TRAFFIC CAPACITY ANALYSIS ON U-TURN AT MIDBLOCK MEDIAN OPENINGS ON URBAN ARTERIALS

(U ターン挙動を考慮した都市内街路の
単路部における交通容量解析)

Thakonlaphat Jenjiwattanakul

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TRAFFIC CAPACITY ANALYSIS ON U-TURN AT MIDBLOCK MEDIAN OPENINGS ON URBAN ARTERIALS

by

Thakonlaphat Jenjiwattanakul

A dissertation submitted in partial fulfillment of the requirements for the degree of Doctor of Engineering.

Examination Committee

Associate Professor Kazushi Sano (Chairperson)
Professor Osamu Takahashi
Professor Toshio Yoshii
Professor Hiroyuki Oneyama
Associate Professor Shu Higuchi

Nationality

Thai

Previous Degrees

Master of Engineering (Infrastructure Eng.), 2001
Asian Institute of Technology
Pathumthani, Thailand

Bachelor of Engineering (Civil Eng.), 1997
Chulalongkorn University
Bangkok, Thailand

Scholarship Donor

Japanese Government Scholarship

Nagaoka University of Technology
Department of Civil and Environmental Engineering
Niigata, Japan
August 2013
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ABSTRACT

U-turn at midblock median opening is frequently provided in developing countries to facilitate the local access. U-turn movement at the midblock median opening is primarily based on gap acceptance process. The u-turn driver musts wait for a large enough gap of through traffic to make a u-turn maneuver. The u-turn traffic interrupts the through traffic streams and sometimes creates problems on the road network. The traffic operation at u-turn should be improved by some measures. Currently, traffic police controls and manages the u-turn facility in peak periods for smoother traffic flow. This study aims to improve the capacity of u-turn at midblock median opening by implementing the control at u-turn.

This study has three main topics: u-turn behavior, u-turn capacity, and u-turn control. The behavior study investigates the behavior of u-turning vehicle to identify the significant factors on driver’s decision. The potential factors were evaluated to know their significance level. The capacity study evaluates the existing capacity model and proposes the estimation improvement. The estimated capacities from the two methods were compared with the field capacity. The control study provides a control warrant for u-turn and evaluates the effect of police control. The waiting time was estimated and its function of the conflicting traffic volume was formulated. The control warrant is recommended based on the waiting time threshold.

The significant factor identification employed the binary logistic regression technique. The technique examines the significance levels of all potential factors. This method also yields the u-turn decision model based on the significant factors. The u-turn capacity was estimated by the gap acceptance model and compared with the field capacity. The adjustment method to improve the u-turn capacity estimation is based on the balancing of volume-to-capacity ratio (v/c). This method incorporates the traffic interactions between the two traffic streams. The waiting time estimation utilized the spreadsheet simulation. The randomly generated gaps were compared with the preset critical headway to determine whether the u-turn vehicles accept the gaps or not. The waiting time of a driver is the accumulation of his/her rejected gaps. The evaluation of police control is based on the comparison of discharge headway between the continuous u-turn movement with and without control.

From the considered eight factors, gap size, speed of conflicting vehicle, and waiting time at the front position of the queue are statistically significant at the confidence interval of 95%. It is interesting to find that the queue time does not significantly affect the u-turn decision. The waiting time of more than 30 seconds would frustrate the drivers to accept the significant smaller gap. The developed u-turn decision model, which explanatory variables included gap size, conflicting speed, and waiting time, can predict the u-turn decision well with the percentage correctness of more than 85%. For capacity estimation, the gap acceptance model overestimates the field capacity in case of negative exponential headway distribution and underestimates in case of Erlang-2 distribution. The difference in driver behavior when responding to different conflicting headway can properly explain the situation. The proposed adjustment method can estimate the capacity closer to the measured field capacity than the gap acceptance model does. For waiting time estimation, the u-turn traffic characteristics (volume, random or queued, follow-up condition) do not affect the amount of waiting time. The relationship between waiting time and conflicting flow rate is in the exponential form. The influence of headway distribution type is much higher than the effect of critical headway value. When considering the randomness of the
critical headway, the estimated waiting time is decreased. The control warrant in term of conflicting traffic volume can be determined by the inverse of waiting time function, given the impatient waiting time threshold. When a police controls u-turn, the queue discharge characteristic is similar to the movement at a signalized junction. The u-turning vehicles move with smaller departure headway; consequently, the police control increases the u-turn discharge flow rate of about 10%. The wider median results in the higher u-turn movement headway and the lower discharge flow rate. However, too narrow median causes uncomfortable u-turn maneuver, decreases the discharge flow rate, and diminishes the efficiency of police control.

