moreover, highest observations at minor approach legs gave approximately same results with major sites. However, after a highest point, decrease rate of minor legs was fast. Although saturation flow was reached to 1000 pce/h/ln at top five data, this limit was reached top 20 to 30 at major legs. Separation of the cycle also was high at minor legs; whereas, saturation flow rate distribution at major legs was close each others. Moreover, a number of saturation flow rates was lower at minor than majors approach legs. There was 50, 65 and 48 cycles at observation periods, AM, NOON and PM respectively. Although the range of the saturation flow was reached to these numbers at majors, 30 or 35 data was observed at minor legs.

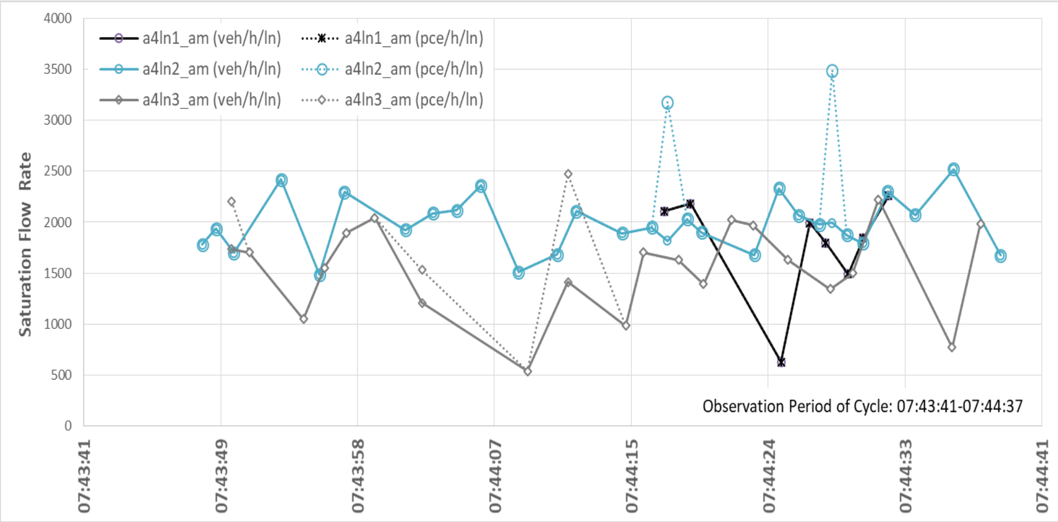

Figure 4.12: Saturation flow rate & 739 cycle at A4

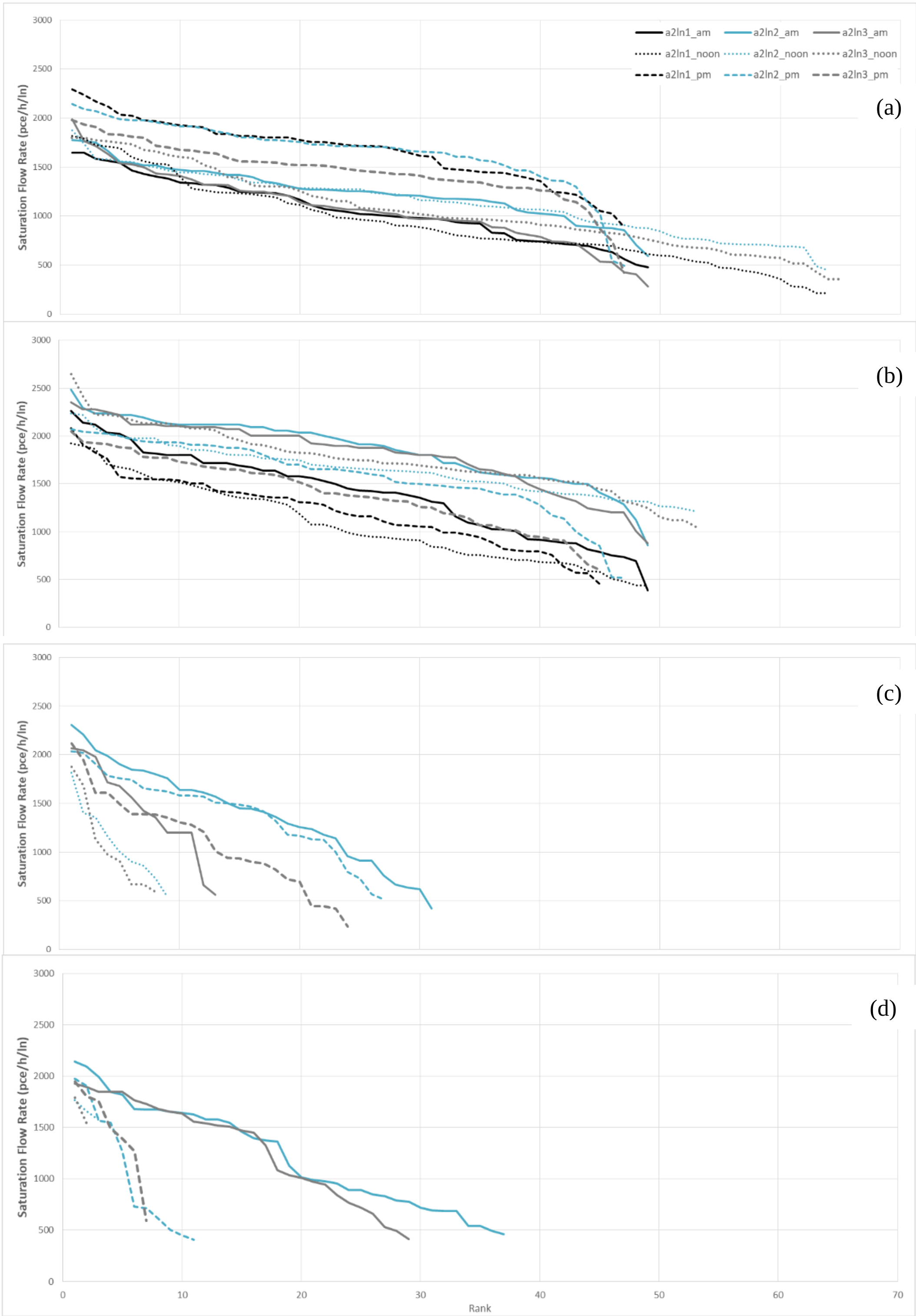

Figure 4.13: Distribution of saturation flow rate from highest to lowest at a) A2 b) A4 c) A1 d) A3

Site-based observations as stated in HCM was calculated for each approach with respect to each lane. Maximum values saturation flow calculations are shown in Table 4.7. Although effects of passenger car equivalency factor are seen, some of the lane saturation flow calculations could not change because a number of private car was mostly dominant in the each cycle. According to the table, saturation flow was higher at major approach legs A2 and A4. When maximum value of saturation flow was reached 2631 veh/h/ln at Approach 4 Lane 3 at noon, Approach 2 forced to 2267 veh/h/ln levels for saturation flow at Lane 1 evening observation period. In parallel, higher values for each approach leg was taken under the same conditions when passenger car equivalency factors were implemented. Like them, for Approach 1, highest saturation flow was seen at morning period at Lane 2, but the rate of change is higher than because the proportion of the vehicles to other types of vehicle was higher at this approach leg.

Lower saturation flow observations were defined at minor legs. All of the lane saturation flow results of them was roughly low from major legs. Saturation flow at Lane 1 at minor legs could not be seen. Vehicles preferred flare lane for movement of right turn and through and left turn movements could be done easily at lane 2 and 3, so there was no saturation flow observed at Lane 1 for minor approach legs. Moreover, saturation flow calculations were done with respect adjustment factors. According to site-based observations, heavy vehicle factors and lane width effects were taken into account while implementing HCM methods. The result of this method is listed in Table 4.7. All of the approach legs have approximately same saturation flow. Also, a calculation of saturation flow was done with respect to HBS. Unlike HCM, HBS is accepting base saturation flow as 2000 pce/h/ln, so calculations were mostly around 2000 pce/h/ln. Each of the observations has been giving an idea about each approach saturation flow. Although there is no number of vehicles observed at Lane 1 at minor legs, calculation dependent on adjustment factors was offered a level of saturation flow as same as major approach legs. This causes overestimation about saturation flows at those lanes. However, saturation flows at major legs at site-based observations was almost higher from minor legs but calculations methods based on adjustment factors offered approximately same for all approaches. In this respect, differences in the methods cause underestimation or overestimations about saturation flow of each lane at each approach legs.

Table 4.7: Saturation flow calculation results at major approach legs (A2&A4)

	Approach Legs (Major)					
	A2			A4		
	Lane 1	Lane2	Lane3	Lane 1	Lane2	Lane3
HCM with site observations (max site observation) S (veh/h/ln)						
AM	1591	1773	1800	2171	2323	2308
NOON	1800	1980	1800	1722	2222	2631
PM	2267	1926	1754	2081	1970	1929
HCM with site observations (max site observation) S (pce/h/ln)						
AM	1647	1773	1989	2264	2323	2353
NOON	1818	2093	1800	1925	2222	2647
PM	2293	2143	1978	2081	2069	2045
HCM using adjustment factors S (pce/h/ln)						
AM	1769	1770	1751	1738	1751	1749
NOON	1778	1766	1780	1748	1759	1750
PM	1746	1757	1763	1771	1761	1764
HBS S (pce/h/ln)						
AM	1985	1985	1969	1958	1969	1968
NOON	1992	1982	1994	1967	1976	1969
PM	1965	1974	1979	1986	1978	1981

Table 4.8: Saturation flow calculation results at minor approach legs (A1&A3)

	Approach Legs (Minor)					
	A1			A3		
	Lane 1	Lane2	Lane3	Lane 1	Lane2	Lane3
HCM with site observations (max site observation) S (veh/h/ln)						
AM	0	2160	2045	0	2057	1800
NOON	0	1818	1800	0	1800	1800
PM	0	2000	2160	0	1978	1946
HCM with site observations (max site observation) S (pce/h/ln)						
AM	0	2308	2069	0	2143	1925
NOON	0	1818	1875	0	1800	1800
PM	0	2034	2160	0	1978	1946
HCM using adjustment factors S (pce/h/ln)						
AM	1900	1752	1752	1900	1696	1710
NOON	1900	1707	1752	1900	1729	1752
PM	1900	1735	1752	1900	1730	1738
HBS S (pce/h/ln)						
AM	2000	2000	2000	2000	1952	1964
NOON	2000	1961	2000	2000	1980	2000
PM	2000	1985	2000	2000	1981	1988

4.5 Performance of Signal Control Approach

Saturation flow calculations depending on the site-based methods of HCM was evaluated with respect to moving average methods. Site-based observations were revised according to Equation 3.8 and 3.9. In order to compare results of them, frequency distributions of site-based method of HCM and moving average were analyzed. Percentile distribution of these methods is shown in Figure 4.14. It reflects the percentile distribution of A2. Percentile distributions of other legs are shown at Appendix D. Although Lane 1 had lower saturation flow rate at am and noon observations period, it reached the highest value for A2 at evening observation period due to traffic congestion at lane 2 and 3. Like site-based observations Lane

1, highest saturation flow rates were observed at lane 2 and 3 evening period as 2143, 1978 pce/h/ln, respectively. Analysis done with moving average methods gave parallel results about which lane shows the highest saturation flow rate but different flow rate observations was seen due to vehicle types. While calculating with moving methods, former and leading vehicle composition was a bus, private car and private car. By using pce factors at TS6407, saturation flow rate calculations was revised, so values at observed period reached 3035, 2802 pce/h/ln for lane 2 and 3 respectively as seen at the figure. While highest value was 2.12 % among the distribution of saturation flow rate of A2, this rates was approximately 0.3 % at moving average methods. This caused jump at results of moving average methods. In order to compare the results of the base saturation flow rate acceptance of HCM, results of the moving average were sorted, so 42% of the pm observation at Lane 1 and Lane 2 was exceeded to 1900 veh/h/ln. 32% of the flow rate of Lane 3 was exceeded at pm period and the same lane reached 27% of Lane 3 was above base saturation flow of HCM. Although exceeding level at noon periods was lower than others, Lane 3 exceeded as 20 percent at noon. Sorting of moving average observation with respect to HCM is figured for each approach legs and analyzed is shown in Appendix D.

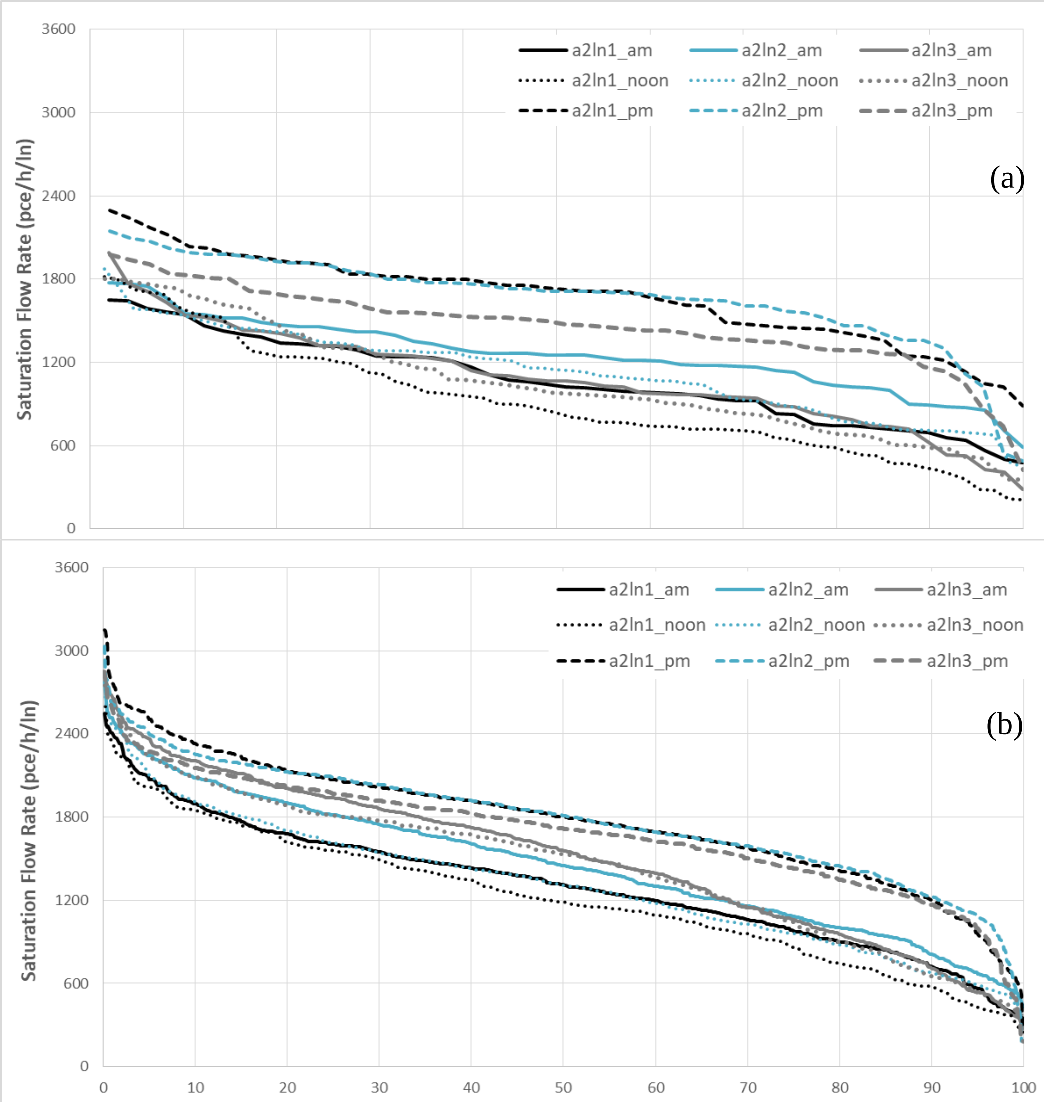

Figure 4.14: Percentile distribution of saturation flow rate frequency at A2 a) HCM b) moving average observation

Unlike base saturation flow rate at HCM, 85 percentile instantaneous flow for each lane was determined as a limit for saturation flow rate. While analyzing, the effectiveness of analyzes was studied with the efficiency based observations for each cycle. Traffic flow of cycle 737 at A2 is seen in Figure 4.15 for Lane 2. For efficiency calculations, used green time was determined for cycle. Dash black lines at the figure are representing the beginning and lost time at cycle. For beginning lost time, first four vehicles at the cycle was eliminated according to HCM site-based observations. Also, as stated Agent & Crabtree (1983), end lost time was accepted as 2.5 seconds, so used green period was determined for cycle. Beginning and end of used green period of cycle are stated at the figure. Redline at the cycle was representing 85 percentile of instantaneous flow for each lane, which was 1980 pce/h/ln for Lane 2 during AM period of the approach leg. The pink part of the figure represents used green period and purple part of it is an area where inefficiency calculations were done. The area below this level shows inefficient flow condition for that cycle. At cycle 737, the area below the level determined from 85 percentile of distribution was divided total flow within a used green and 26.8% of traffic flow below the limit. This percentage represents the inefficiency for cycle 737 for that lane.

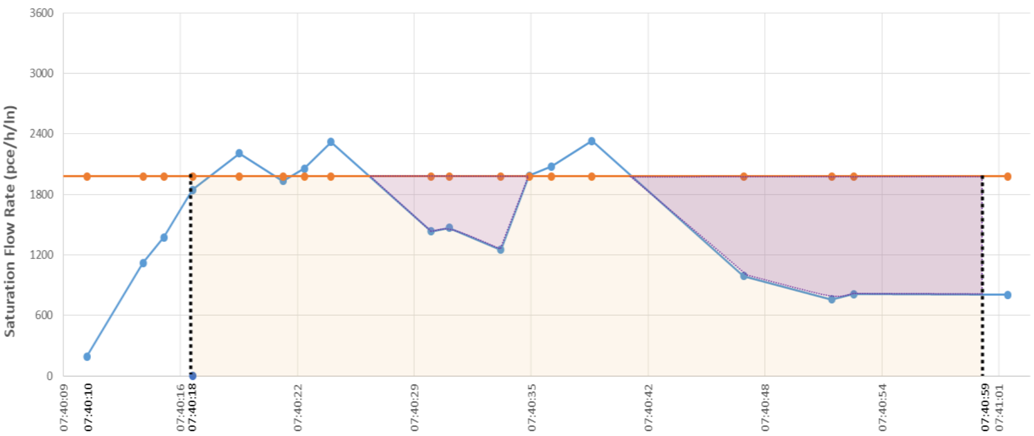

Figure 4.15: Inefficiency observation at A2 at cycle 737 Lane2

Table 4.9: 85 percentile instantaneous saturation flow at each approach (veh/h/ln)

veh/h/ln	A2			A4		
Lane	1	2	3	1	2	3
AM	1680	1901	1998	2096	2380	2400
NOON	1660	1730	1878	1691	2145	2290
PM	2120	2134	2037	1975	2200	1960
	A1			A3		
Lane	1	2	3	1	2	3
AM	---	1614	1666	---	1749	1760
NOON	---	1356	1544	---	1008	1421
PM	---	1781	1507	---	1446	1503

Table 4.10: 85 percentile instantaneous saturation flow at each approach (pce/h/ln)

pce/h/ln	A2			A4		
Lane	1	2	3	1	2	3
AM	1800	1980	2080	2200	2500	2500
NOON	1800	1800	1980	1800	2200	2400
PM	2200	2180	2080	2000	2300	2000
	A1			A3		
Lane	1	2	3	1	2	3
AM	---	1700	1700	---	1900	1850
NOON	---	1605	1700	---	1750	1800
PM	---	1900	1700	---	1600	1800

Before and after cycle conditions of cycle 737 at A2, which is examined at Figure 4.15, are shown in Figure 4.16. Inefficiency observation of the cycle 736 is seen at left part of the figure and its inefficiency percentage was 33.8%; whereas, 35 percentile of the flow at the cycle 738 was observed as inefficient. Observations were done for the whole of the cycles at am, noon, pm periods and observed period cycle conditions are shown in the Appendix E. Moreover, inefficiency ratio for each cycle was shown. Minor approach legs contained low traffic flow and density. There were no continuous flow conditions at minor legs, so inefficiency ratios are shown for major approach legs at Figures able 4.17 and 4.18.