This dissertation is organized into 6 chapters as follows:

- Chapter 1 introduces the background of the study, discusses the problem statement, states the research objectives, and explains the scope and limitation. The research framework and process, including the data collection, are also addressed.

- Chapter 2 reviews the related past research, mainly on gap acceptance process, unsignalized intersection capacity analysis, critical gap parameter estimation, application of u-turn facility and its safety, u-turn capacity estimation, waiting time estimation, and intersection traffic control.

- Chapter 3 investigates the factors affecting the decision of the u-turning drivers and evaluates the statistical significance levels of each factor. The u-turn decision prediction model has been developed after finalizing the significant factors. This chapter also examines the effect of waiting time on the size of accepted gap.

- Chapter 4 evaluates the gap acceptance model for u-turn capacity estimation. The estimated capacity is validated by field capacity. An adjustment method is proposed to improve the estimation. The proposed method is based on v/c balancing, which incorporates the interactions between the u-turn and through traffic streams.

- Chapter 5 estimates the theoretical waiting time of the u-turn traffic at the stop line, when facing the conflicting traffic stream. The waiting time as a function of conflicting traffic volume and critical headway is developed. Then, the u-turn control warrants are determined. This chapter also investigates the effect of police control on u-turn discharge flow.

- Chapter 6 concludes all the results from this study, recommends for the real world application, and lists some future research directions.
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CHAPTER 1 INTRODUCTION

1.1 Background

U-turn facilities can be found in most urban arterials in the developing countries. One prominent cause is the improper city planning. In other words, the highway functional class is not well implemented in the urban street network in the developing countries. Many arterials cannot remain their function of providing high mobility with low access permission. This results in the saturated flow condition during typical peak hours in the urban arterial. U-turn movement is one of the main causes of traffic stream interruption of through movement. The open-grid street network system can accommodate the u-turning vehicles by allowing the drivers to make either one left turn plus three right turns or one right turn plus three left turns at the adjacent junctions to complete the u-turn movement. Alternatively, the drivers can change their route choices and use collector/local road instead. To serve u-turn traffic on urban arterial with the least interruption to the through traffic is the challenging traffic management issues in urban environment. U-turn traffic is not only affect the traffic in the opposite direction, but also affect the traffic in the same direction if the length of exclusive u-turn lane is not sufficiently provided.

To illustrate the need for u-turn on urban arterial in developing countries, a comparison of the street network in Bangkok, Thailand, and Tokyo, Japan is showed in Figure 1.1. The road network in Bangkok is not in a rectangular grid system while the road system in Tokyo is well organized. Most roads in Tokyo are connected with other roads in an acceptable distance. The drivers can have more choices of travel to their destinations. There is no need to provide u-turn facilities on the road system. On the contrary, the road in Bangkok is not well connected. In addition, most local roads and collectors are dead-end. The drivers have to rely on the arterial to go to their destinations. When the destinations are on the opposite site of the road, the drivers need to make u-turn at any point on the urban arterial. So, u-turn facilities are usually provided on urban arterials in order to accommodate such movements. Consequently, through traffic on urban arterials are interrupted by the u-turn maneuver.

Usually, the u-turn maneuver on the urban arterials can be made at the following locations: (1) signalized intersections, (2) unsignalized intersections, and (3) midblock median openings. The u-turn movement at midblock median opening can be classified based on the level of traffic control as follows:

- Uncontrolled: the u-turn vehicles will freely select their acceptable gaps;
- Human controlled: the u-turn movement is controlled by human, e.g. police; and
- Signal controlled: the u-turn movement is controlled by traffic signal.