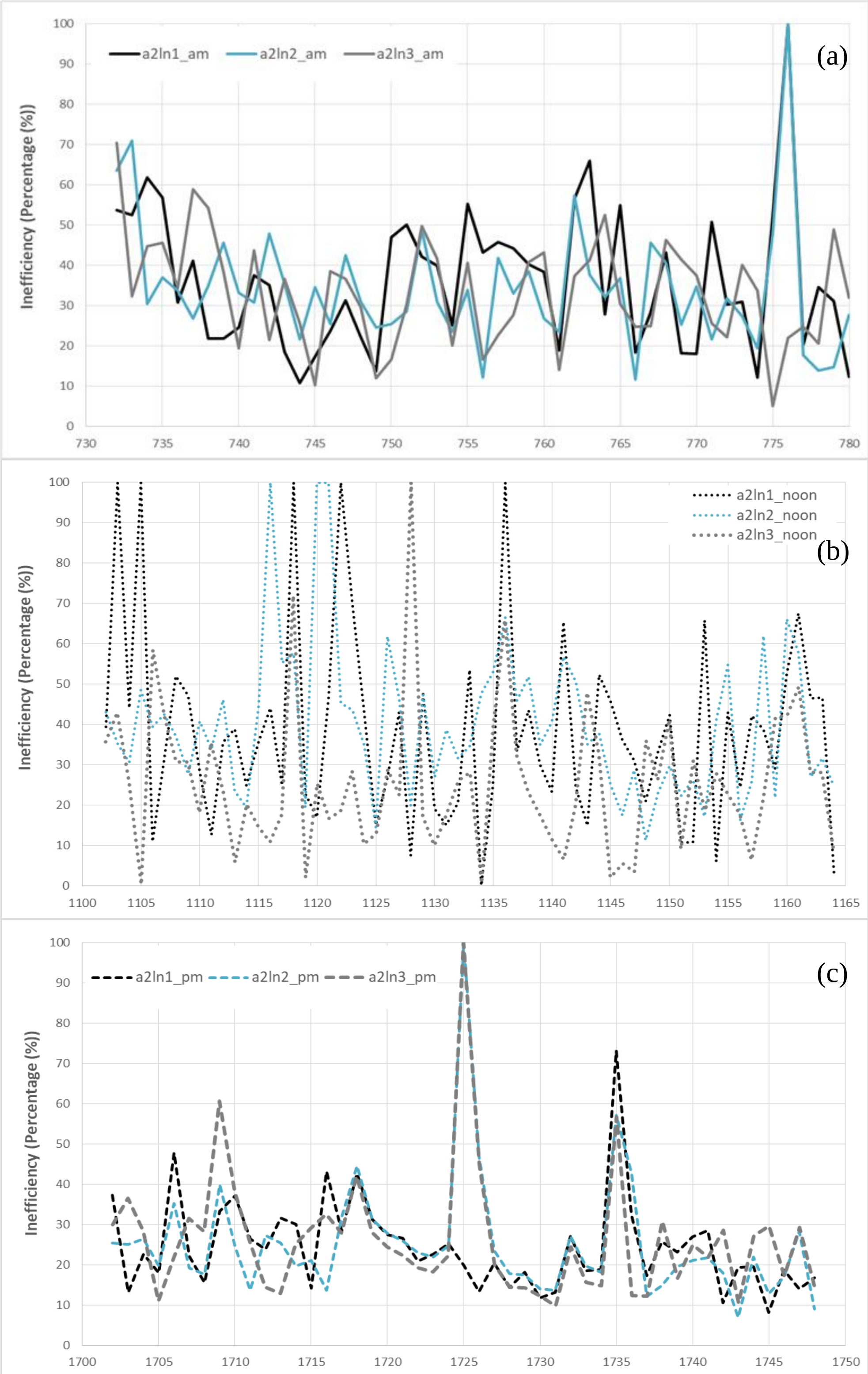

Figure 4.16: Cycle base inefficiency result at A2 during a) AM, b) NOON and c) PM periods

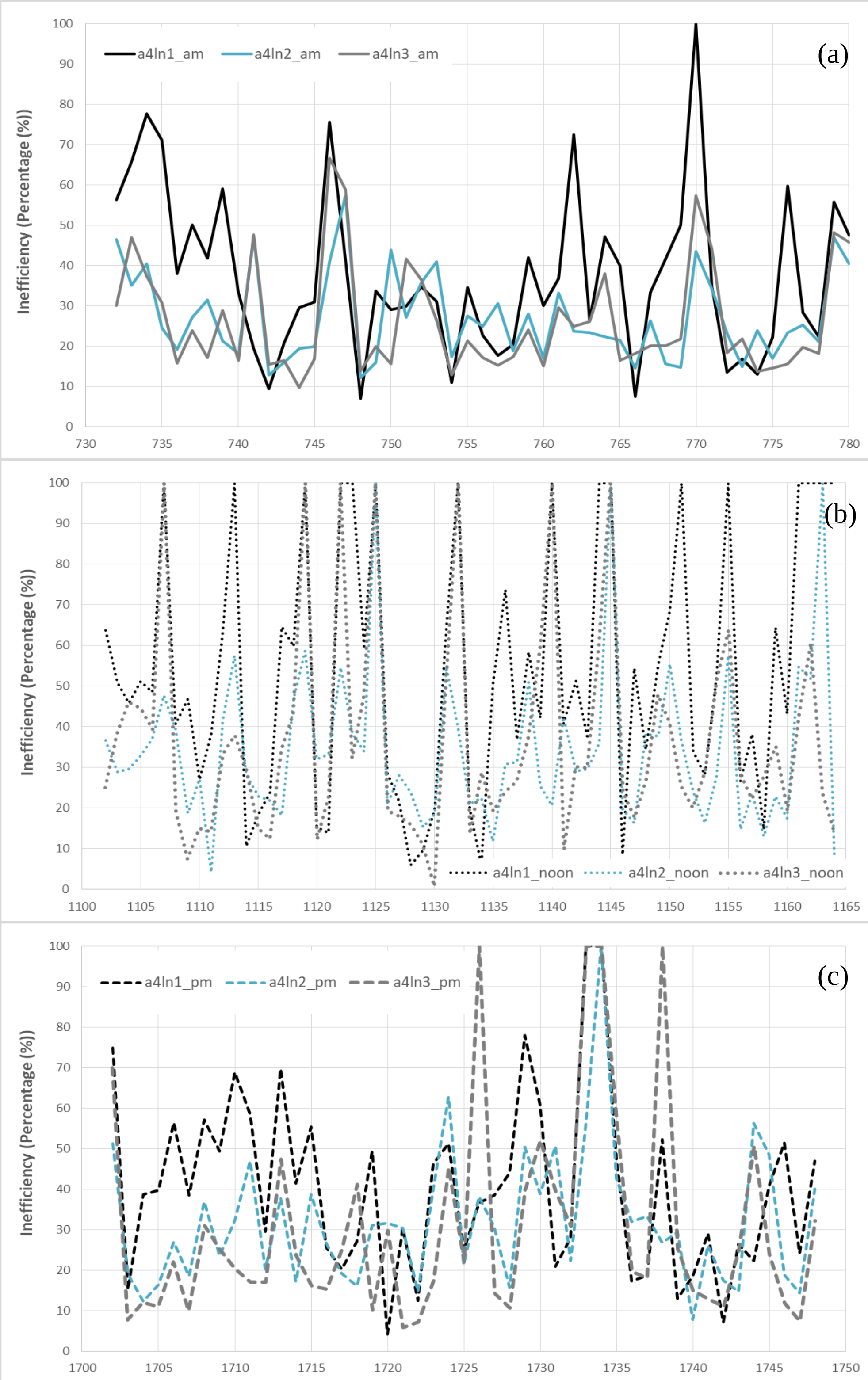

Figure 4.17: Cycle base inefficiency result at A4 during a) AM, b) NOON and c) PM periods

CHAPTER 5

CONCLUSIONS

The roundabout is a kind of intersection, which control traffic without signalization as in many applications in Europe and USA. However, problems in the design standards and differences in traffic culture, signalization of roundabouts in Turkey as well as Asian countries is recently introduced topic in traffic signal control. There is very little in the literature, which focused on mostly the modelling of the traffic in the circulating flows. This study however, focused more on the capacity estimation of approach legs of a fully signalized roundabout, which act similar to those of the signalized intersections, for which certain performance measures were developed in HCM for the USA applications (similar concepts were introduced in HBS for Germany).

Analysis of time headways on every lane in every approach leg of a signalized roundabout in Konya (which connects a major and a minor arterial road) during both morning and evening peak periods as well as the noon off-period showed that there are great fluctuations in the approach conditions. Furthermore, introduction of a new concept of “instantaneous flow rate” using the 3-average method, also enabled the development of a new concept of “inefficiency” for the approach legs lanes compared to a “practical saturation flow rate”, denoted as S_{85}, which corresponded to the 85 percentile value of the saturation flows rates calculated HCM and HBS methods. The major findings of the case study and further recommendations for future research are given below.

5.1 Major Research Findings

- The statistical analysis of the 19057 time headway data for 19057 vehicles passing through study intersection showed a lognormal distribution with an interquartile range (IQR) of [1.6s;3s] on major approaches during the peak periods, while IQR on minor legs were [2s;4s].
- Time headways were more scattered during the off-peak noon with IQR values of [2s;3s] and [2s;5s] for major and minor approaches, respectively.
- Saturation flows based on design characteristics using HCM and HBS methods suggested values in the range of 1900 – 2000 veh/h/ln for different lanes in the major approaches. Estimation of HCM site-based observation method produced higher values reaching up to 2631 veh/h/ln
- The practical S values, S_{85}, were found within the range of 2400 veh/h/ln, which were reached mostly during the peak hour cycles, but also, some of the cycles in the noon, as well. 3-moving average flow rates showed values as high as 2900 veh/h/ln and as low as 600 veh/h/ln, even sometimes in the same cycle.
- Lane 2 and Lane 3 (the outermost lane) on all approaches generally experienced higher flows regardless of the time or assumed methodology.
- Average inefficiency during green periods was found as 30 percent even on the major approaches, which were due to traffic break in the upstream flow conditions.
- Cycle specific analysis of the time headways revealed an average of 1200 - 1800 veh/hr/ln in most of the cases
- A rather simplistic assumption of a constant, saturation flow rate as mentioned at HCM could not be reached constant level after 4th vehicle at the cycle. It is not easy to define the constant level for each cycle according to site-based observation at Konya.

5.2 Recommendations and Future Research

Dynamic signal control systems are the most commonly implemented Intelligent Transportation Systems (ITS) applications in Turkey, especially in the urban networks due to severe congestions that have been observed parallel to the increase of the vehicle ownership and usage in the cities. These systems always assume a certain saturation flow while choosing a cycle length and distributing green times based on the vehicular density in the approach legs of the intersections/roundabouts. However, the case study results showed that assuming simple constant saturation flow during green periods is rather simplistic; it may be good only for basic design or phasing determinations. But, in an era of fully adaptive signal control, adaptive real-time signal control has to include the inefficiency due to large time headways stemming from upstream conditions; early cut-off in a phase or even phase skipping should be coded based on the time headway and instantaneous flow rate values. For this purpose, image-processing algorithms at the dynamic signal control systems should be enhanced to keep track of not the only measurement of a number of vehicles in the approach but also the time headways using virtual detectors. Alternatively, signal control camera systems should be supported by new and cheap lane-based sensors for headway measurements; it would be even better if they are located slightly further than the camera vision

Signalized roundabouts have to be modeled more carefully for the flow in the circulating lanes to determine whether signalization is introducing any more efficiency or under what conditions. This, however, requires not only vehicle tracking within the roundabout but also traffic flow model fitting for the merge and diverge movements, which would be the topic of a follow-up study.

REFERENCES

Agent, K., & Crabtree, J. (1983). *Analysis of Lost Times at Signalized Intersections.* Kentucky: Kentucky Transportatio Research Program college of Engineering University of Kentucky Lexington.

Akçelik, R. (1981). *Traffic signals: capacity and timing analysis.* Retrieved from http://trid.tb.org/view.aspx?id=173392.

Akçelik, R. (2011). Roundabout metering signals: capacity, performance and timing. *Transportation Research Borad*.

Alam, M., Osra, K., Al-Bar, H., & Zahran, S. (2011). Signalized Intersection Capacity Adjustment Factors for Makkah,Saudi Arabia. *Canadian Journal of Transportation*, 56-66.

Bai, Y., Chen, W., & Xue, K. (2012). Association of Signal-Controlled Method at Roundabout and Delay. *Journal of Software*, 346-351.

Bajad, D., & Ali, M. (2016). Design Principles of Traffic Signals. *Imperial Journal of Interdisciplinary Research*, 802-804.

Bang, K. (1978). Capacity of Signalized Intersection,Swedish Capacity Manual. *Transportation Research Record 667,TRB*, 11-21.

Bonneson, J., Nevers, B., Pratt, M., & Bonyani, G. (2006). Influence of Area Population, Number of Lanes, and Speed Limit on Saturation Flow Rate. *Transportation Research Board*, 76-85.

Boumediene, A., Brahimi, K., Belguesmia, N., & Bouakkaz, K. (2009). *Saturation Flow Versus Green Time At Two Stage Signal Controlled Intersections*. Retrieved from www.transport.vgtu.lt.

Brılion, W., & Vandehey, M. (1998). Roundabouts-The State of the Art in Germany. *ITE Journal*, 48-54.

Chen, W., Bai, Y., & Xue, K. (2012). Association of Signal-Controlled Method Roundabout and Delay . *Journal of Software*, 346-351.

Crockett, Z. (2015, September). *The Case for More Traffic Roundabouts*. Retrieved from http://priceonomics.com/the-case-for-more-traffic-roundabouts/?linkId=17225036.

Çakıcı, Z., & Murat, Y. (2016). Sinyalize Dönel Kavşaklar için Hesap Yöntemi Önerisi ve Performanz Analizi. *İMO Teknik Dergi*.

Çetin, M., & Murat, Y. (2013). Doygun Akım Belirlenmesinde Sürücü Davranışlarına Bağlı Bir Matematik Yaklaşım. *İMO Teknik Dergi*, 6399-6414.

Droke, C. (2001). *Moving Averages.* Columbia: Marketplace Books.

European Commision. (2017). *Mobility and Trasnport*. Retrieved from https://ec.europa.eu/transport/road_safety/home_en.

Flannery, A. (2001). Geometric Design and Safety Aspects of Roundabouts . *Transportations Research Record*, 76-81.

Florida Department of Tramsportation Roadway Desgin Office . (2014). *Florida Intersection Design Guide.*

Garber, N., & Hoel, L. A. (2009). *Traffic & Highway Engineering.* Toronto: Cengage Learning Customer & Sales Support .

ISSD Inc. (2016). *CHAOS, Dinamik Kavşak Kontrol Sistemi.* Ankara. Retrieved from http://www.issd.com.tr/en/22985/Dynamic-Junction-Management-System-CHAOS.

Jakimavicius, M., & Burinskiene, M. (2009). A GIS and multi-criteria-based analysis and ranking of transportation zones of Vilnius city. *Technological and Economic Development of Economy* , 39-48.

Kusuma, A., & Koutsopoulos, H. (2011). Critical Gap Analysis of Dual Lane Roundabouts. *Procedia Social and Behavioral Sciences*, 710-717.

Liu, X.-m., & Shao, C.-g. (2012). EStimation of Saturation Flow Rates at Signalized Intersections. *Hindawl Publishing Corporation Discrete Dynamics in Nature and Society*, 9.

Lu, J., & Pernia, J. (2000). The Difference of Driving Behavior Among Different Driver Age Groups at Signalized Intersection. *IATSS Research*, 75-84.

Mathew, D., & Bombay, I. (2014). Design Principles of Traffic Signal. In *Transportation Systems Engineering* (pp. 34.1-34.13).

National Research Council. (2010). *Highway Capacity Manual.* Washington, D.C.

NCHRP Synthesis 264. (1998). *Modern Roundabout Practice in the United States.* Washington: National Academy Press.

Nııttymaki, J., & Pursula, M. (1997). Saturation Flows at Signal-Group-Controlled Traffic Signals. *Transportation Research Record* , 24-32.

Radhakrishnan, S., & Ramadurai, G. (2015). Discharge Headway Model for Heterogenous Traffic Conditions. *Transportation Research Procedia*, 145-154.

Retting , R., Persaud, B., & Garder, P. (2001). Crash and Injury Reduction Following Installatıon of Roundabouts in the United States. *American Journal of Public Health*, 628-631.

Saha, P., Mahmud, H., Hossain, Q., & Islam, Z. (2009). Passenger Car Equivalent (PCE) of Through Vehicles at Signalized Intersections in Dhaka Metropolitan City, Bangladesh. *IATSS RESEARCH*, 99-104.

Salim, A., Vanajakshi, L., & Subramanian, S. (2010). Estimation of Average Space Headway under Heterogeneous Traffic Conditions. *International J. of Recent Trens in Engineering and Technology*, 6-10.

Shao, C.-q., & Liu, X.-m. (2012). Estimation of Saturation Flow Rates at Signalized Intersection. *Discrete Dynamics in Nature and Society*.

Shao, C.-q., Rong, J., & Lıu, X.-m. (2011). Study on the Saturation Flow Rate ans Its Influence Factors at Signalized Interection in China. *Procedia Social and Behavioral Sciences*.

Souliman, B. (2016). Effect of Roundabouts on Accident Rate and Severity in Arizona.

Stanic, B., Tubic, V., & Celar, N. (2011). *Straight Lane Saturation Flow and Its Rate In Serbian Cities*. Retrieved from www.tandfonline.com/loi/tran20.

SWEROAD. (2000). *Proposed Design Principles for Modern Roundabouts.*

Tanyel, S. (2005). Yuvarlakada Kavşaklarda Ana Akımdaki Ağır Araç Yüzdesinin Yanyol Kapasitesi Üzerindeki Etkisi. *Dokuz Eylül Üniversitesi Mühendislik Fak. Fen ve Mühendislik Dergisi*, 19-30.

U.S. Department of Transportation Federal Highway Adminstration. (2000). *Roundabouts: An Informational Guide.*

Ward, D. (2006). OECD Main Economic Indicators . 1-28.

Washington State Department of Trasnportation, . (2017). *Design Manual*. Washington.

Wu, N. (2017). New Features in the 2015 German Highway Capacity Manual (HBS2015). *Transportation Reseach Procedia* , 1-16.

Xu, F., & Tian, Z. (2008). Driver Behavior and Gap-Acceptance Characteristics at Roundabouts in California. *Transportation Research Borad*, 117-124.

Yang, X., Li, X., & Xue, K. (2004). *A New Traffic Signal Control for Modern Roundabouts: Method and Application*. Retrieved from http://ieeexplore.ieee.org/xpl/RecentIssue.jsp?punumber=6979.

APPENDIX A

CONFLICT POINTS AT ROUNDABOUT AND TYPES OF ROUNDABOUT

A.1 Conflict Points at Roundabout

Figure A.1 shows indicates how conflict points to the intersection changes if the traditional T-shaped intersection is converted to a T-shaped roundabout. Figure A.1 shows that there are 32 conflict points within a 4-way traditional intersection, while in the most complicated form or a roundabout, a maximum of 8 conflict points is possible to exist. This makes roundabout safer than an intersection. In this respect, 24 intersections at 8 different states of United States was converted from intersection to roundabout. 20 of them was controlled by traffic signals and 4 of them is controlled stop signs. According to observations done by Retting, Persaud, & Garder (2001), crash rates and average annual daily traffic was observed in before and after periods. The analysis shown in Figure A2 shows that annual average daily traffic was increased at all sites; whereas, the crash rate decreased in the same respect.

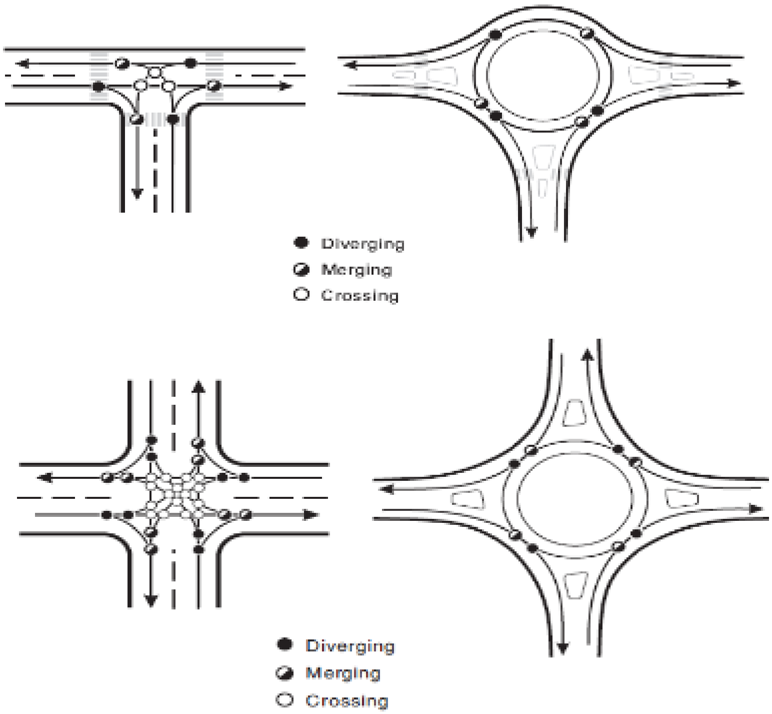

Figure A.1: Conflict points comparison T and 4-way intersection with 4-way roundabout (FHWA, 2000)

Jurisdiction	Year Opened	Control Before Conversion[a]	Single or Multilane	Annual Average Daily Traffic		Months		Crash Count			
								Before		After	
				Before	After	Before	After	All	Injury	All	Injury
Anne Arundel County, Md	1995	1	Single	15345	17220	56	38	34	9	14	2
Avon, Colo	1997	2	Multilane	18942	30418	22	19	12	0	3	0
Avon, Colo	1997	2	Multilane	13272	26691	22	19	11	0	17	1
Avon, Colo	1997	5	Multilane	22030	31525	22	19	44	4	44	1
Avon, Colo	1997	5	Multilane	18475	27525	22	19	25	2	13	0
Avon, Colo	1997	5	Multilane	18795	31476	22	19	48	4	18	0
Bradenton Beach, Fla	1992	1	Single	17000	17000	36	63	5	0	1	0
Carroll County, Md	1996	1	Single	12627	15990	56	28	30	8	4	1
Cecil County, Md	1995	1	Single	7654	9293	56	40	20	12	10	1
Fort Walton Beach, Fla	1994	2	Single	15153	17825	21	24	14	2	4	0
Gainesville, Fla	1993	5	Single	5322	5322	48	60	4	1	11	3
Gorham, Me	1997	1	Single	11934	12205	40	15	20	2	4	0
Hilton Head, SC	1996	1	Single	13300	16900	36	46	48	15	9	0
Howard County, Md	1993	1	Single	7650	8500	56	68	40	10	14	1
Manchester, Vt	1997	1	Single	13972	15500	66	31	2	0	1	1
Manhattan, Kan	1997	1	Single	4600	4600	36	26	9	4	0	0
Montpelier, Vt	1995	2	Single	12627	11010	29	40	3	1	1	1
Santa Barbara, Calif	1992	3	Single	15600	18450	55	79	11	0	17	2
Vail, Colo	1995	1	Multilane	15300	17000	36	47	16	. . .	14	2
Vail, Colo	1995	4	Multilane	27000	30000	36	47	42	. . .	61	0
Vail, Colo	1997	4	Multilane	18000	20000	36	21	18	. . .	8	0
Vail, Colo	1997	4	Multilane	15300	17000	36	21	23	. . .	15	0
Washington County, Md	1996	1	Single	7185	9840	56	35	18	6	2	0
West Boca Raton, Fla	1994	1	Single	13469	13469	31	49	4	1	7	0

Note. Ellipses (. . .) indicate that data are not available.
[a]1 = 4-legged, 1 street stopped; 2 = 3-legged, 1 street stopped; 3 = all-way stop; 4 = other unsignalized; 5 = signal.

Figure A.2: Conversion and crash counts of roundabouts (Retting, Persaud, & Garder, 2001)

A.2 Types of Roundabout

Different types of roundabout in different countries could be seen. Although they are not implemented as same as in others countries. Basic design requirements of them could be same with other countries. According to FHWA, Roundabout Design Guide (2000), roundabouts have been categorized depending on size and environment to facilitate discussion of specific performance or design issues. In this respect, roundabouts are classified into six categories. Each type of the roundabout are shown in figures.

A.2.1 Mini Roundabout

It is a small type of roundabouts, which is mostly used in low-speed urban environments. Due to short cross-passing sites for pedestrians, this is also be accepted as a pedestrian-friendly type of roundabout. However, it is not convenient for the right turning activities. Due to short angular movement and pedestrian activities probability to there, it is insufficient for compact urban environments. According to FHWA, mini-roundabouts are relatively inexpensive because they typically require minimal additional pavement at the intersecting roads. Mini-roundabout has a one-way circulatory roadway around a flush or slightly raised circular island less than 4 meters in diameter and with or without flared approaches. Also, it leads vehicle to decrease their speed. Pavement delineation and small splitter islands could be used to reduce vehicles deflection. The design speed of the vehicle inner of the roundabout is provided by horizontal deflection.

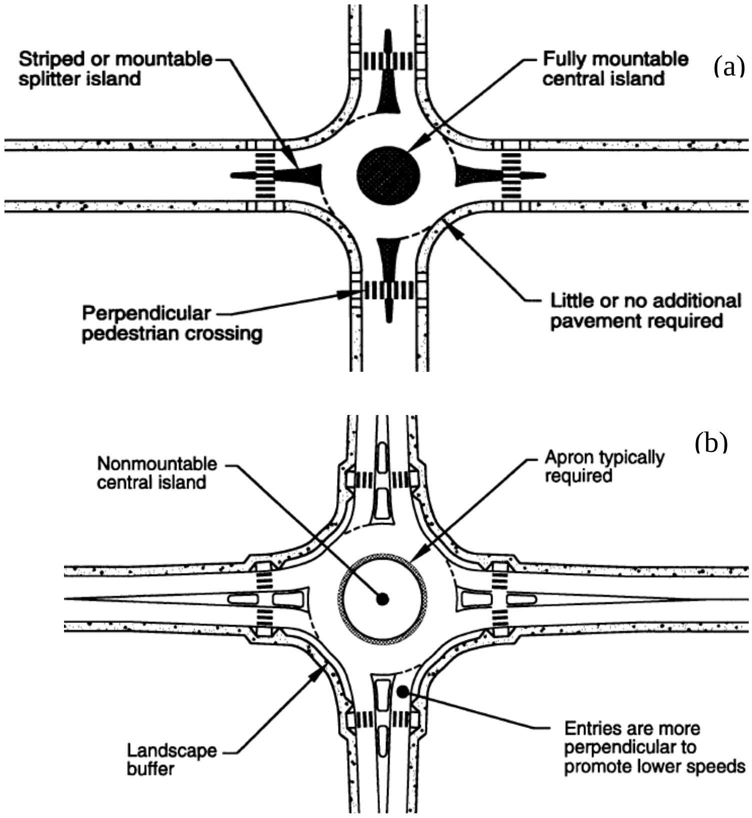

Figure A.3: Typical (a) mini-roundabout (b) urban compact roundabout (FHWA, 2000)

A.2.2 Urban Compact Roundabout

Like mini-roundabouts, urban compact roundabout has been designed with respect to low design requirements. According to FHWA, the principal objective of this design is to enable pedestrians to have safe and effective use of the intersection. In this respect, urban compact design requirements for safety is implemented in this areas, like a buffer zone, apron for a large vehicle at Splitter Island. Also, these types of roundabouts have raised splitter islands that incorporate at-grade pedestrian storage areas, and a non-mountable central island. Figure A.3 shows typical urban compact roundabout.

A.2.3 Urban Single-Lane Roundabout

In design process, all entry and exit and circulatory traffic at inner site of these types of roundabouts have a single lane. It requires larger inscribed circle diameters and more tangential entries and exits. This increase the speed of the circulatory traffic both at inner and at approach legs. With increasing the speed at the approach, raised splitter island is preferred. This led traffic to approaches and warns vehicles to follow inner site traffic. Design details and typical view of the urban single roundabout is shown Figure A.4.

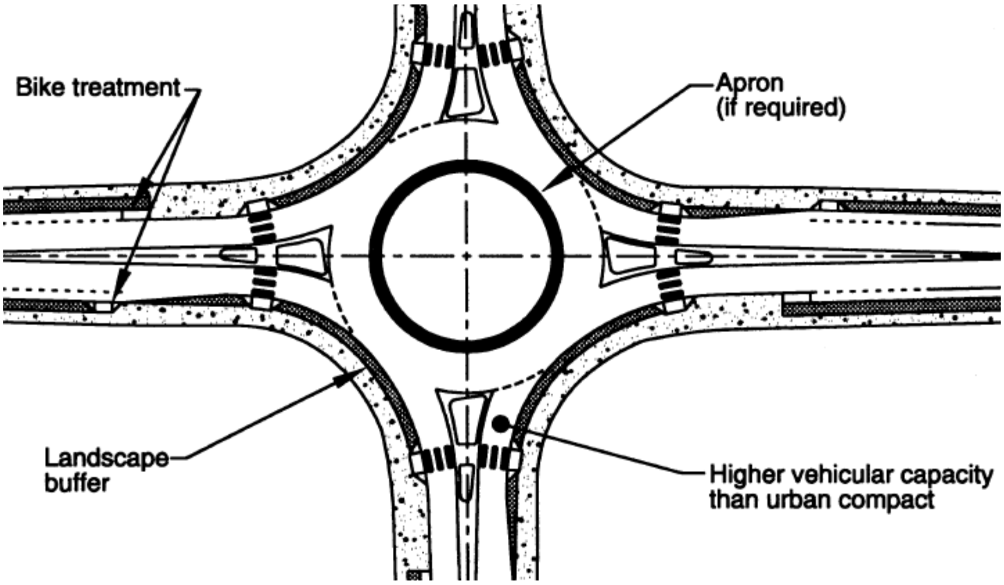

Figure A.4: Typical urban single-lane roundabout (FHWA, 2000)

A.2.4 Urban Double-Lane Roundabout

Urban double-lane roundabouts consist of at least one entry with two lanes in urban areas. Like single-lane roundabout, design requirement of the double-lane includes raised splitter islands, no truck apron, a mountable central island, and appropriate horizontal deflection. According to FHWA, bicycle and pedestrian pathways must be clearly delineated with sidewalk construction and landscaping to direct users to the appropriate crossing locations and alignment, however, pedestrian traffic and bicyclist movement of the inner site of roundabout cause much more difficulties if they exit at inner site of roundabout because high-speed flow and high capacity could be affected. Typical urban double lane roundabout is shown Figure A.5.

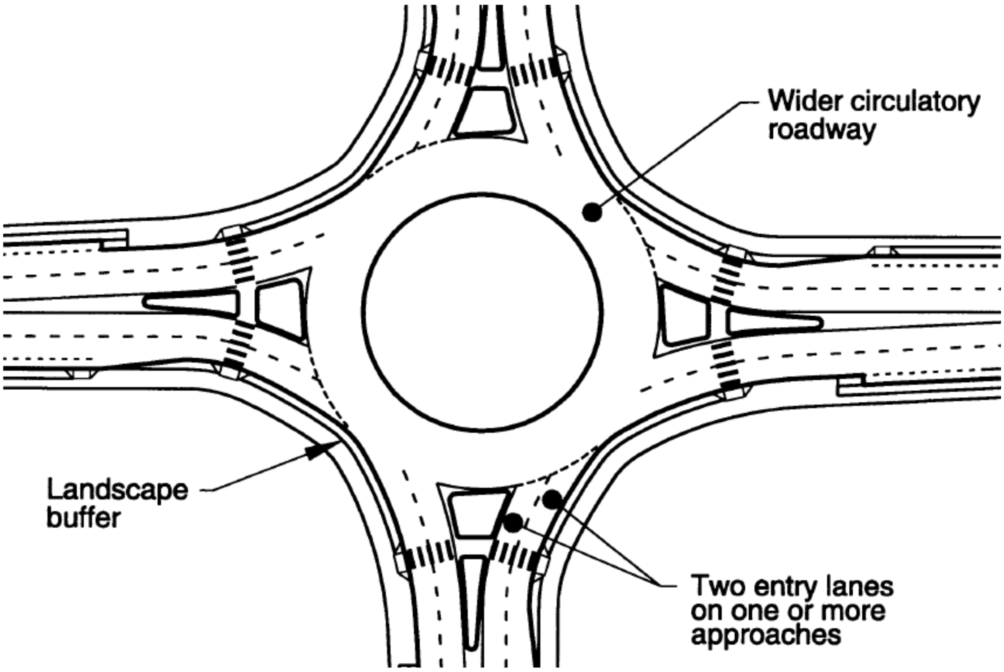

Figure A.5: Typical urban double-lane roundabout (FHWA, 2000)

A.2.5 Rural Single-Lane Roundabout

Roundabouts in rural areas have higher approach speed capacity than urban, so proper geometric conditions and traffic control device treatments lead driver to reduce their speeds. This effects pedestrian movement because of high speed cause threats to them. Therefore pedestrian activities could not be observed these types of areas. According to FHWA, rural roundabouts may have larger diameters than urban roundabouts to allow slightly higher speeds at the entries, on the circulatory roadway, and at the exits. Also, geometric design component consists of extended and raised splitter islands, a mountable central island, and adequate horizontal deflection. Figure A.6 shows typical rural single-lane roundabout.

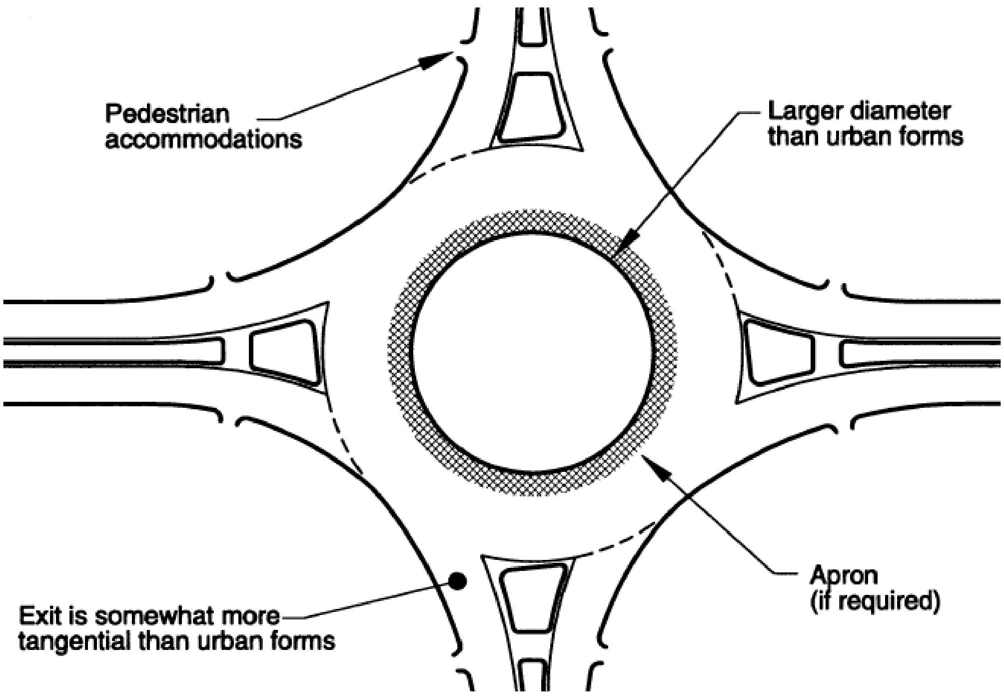

Figure A.6: Typical rural single-lane roundabout (FHWA, 2000)

A.2.6 Rural Double-Lane Roundabout

Priorities of the double-lane roundabout show similar behavior with single-lane behavior roundabout; whereas they are differentiated at two entry lanes, or entries flared from one to two lanes, on one or more approaches. In this respect, it is same with the urban double-lane roundabout. However, according to FHWA, main design differences are designs with higher entry speeds and larger diameters and recommended supplementary approach treatments. Therefore, splitter island extension and pedestrian movement requirements could be differentiated another issue. In rural areas, to reduce the speed of vehicle and lead approach vehicle activities while closing the inner side of the roundabout, Splitter Island is extended. Also, pedestrian movement design requirement is not taken into account as well as urban areas due to high-speed capabilities at rural areas design requirement.