The investigation of each u-turn control type is necessary to understand the behavior of each u-turn maneuver and evaluate the control performance. The result of the investigation can help city administration decide the control measure at the specific u-turn location in the specified time period or traffic condition appropriately. The studies in this research cover the u-turn operation under the traffic control conditions of uncontrolled and human controlled by police. The police control is analyzed as a representative of the control at u-turn and assumed to act similar to the signal control. The behavior of individual traffic police is also neglected in this study.
The Highway Capacity Manual (HCM) is the well-known handbook, presenting the methodologies for traffic capacity analysis of various highway facilities. Many previous research works on the u-turn traffic capacity analysis were studied based on the methodology provided for the unsignalized intersection in HCM 2000. Recently, the updated HCM 2010 also includes the capacity analysis of major traffic u-turn movement in the Chapter 19, Two-Way Stop-Controlled (TWSC) Intersection. However, the traffic operation and behavior in the United States are different from those in the developing countries. Therefore, the application of the method described in the HCM requires local calibration and validation.
The traffic operation at some midblock median opening u-turn facilities on urban arterials in Bangkok, Thailand, is illustrated in Figure 1.2. It can be noticed that the through and u-turn traffic streams are not ideally operated in a major-minor traffic manner/discipline. The u-turn vehicles often do not wait for the large enough acceptable gap of the through traffic. They gradually move onto the conflicting lane to show the intention to go. The through vehicles sometimes do not allow for u-turn, by increasing speed or changing lane or honking car horn or opening headlight. Eventually, when the time passes by, the through traffic stops and allows the u-turn traffic to move.

Figure 1.2 Traffic operation at the u-turn locations

1.2 Problem Statement

Unlike the developed countries, the midblock median opening u-turn facilities are utilized on many urban arterials in the developing countries’ cities, such as Bangkok. The u-turning traffic will cause the additional interrupted flow phenomenon on the urban street. The interrupted flow of the main through traffic stream creates plenty of accumulated delays, i.e. deceleration, stop, start-up loss, and acceleration, comparing with uninterrupted flow. This cause a number of unsatisfied stop-and-go traffic conditions on the urban arterials.

The u-turn traffic at the uncontrolled midblock median opening faces its own risk to select the acceptable gap of the main traffic stream to complete the u-turn movement. The gap acceptance behavior is somewhat subjective and is based on each driver’s skill and
experience. As a result, the u-turn movement is quite complex and risky. The traffic control at the u-turn location is applied to increase traffic flow and improve traffic safety of the u-turn movement. For instance, some traffic policemen present to operate the u-turn traffic at the midblock median openings during peak periods. However, there is lack of study on the followings:

- When should the u-turn be controlled?
- How effective of such u-turn control?

Development of a u-turn traffic control guideline is required for the effective u-turn traffic management at the midblock median opening. This guideline can improve the traffic safety of the u-turn movement as well as increase the capacity of the u-turn junctions.

In addition, the capacity of the u-turn movement at midblock median opening is of interest. The method of capacity estimation provided in HCM 2010 should be investigated to see whether it can apply for these facilities in developing countries. The practical application requires the reliable method to estimate the capacity of u-turn, with acceptable tolerance. Knowing the capacity of all traffic components at such u-turn junctions leads to the better traffic operation management as well as facilitates the quality/level of service assessments.

1.3 Research Objectives

The traffic congestion on urban arterial, which is expected to provide high mobility, is a burden to the country. There are many practical efforts to improve the mobility of such urban arterial. However, most mitigation measures are based on trial-and-error process and no concrete supporting academic evidence. This study will bridge the gap of real-world practice and academic research to yield the most effective traffic management measures. The study is necessary as a tool to explain the traffic characteristics of the midblock u-turn operation on the urban arterial. The study can also lead to the appropriate solutions for improving the traffic operation at u-turn.

The goal of this study is to develop a control warrant for u-turn movement at the midblock median opening. This is to enhance the traffic flow and safety on the urban arterials. The study also evaluates the performance of the control measures. As the capacity information is the basic important data for traffic engineering, this study validates the existing capacity analysis guideline and proposes a new method in order to improve the estimation results.