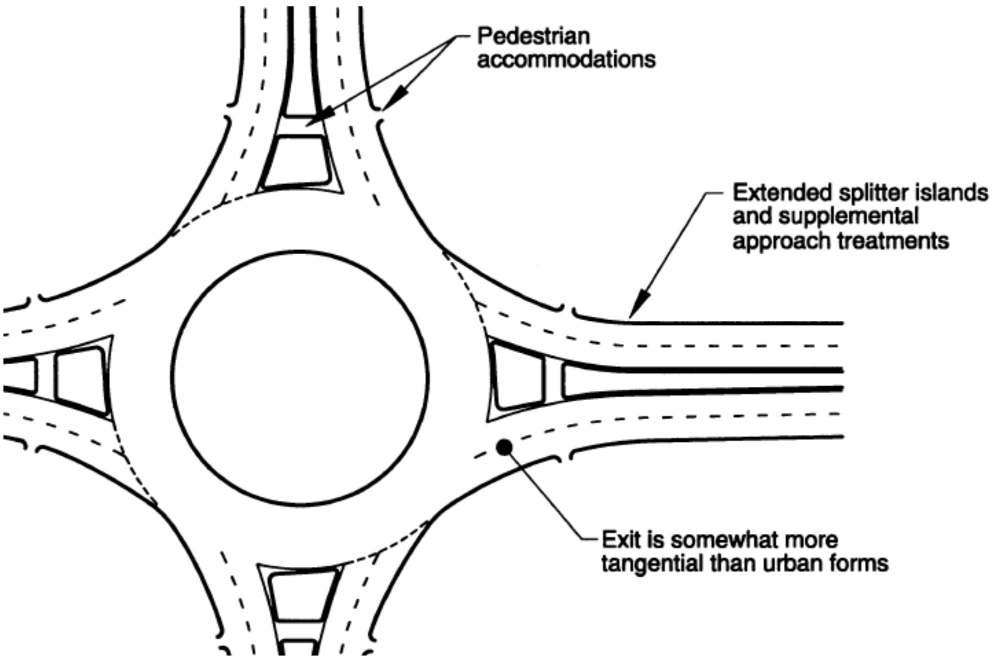

Figure A.7: Typical rural double-lane roundabout (FHWA, 2000)

APPENDIX B

HEADWAY DATA AND PROBABILITY DISTRIBUTIONS FOR THE STUDY INTERSECTION

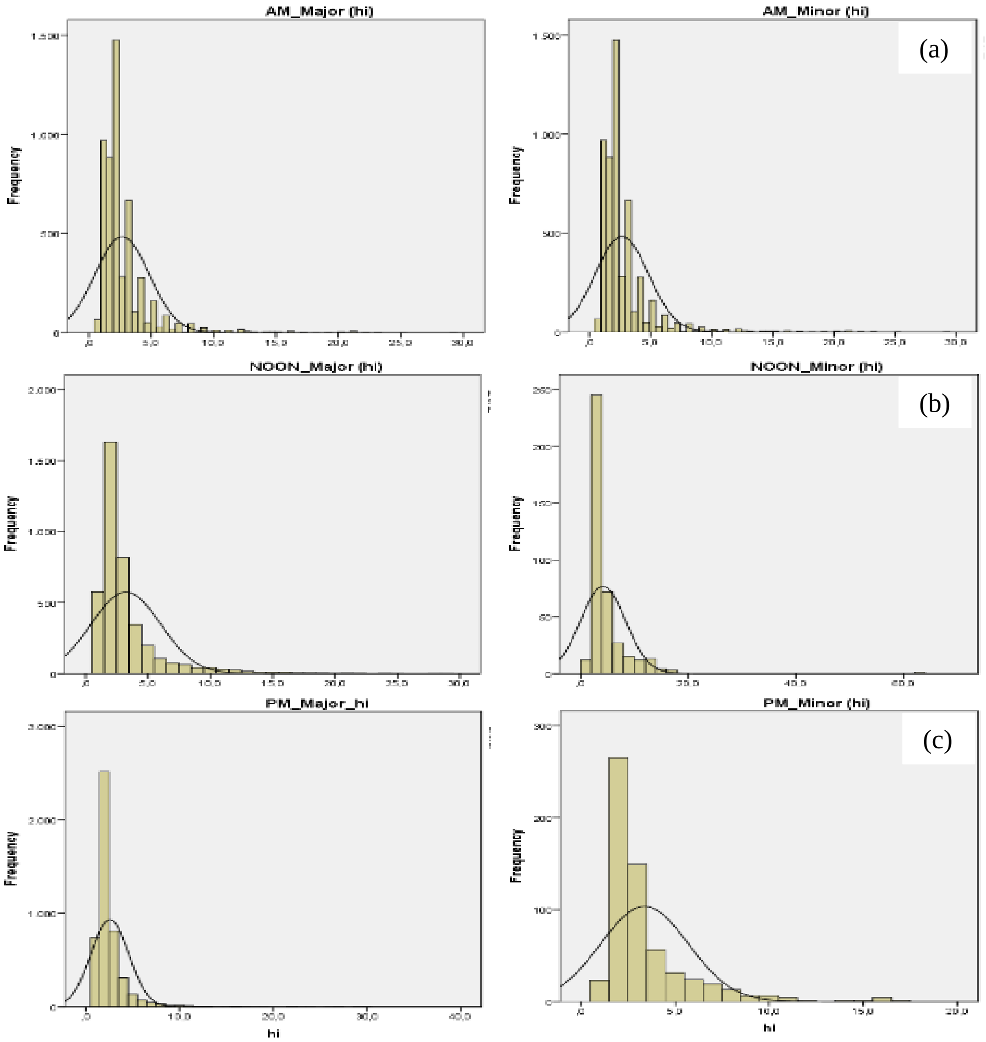

Figure B.1: Headway histograms at each approach legs during (a) AM (b) NOON (c) PM

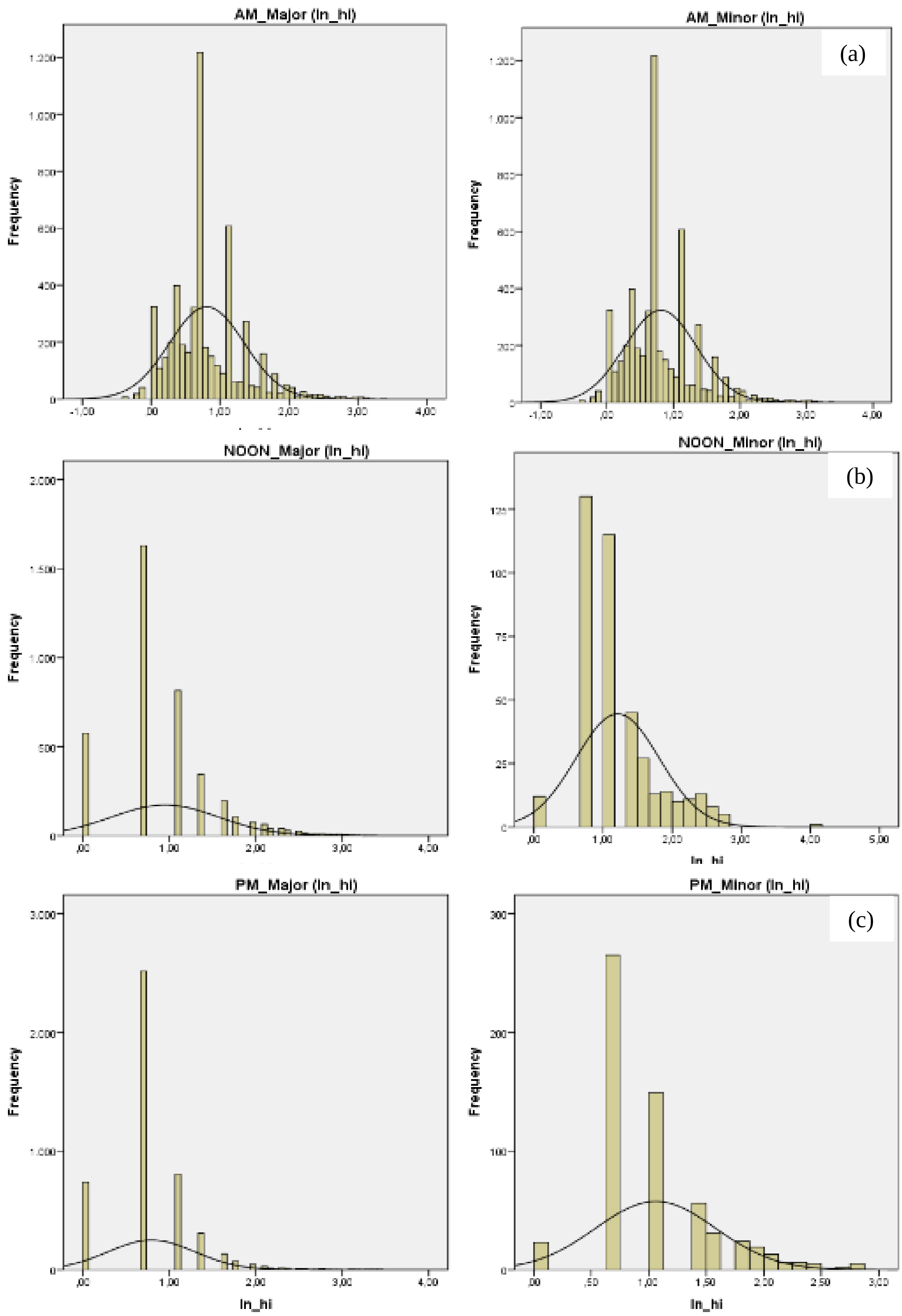

Figure B.2: Histograms of Ln (headway) at each approach legs during (a) AM (b) NOON (c) PM

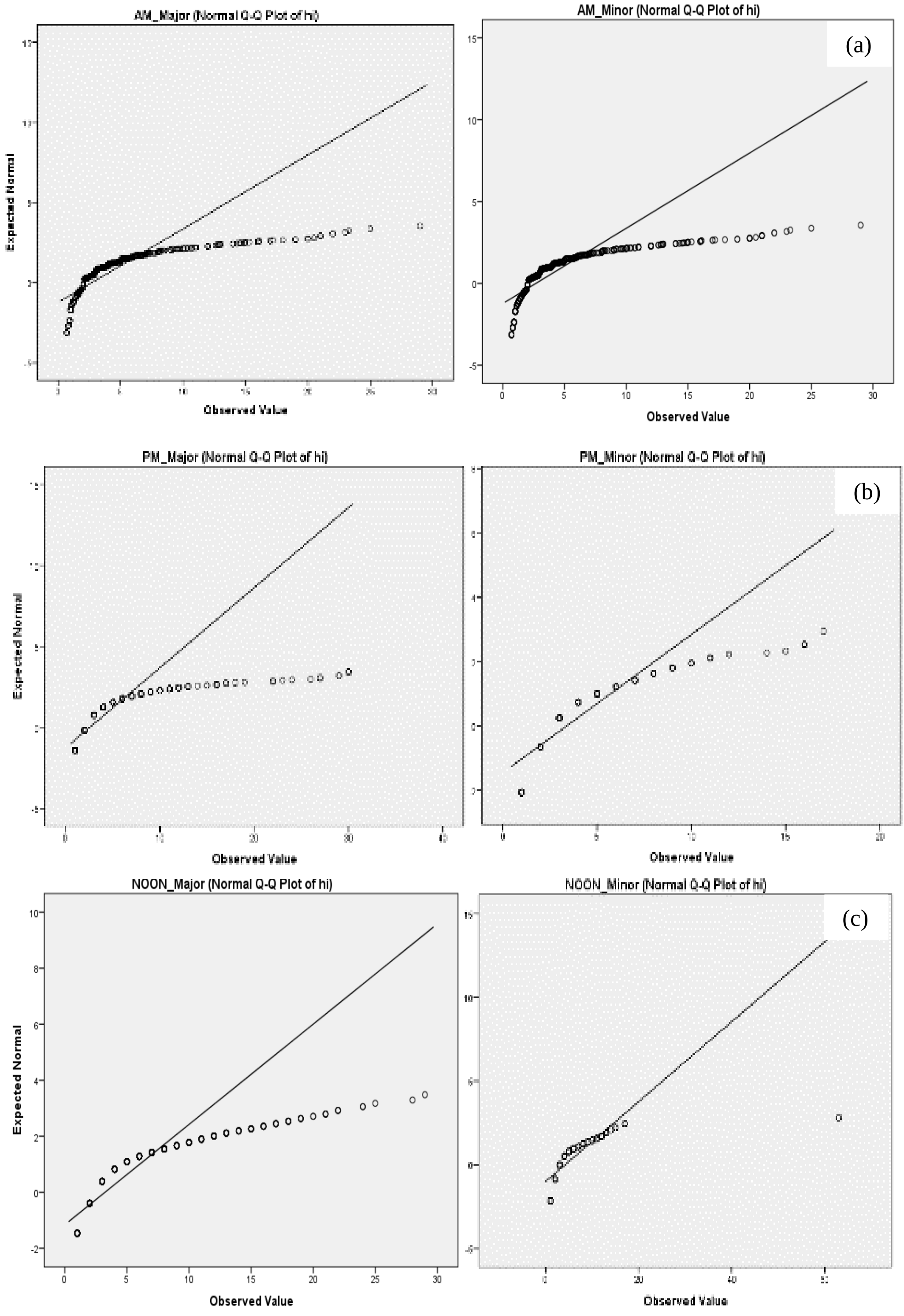

Figure B.3: Q-Q plot normal distribution at each approach legs during (a) AM (b) NOON (c) PM

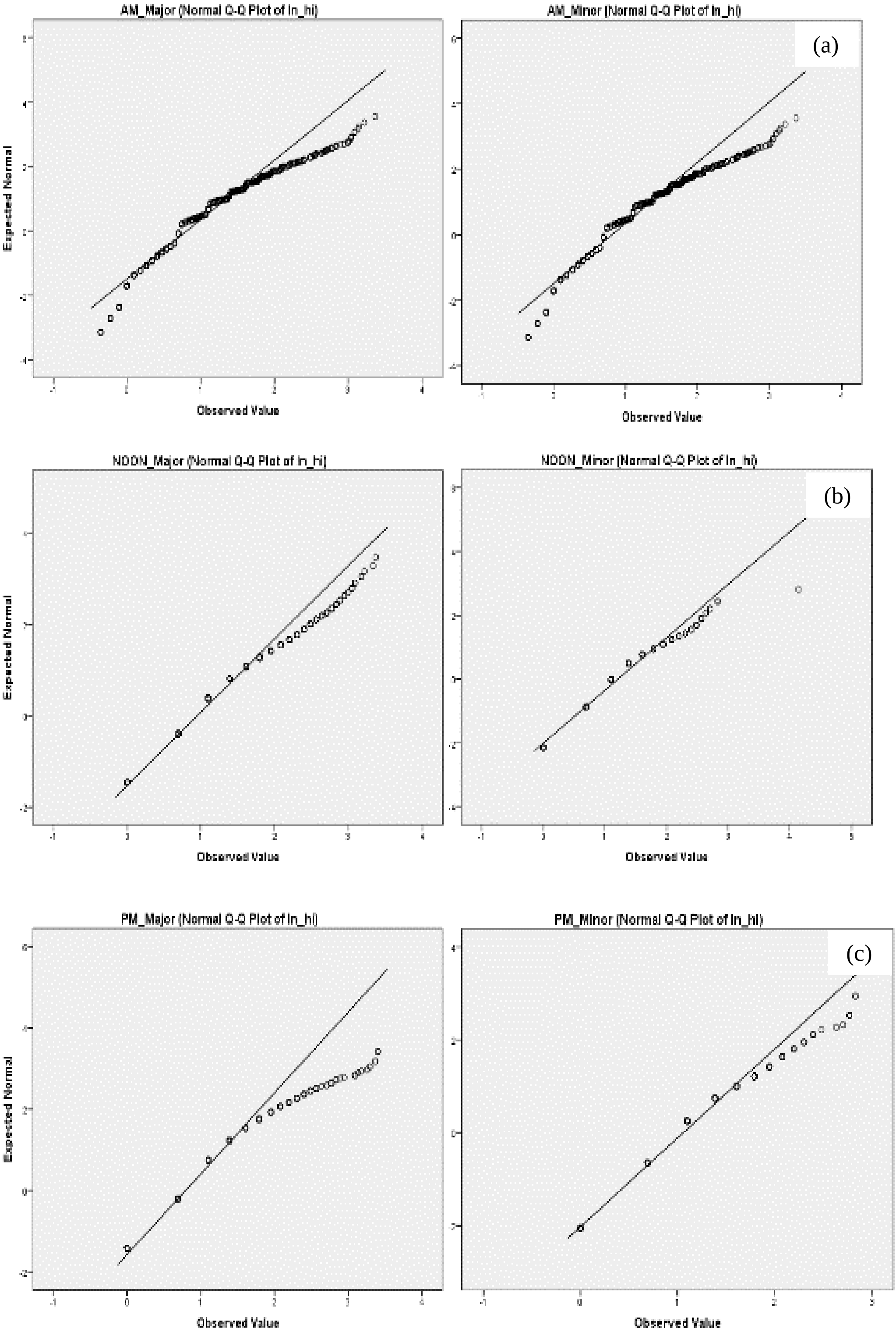

Figure B.4: Q-Q plot lognormal distribution at each approach legs during (a) AM (b) NOON (c) PM

APPENDIX C

SATURATION FLOW RATE OBSERVATION BASED ON HCM METHOD

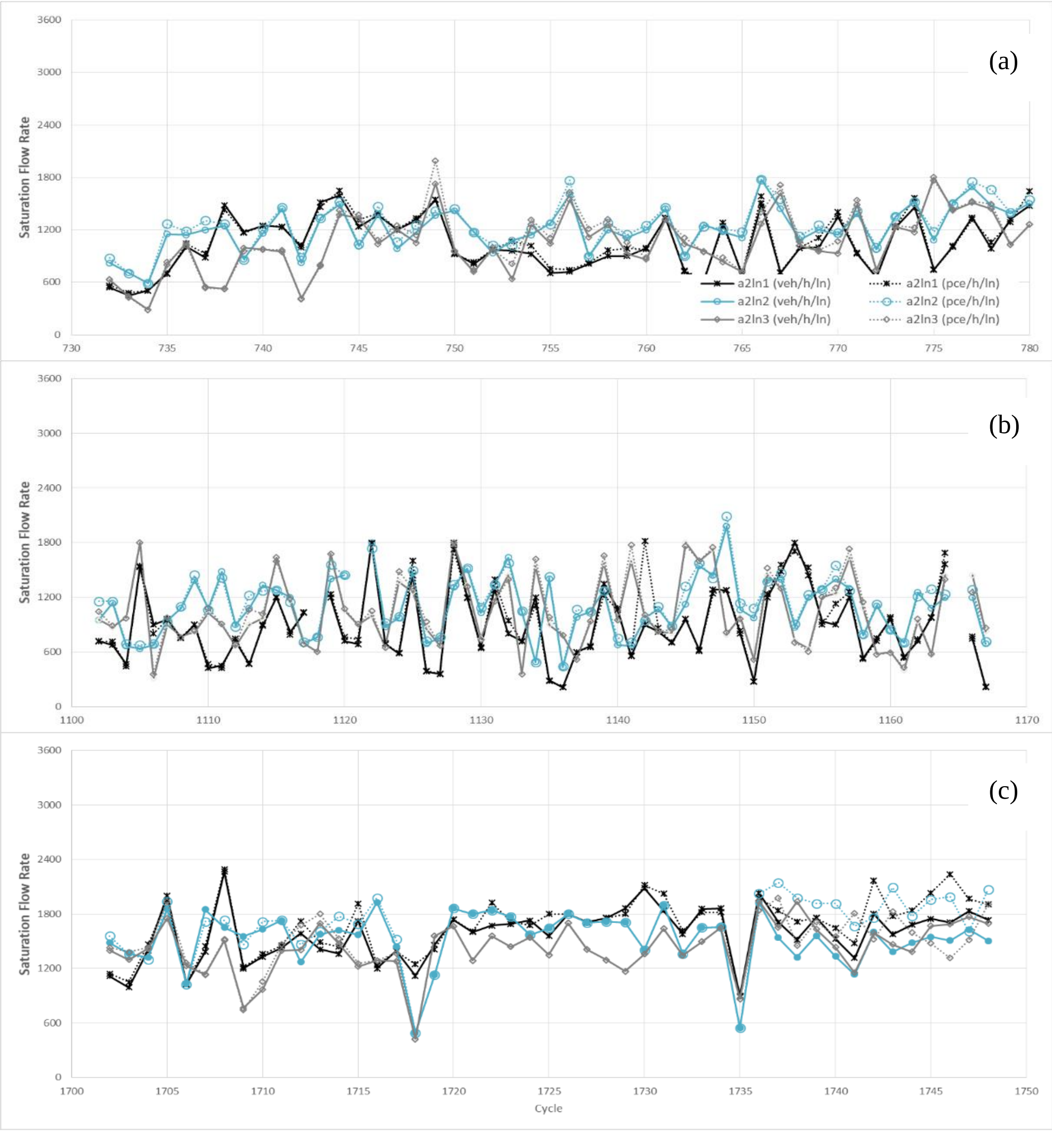

Figure C.1: Saturation flow rate & cycle observation at A2 during (a) AM (b) NOON (c) PM

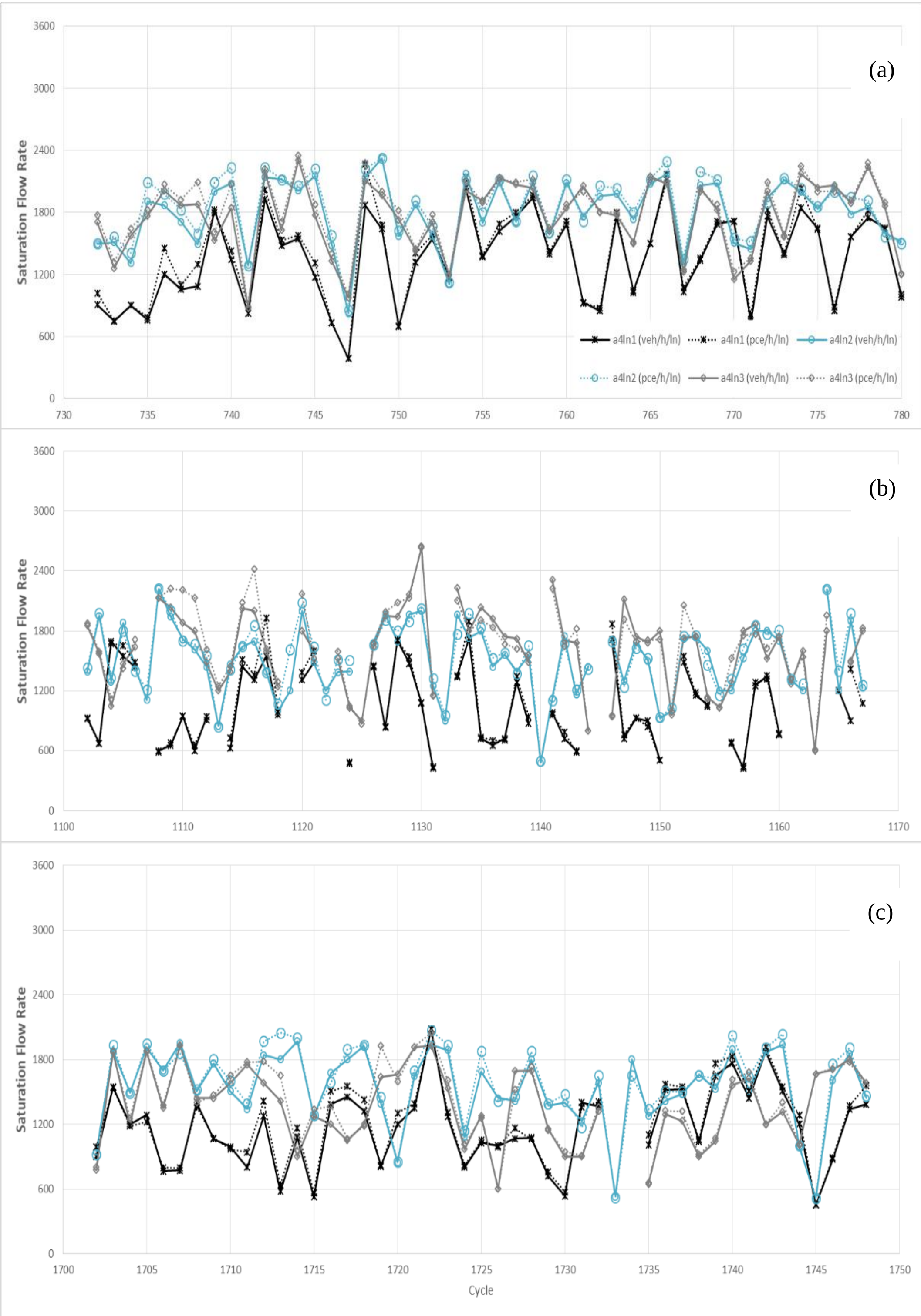

Figure C.2: Saturation flow rate & cycle observation at A4 during (a) AM (b) NOON (c) PM

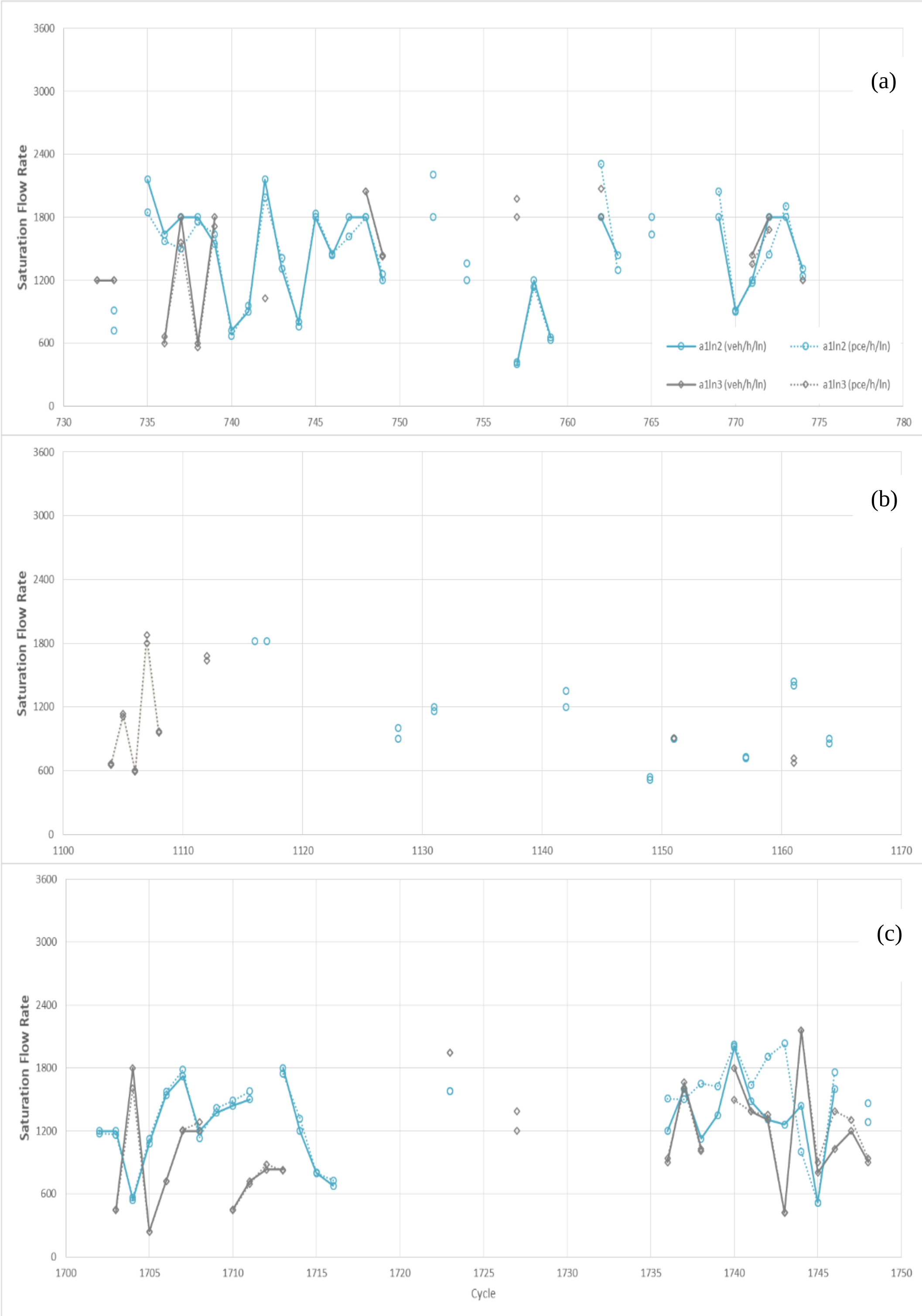

Figure C.3: Saturation flow rate & cycle observation at A1 during (a) AM (b) NOON (c) PM

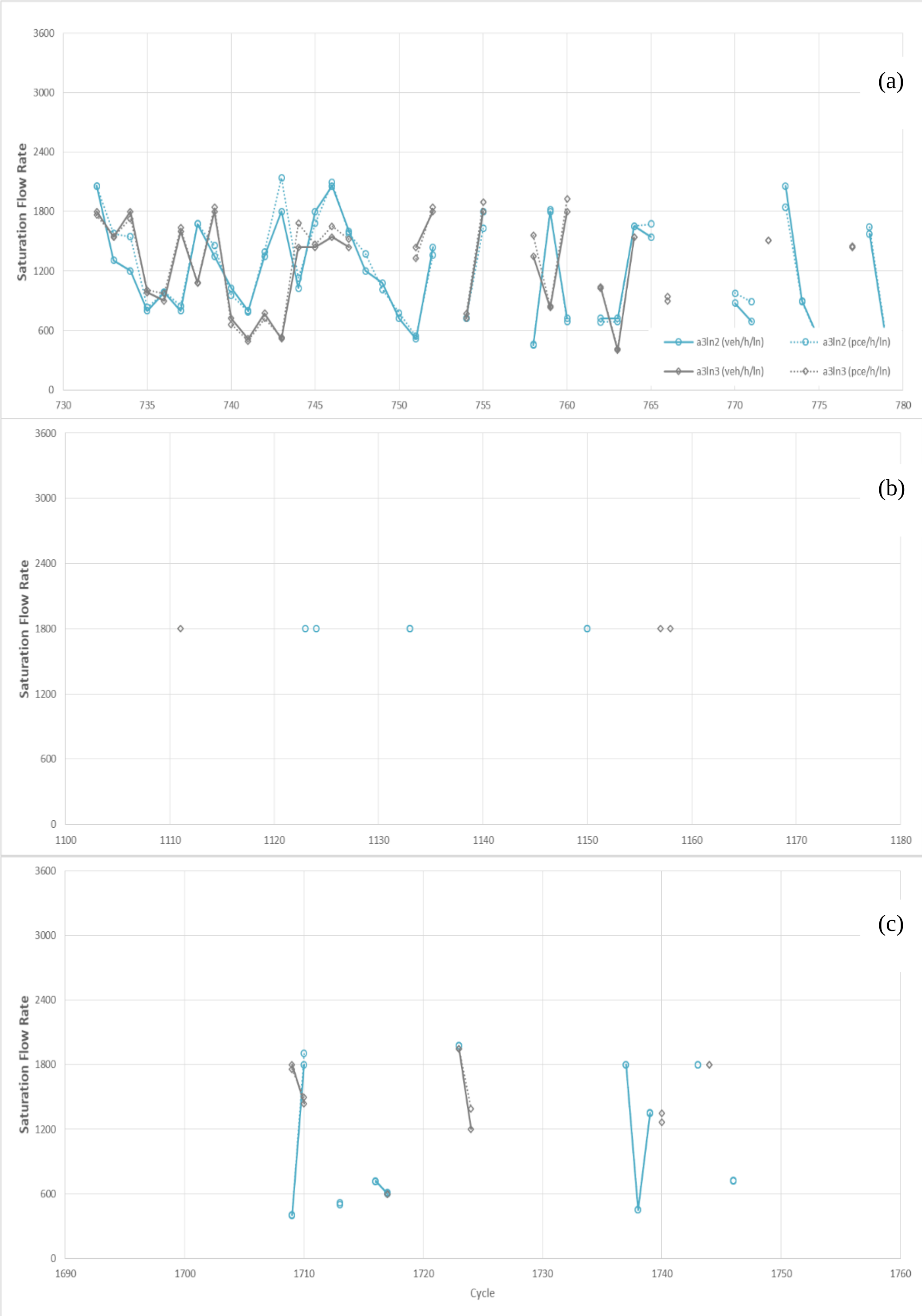

Figure C.4: Saturation flow rate & cycle observation at A3 during (a) AM (b) NOON (c) PM

APPENDIX D

PERCENTILE DISTRIBUTION OF SATURATION FLOW RATE FREQUENCY

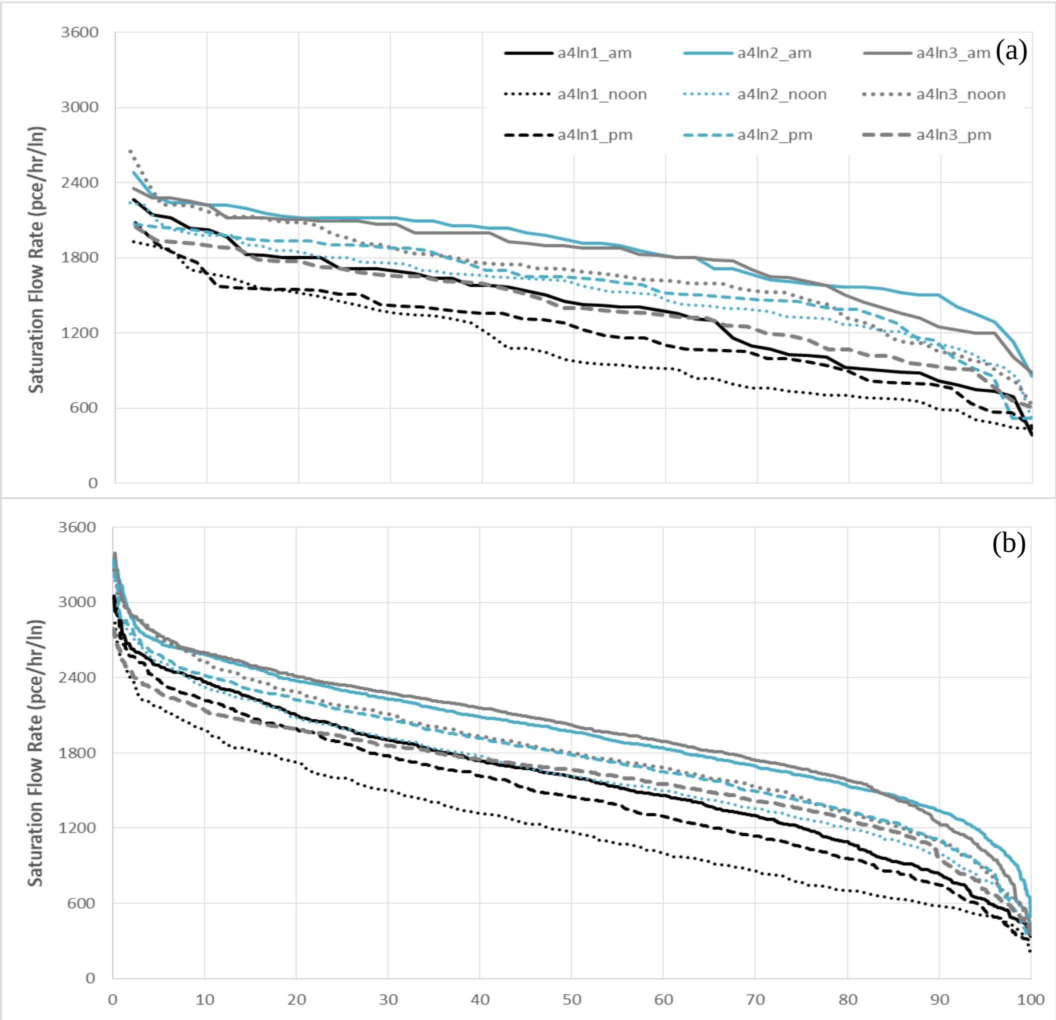

Figure D.1: Percentile distribution of saturation flow rate frequency at A4 (a) HCM (b) moving average observation

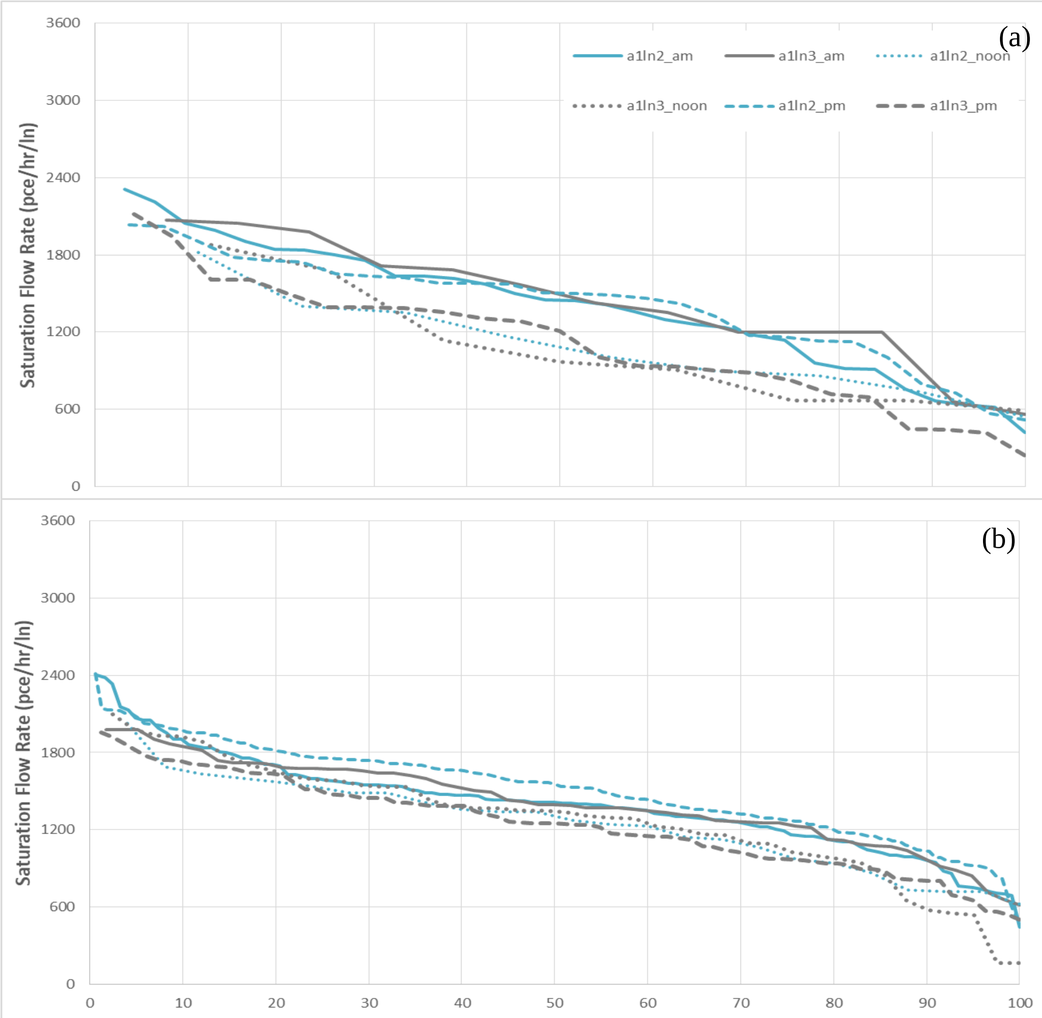

Figure D.2: Percentile distribution of saturation flow rate frequency at A1 (a) HCM (b) moving average observation

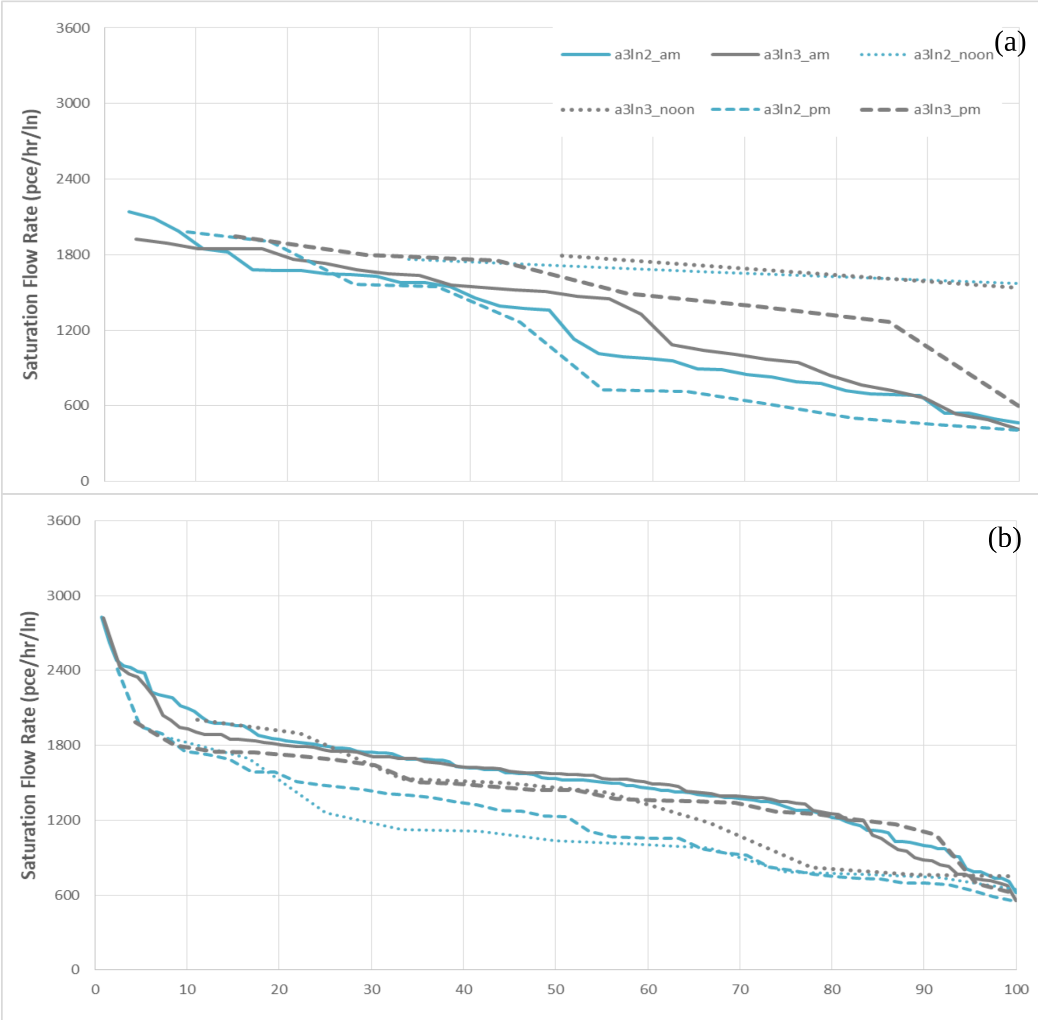

Figure D.3: Percentile distribution of saturation flow rate frequency at A3 (a) HCM (b) moving average observation

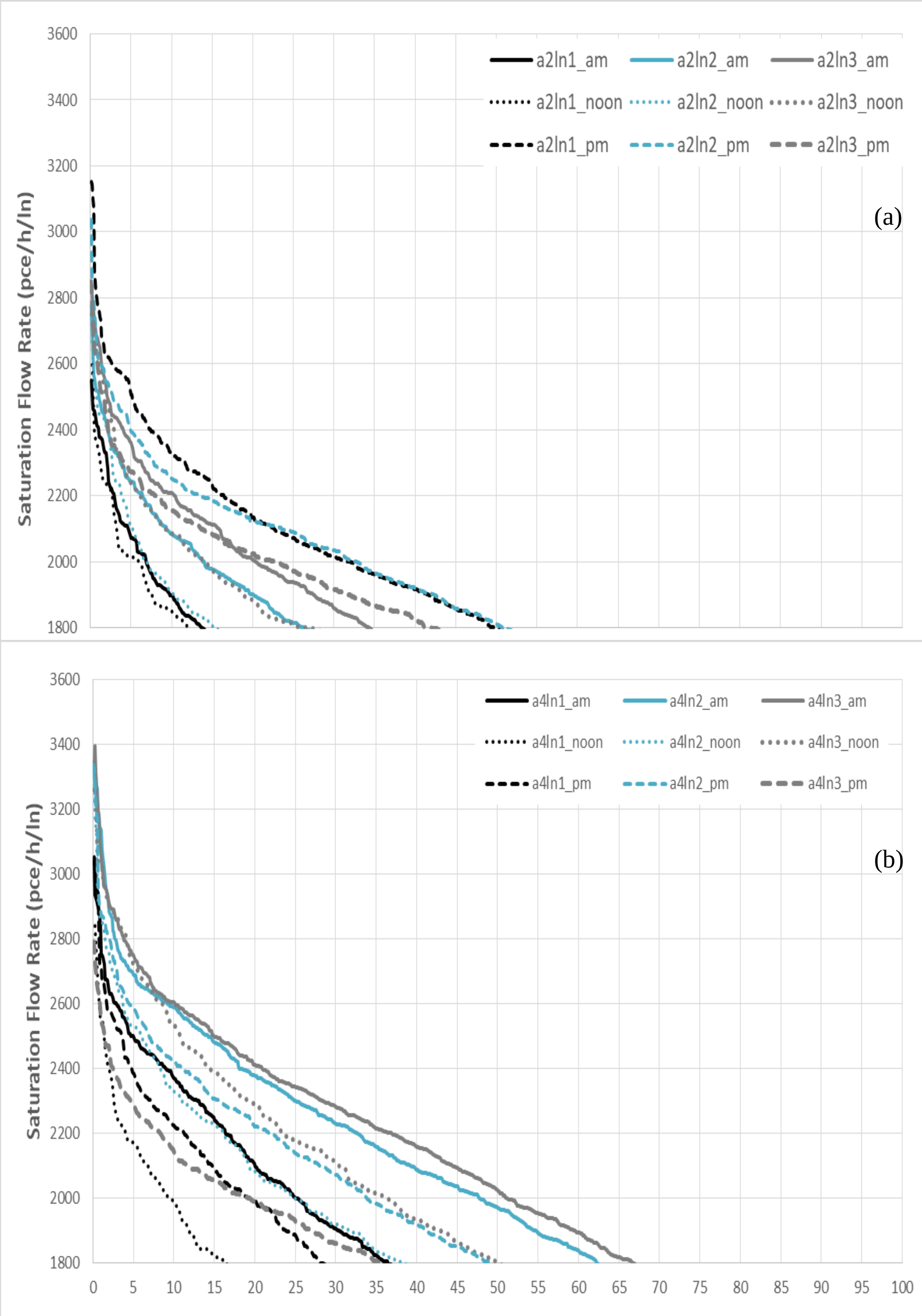

Figure D.4: Sorting of the moving average method according to HCM (a) A2 (b) A4

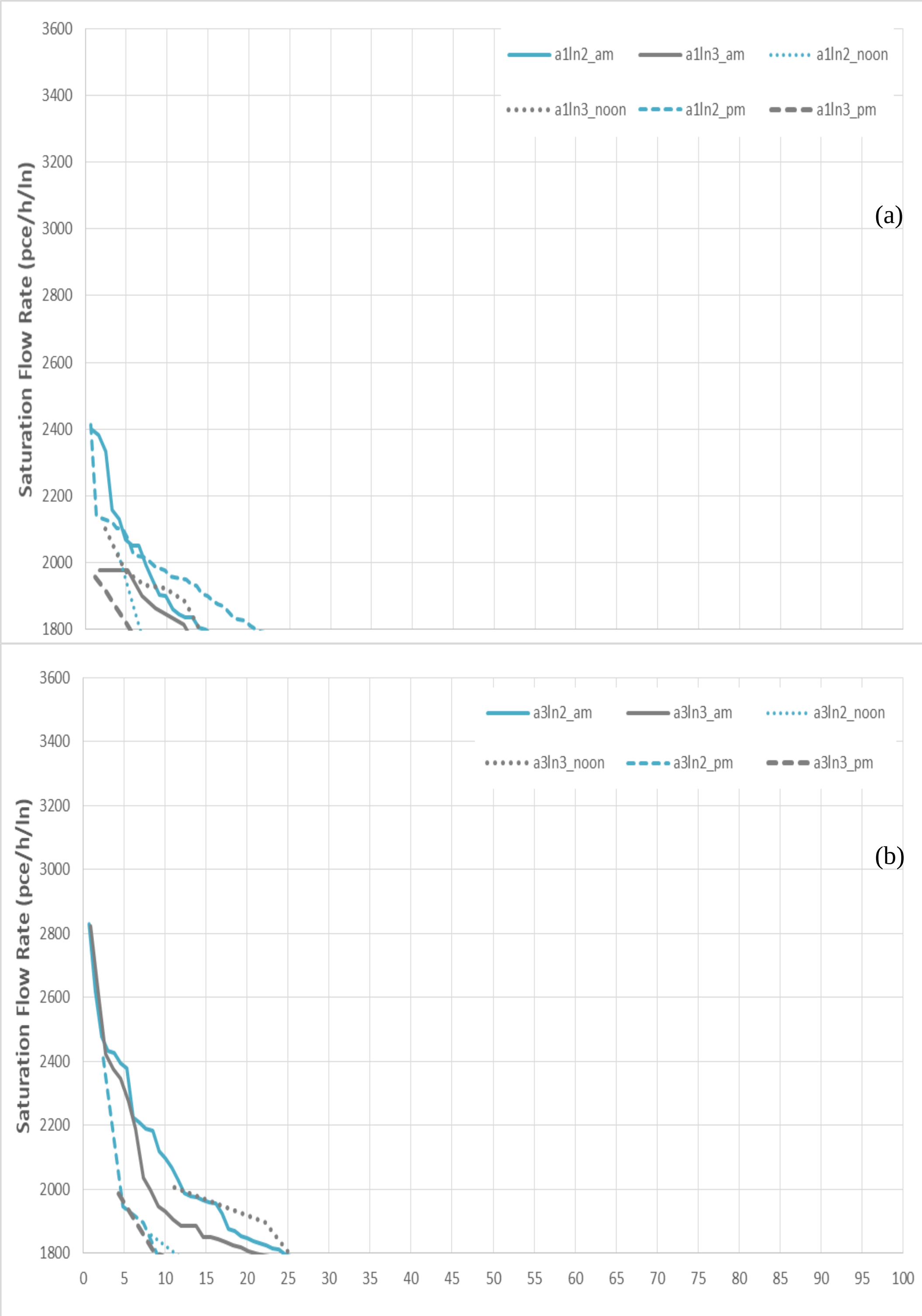

Figure D.5: Sorting of the moving average method according to HCM (a) A1 (b) A3

APPENDIX E

INEFFICIENCY CALCULATIONS

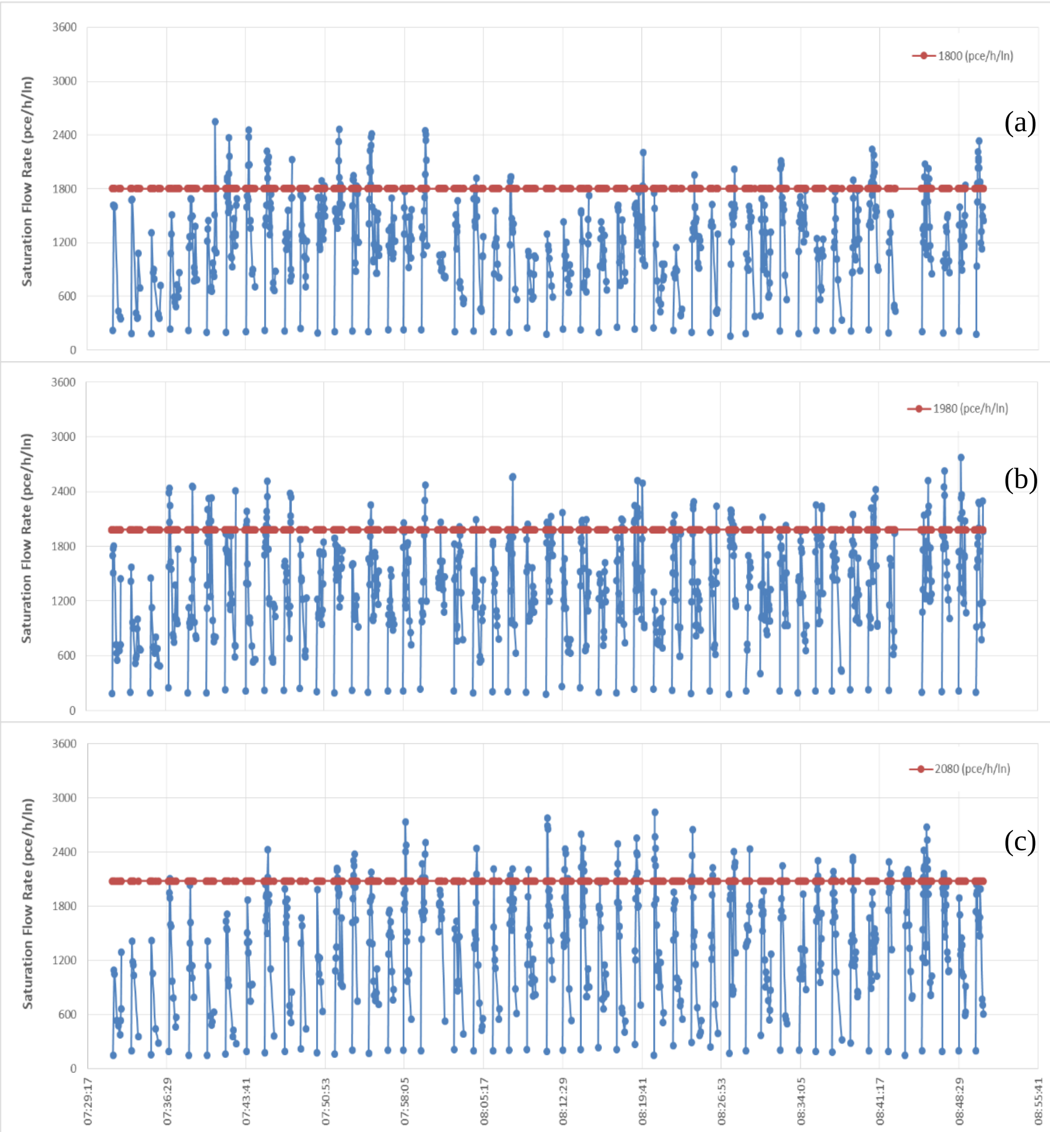

Figure E.1: Inefficiency observation of A2 during AM on (a) Lane 1 (b) Lane 2 (c) Lane 3

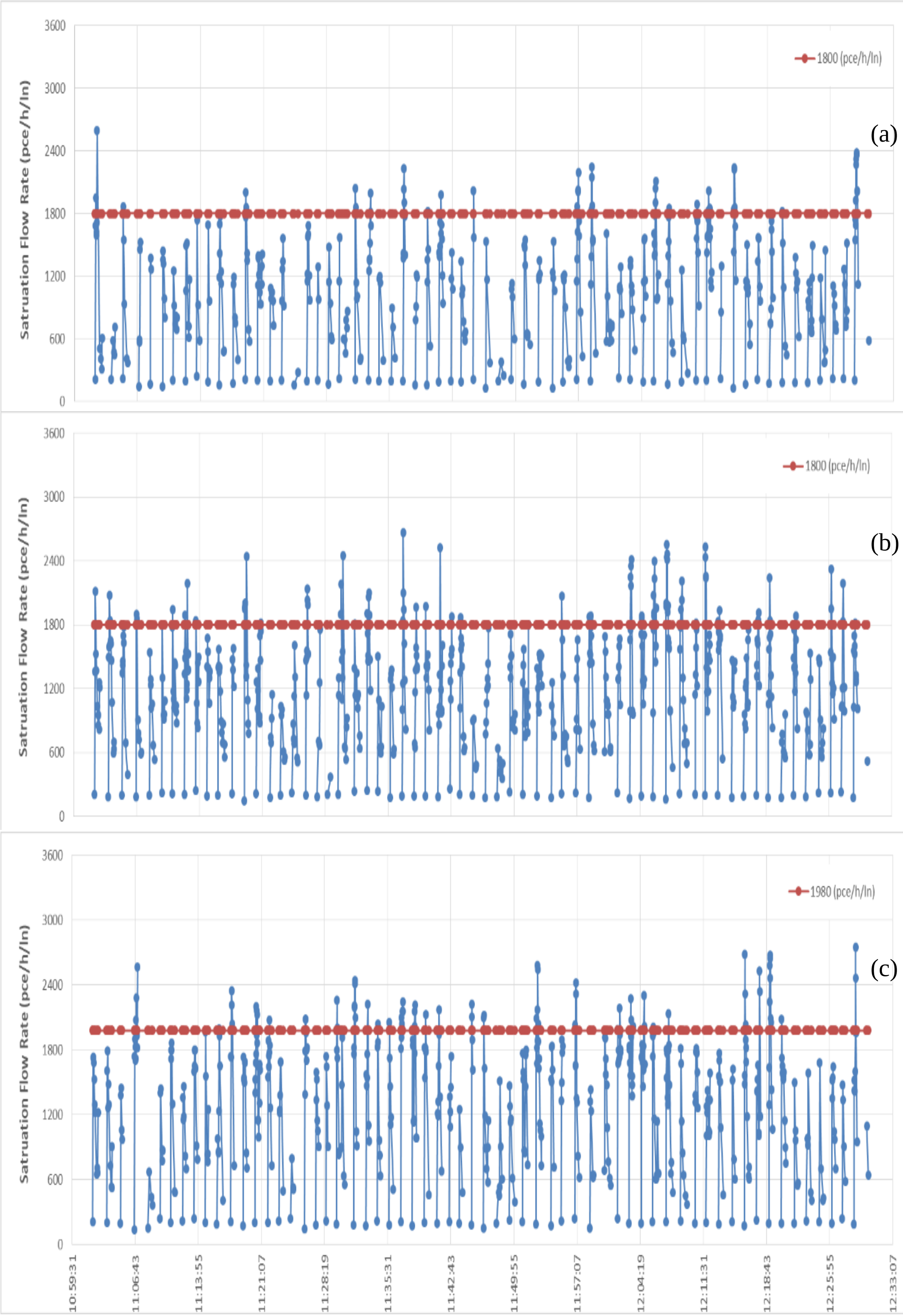

Figure E.2: Inefficiency observation of A2 during NOON (a) Lane 1 (b) Lane 2 (c) Lane 3

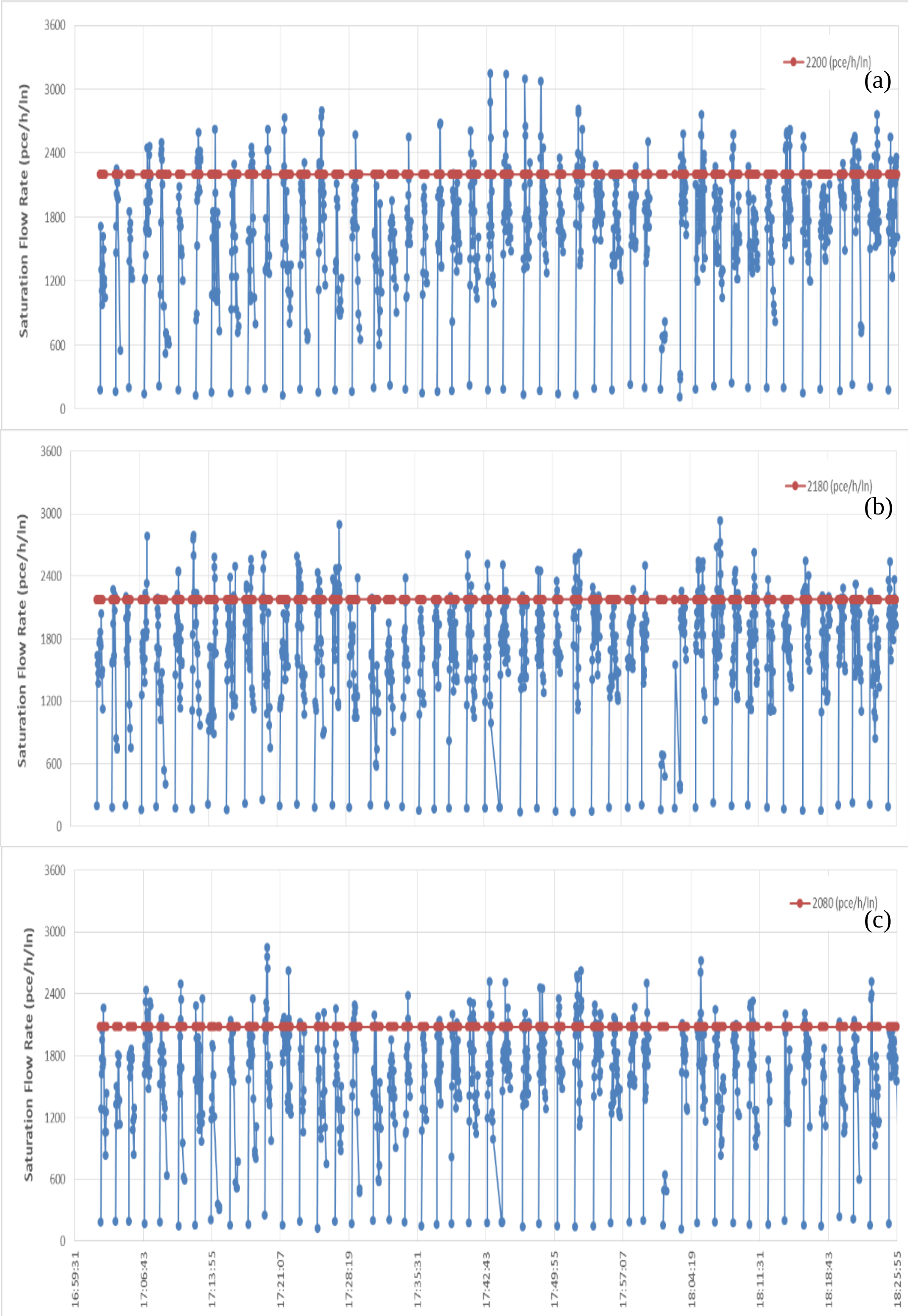

Figure E.3: Inefficiency observation of A2 during PM (a) Lane 1 (b) Lane 2 (c) Lane 3

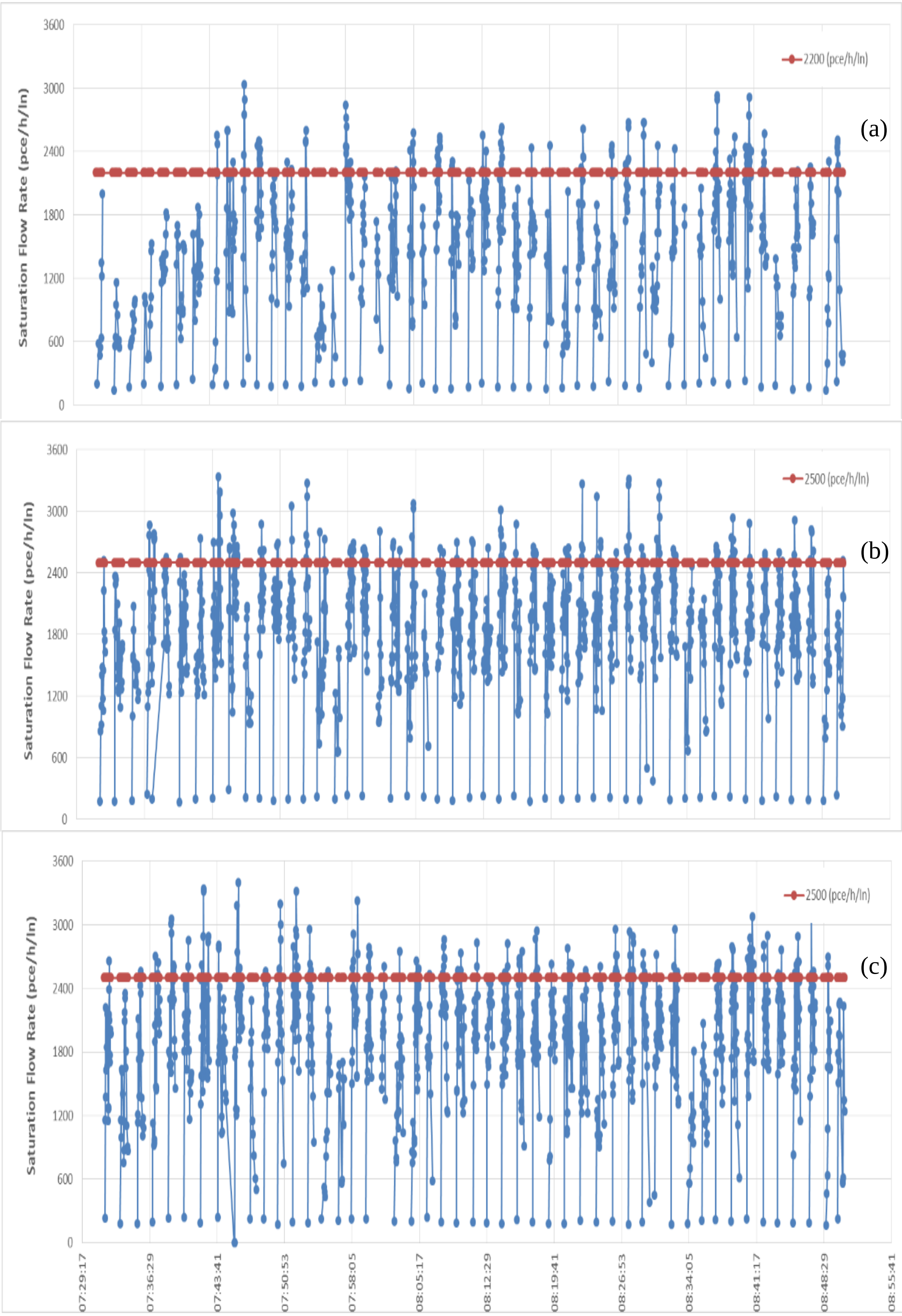

Figure E.4: Inefficiency observation of A4 during AM (a) Lane 1 (b) Lane 2 (c) Lane 3

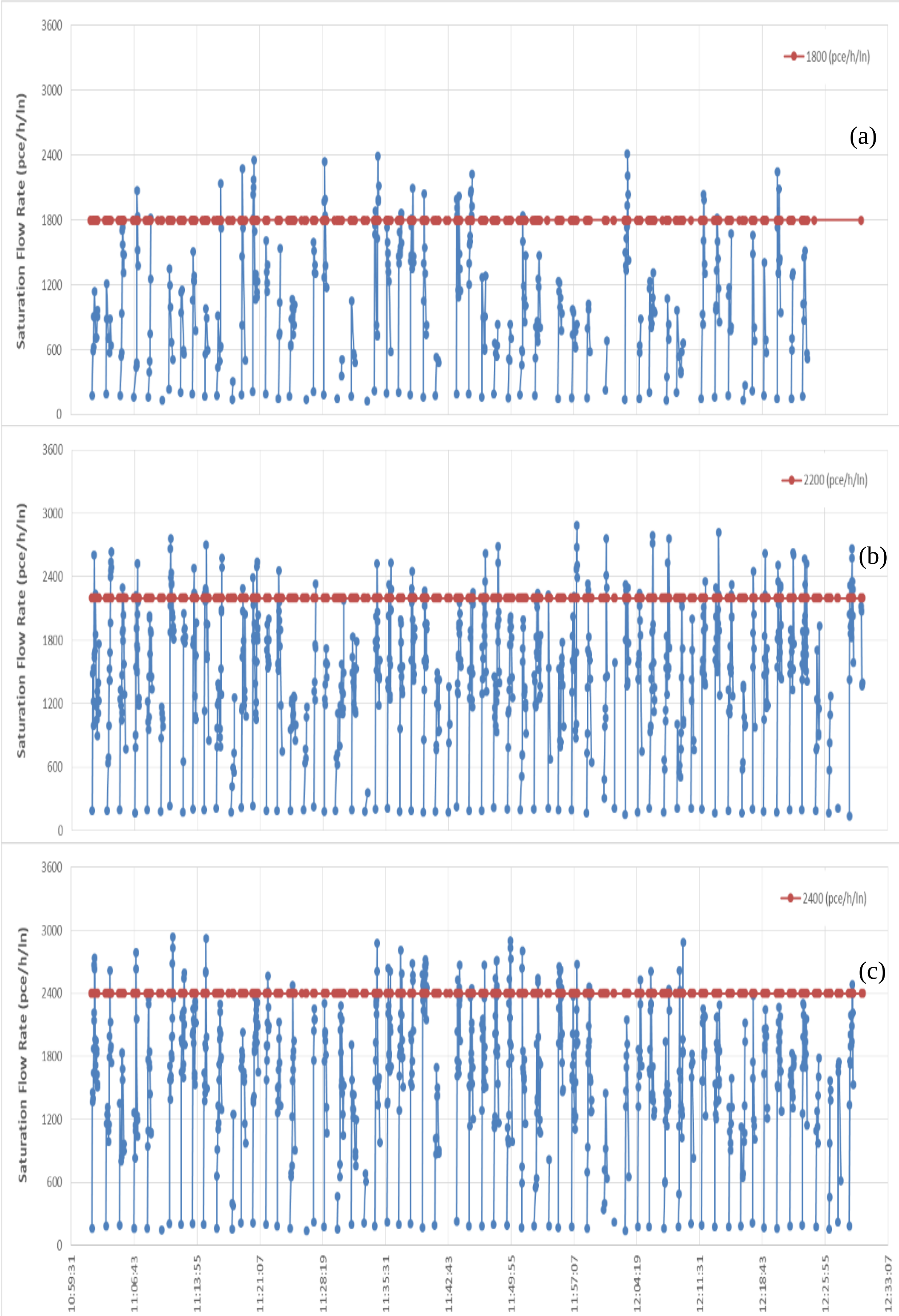

Figure E.5: Inefficiency observation of A4 during NOON on (a) Lane 1 (b) Lane 2 (c) Lane 3

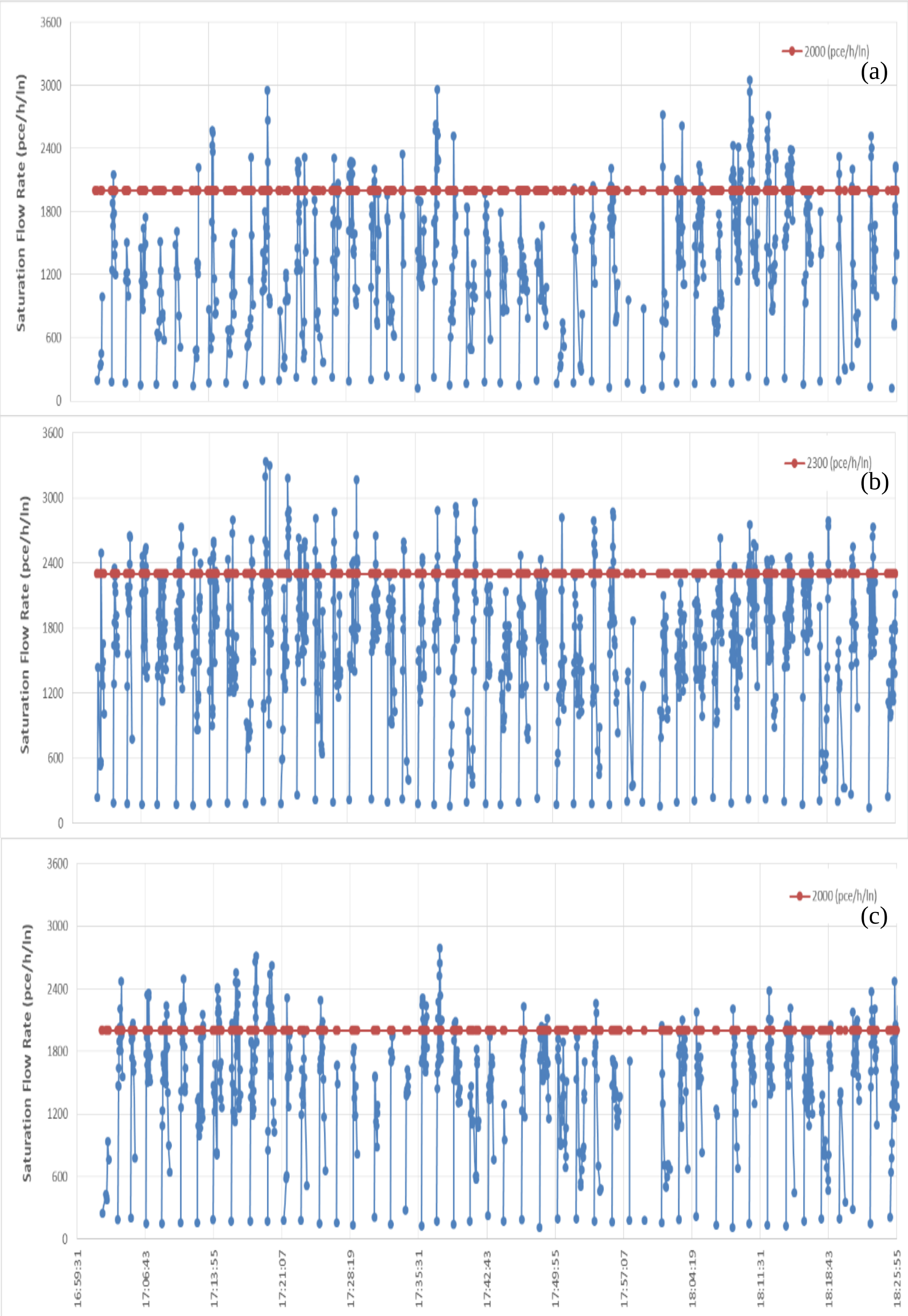

Figure E.6: Inefficiency observation of A4 during PM on (a) Lane 1 (b) Lane 2 (c) Lane 3

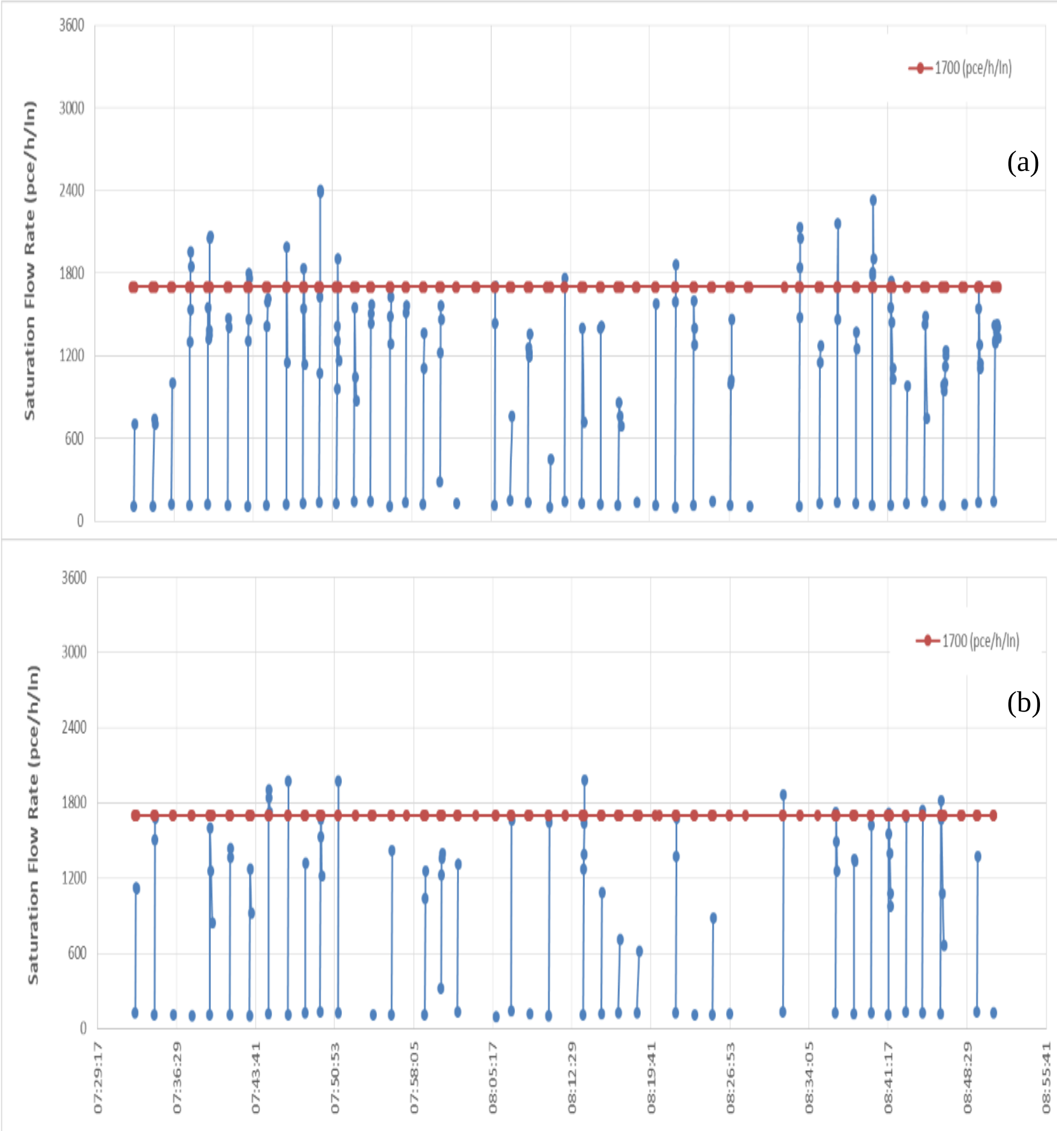

Figure E.7: Inefficiency observation of A1 during AM period on (a) Lane 2 (b) Lane 3

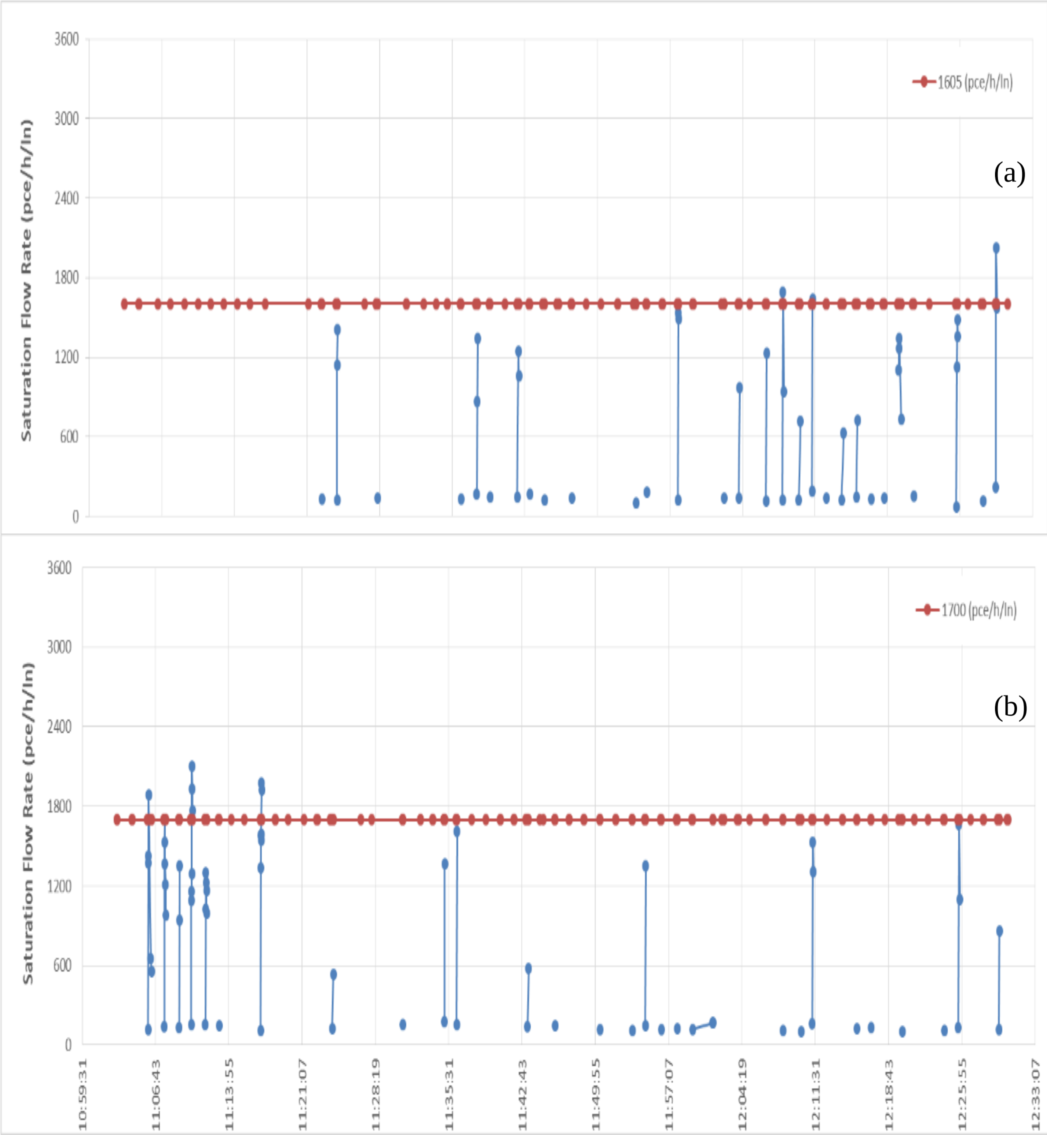

Figure E.8: Inefficiency observation of A1 during NOON period on (a) Lane 2 (b) Lane 3

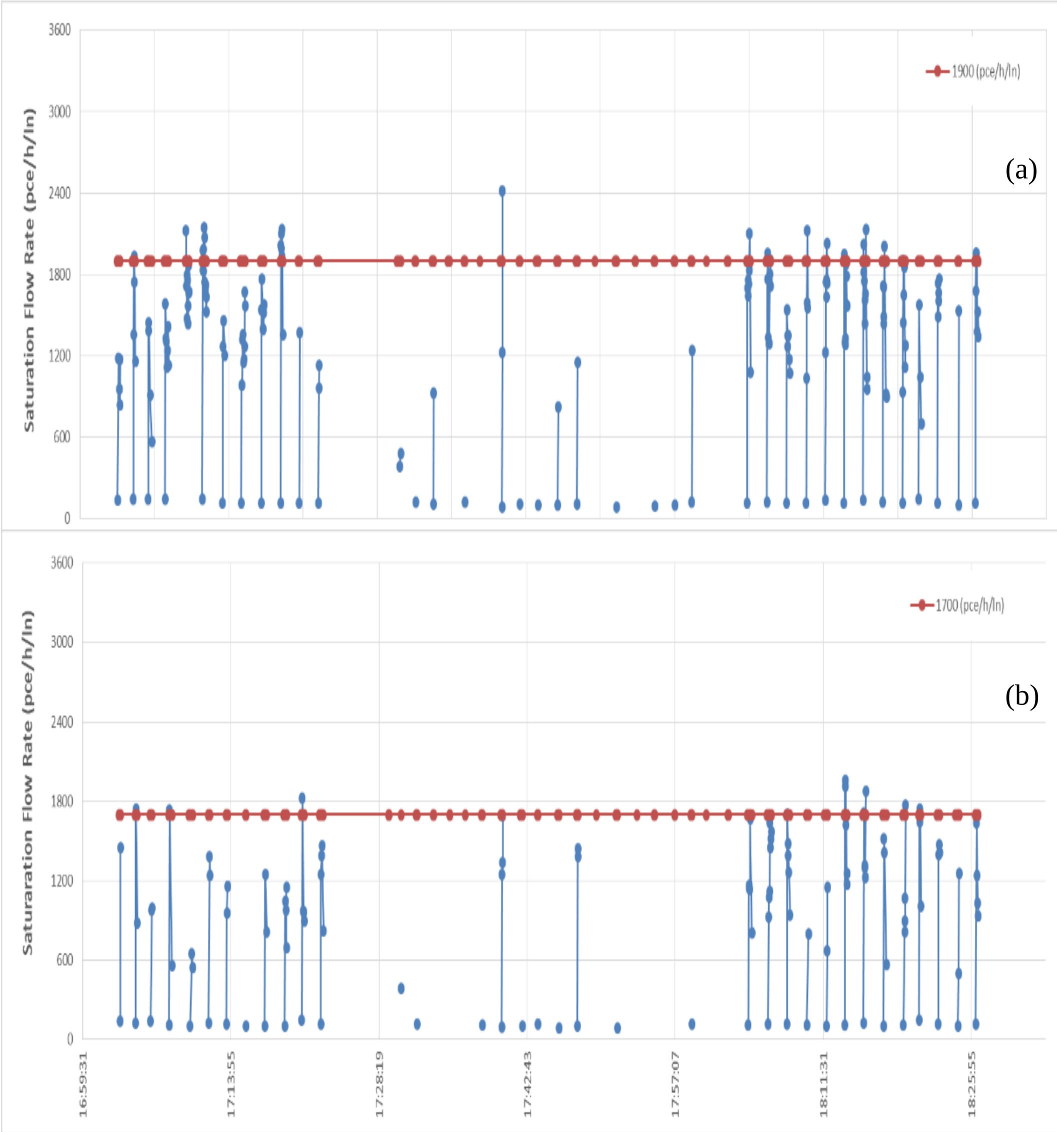

Figure E.9: Inefficiency observation of A1 during PM period on (a) Lane 2 (b) Lane 3

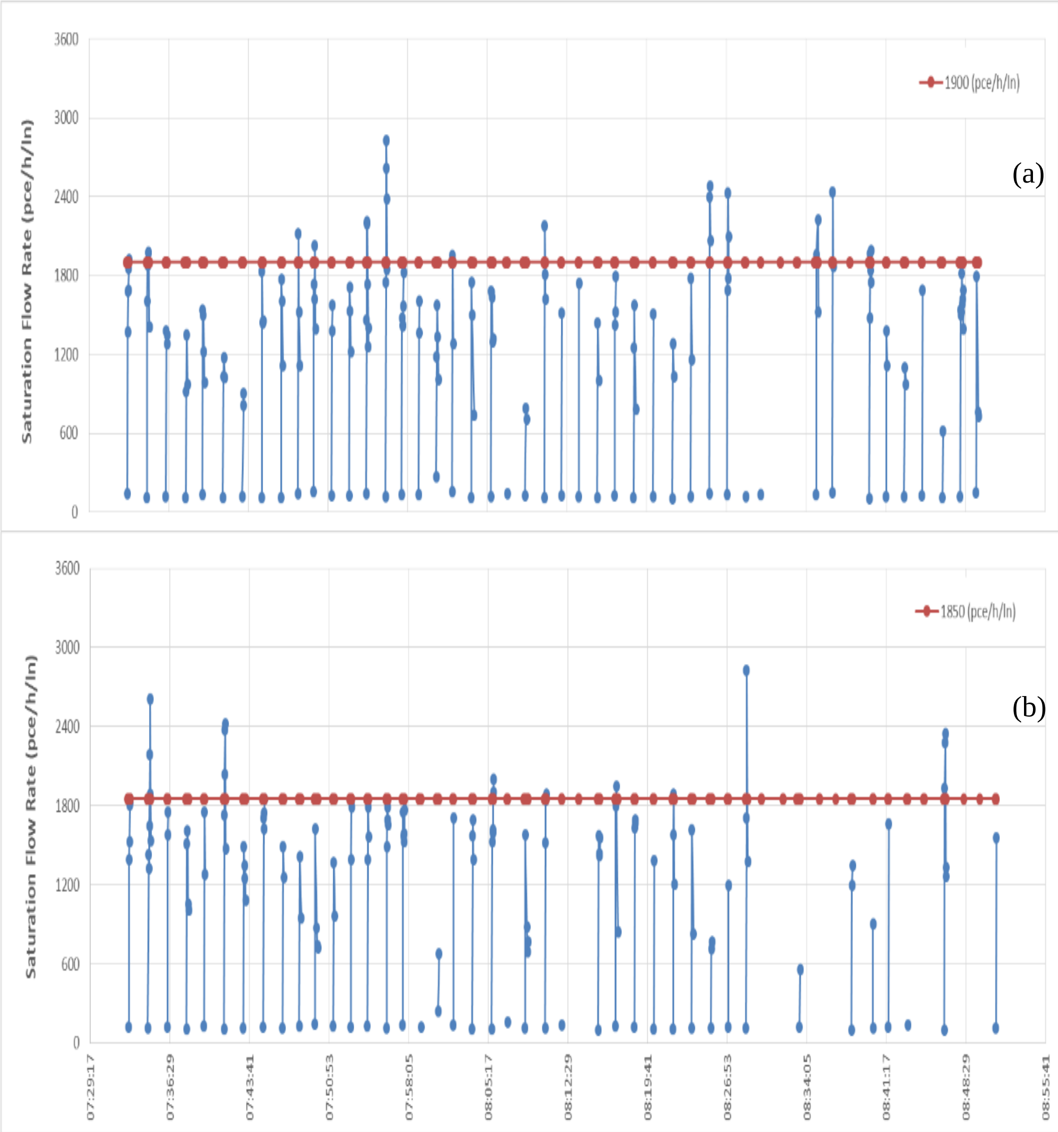

Figure E.10: Inefficiency observation of A3 during AM period on (a) Lane 2 (b) Lane 3

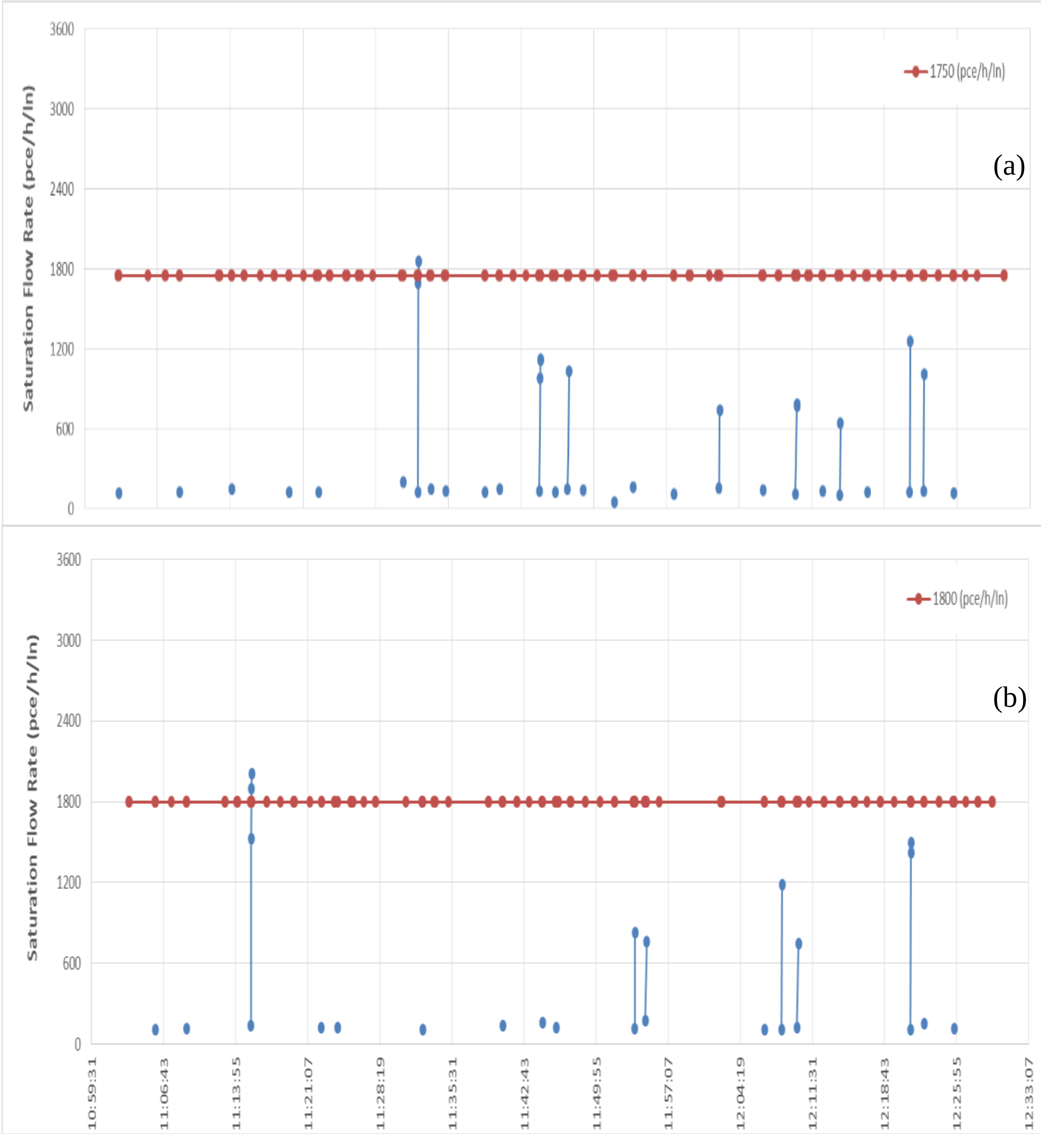

Figure E.11: Inefficiency observation of A3 during NOON period on (a) Lane 2 (b) Lane 3

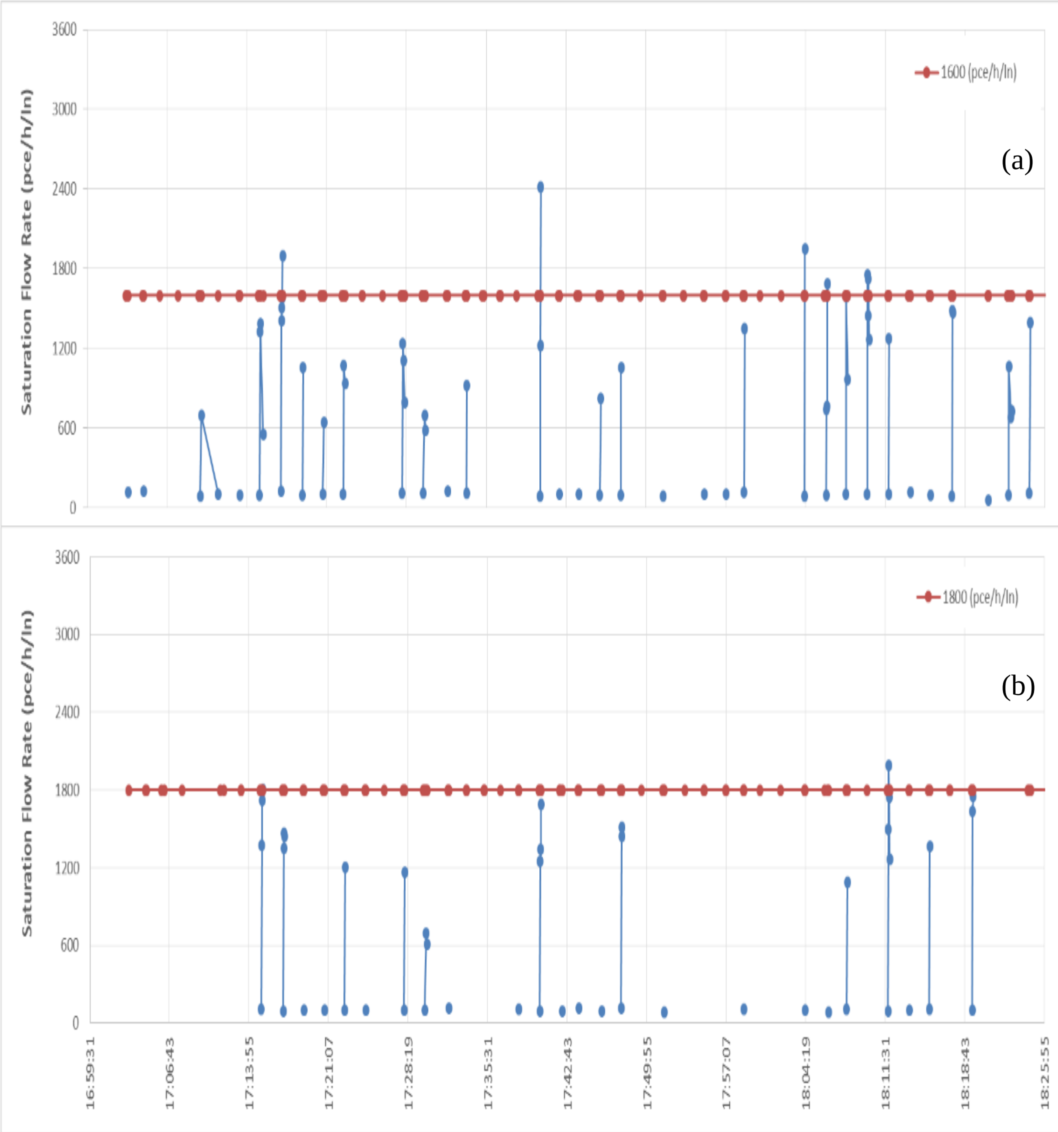

Figure E.12: Inefficiency observation of A3 during PM period on (a) Lane 2 (b) Lane 3