The objectives of this research are listed as follows:

- To study the behavior of u-turning vehicle at midblock median opening and identify the factors affecting the gap acceptance decision;
- To determine the critical gap parameters (critical headway and follow-up headway) of the u-turning movement, which represent the capacity of the movement;
- To develop and evaluate the capacity estimation method for u-turn movement;
- To formulate the models to estimate waiting time of u-turning traffic;
- To recommend the u-turn traffic control warrants at midblock median opening; and
- To evaluate the effect of u-turn traffic control on discharge flow rate.
1.4 Scope and Limitation

The studies on the u-turn traffic movement are separated into three topics; u-turn behavior study, u-turn capacity study, and u-turn control study. The research framework and process can be illustrated in diagram as shown in Figure 1.3.

![Diagram](image)

Figure 1.3 Research framework and process

The research works can be divided into the following tasks:

- Initially investigate the effect of waiting time on u-turn accepted gap.
- Evaluate the factors affecting the u-turn decision, in order to confirm the effect of waiting time.
- Evaluate the u-turn capacity, estimated by gap acceptance model, and propose the estimation improvement.
- Formulate the waiting time function, which leads to the u-turn control warrant determination.
- Study the performance and effect of u-turn traffic control.
The goal of this research is to determine the control warrant for the u-turn movement at the midblock median opening and evaluate its performance. Actually, there are a number of factors that may affect the u-turn movement and can be controlled for safer and smoother traffic operation. Those factors should be identified. According to the past research on the unsignalized intersection, the waiting time of the minor street vehicle also affects the movement decision; the more waiting time, the less accepted gap size. This behavior is undesirable because it interrupts the through traffic movement and may cause accident.

Firstly, the effect of waiting time on u-turn accepted gap has been studied at a midblock median opening u-turn facility. After that, more comprehensive study has been conducted to identify the significant factors, by collecting more data at more sites. The significant factor, which directly relates to the u-turn traffic, should be used as a basis for control warrant determination.

Capacity is one of the basic parameters in traffic engineering. For u-turn movement, the capacity information is also of interest. The current guideline by HCM, has been evaluated on the validity to apply for u-turn movement at midblock median opening on urban arterials. In addition, the adjustment method to improve the capacity estimation has been proposed and evaluated, comparing to the HCM method.

The control warrant for u-turn movement is then recommended, based on the unsafe waiting time threshold. This requires a theoretical relationship between the conflicting traffic volume and u-turn waiting time. Combining this relationship with waiting time threshold, the control warrant in term of conflicting traffic volume can be determined. However, a study is needed to evaluate the performance of traffic control at u-turn. In this research, the u-turn discharge flow is determined and compared between with and without police control at different u-turn locations.

This study concentrates on the vehicular traffic analysis of the u-turn facilities at the midblock median opening on six-lane divided urban arterials. The typical layout of such a u-turn facility is shown in Figure 1.4. The data collection was conducted at u-turn locations on urban arterials in Bangkok, Thailand.

Figure 1.4 Typical layout of the u-turn at midblock median opening (OTP, 2004)
The site selection is based on the appropriate criterions, which have been set in accordance with the research objectives and scope. The selected site should

- be a midblock median opening for u-turn only, not 3-leg or 4 leg intersection;
- locate on an urban arterial with high traffic volume;
- be far from signalized intersection to minimize the effect of upstream signal;
- have sufficient space to accommodate a smooth u-turn maneuver for passenger car, so should be located on a highway with 3 lanes or more on each direction;
- have an exclusive lane for u-turn traffic;
- accommodate one line of u-turn queue;
- locate near a pedestrian bridge and have clear view for data collection;
- have reasonably high u-turn traffic volume; and
- be controlled by police during peak and uncontrolled during off-peak periods.

Based on the above criteria, four u-turn facilities on Phetkasem Road, in Phasicharoen District, western area of Bangkok, Thailand, have been selected for data collection. The location of these sites is shown in Figure 1.5. The reason why all selected sites are located on the same street is to maintain the driver population, so that the driver behaviors are similar at all sites.

![Figure 1.5 Site location for data collection](image)

The data collection utilized many techniques of traffic engineering studies. To collect the microscopic-related traffic flow data, a digital video camera and a sound recorder have been utilized for the field data collection. The digital video camera is set up on a nearby pedestrian bridge to record the traffic movement at the interested site. The sound recorder is used by the observer to quickly collect necessary attributes, instead of taking note on a paper. The sound recorder is continuously operated so that the recording media is easily synchronized with the video data.

Firstly, the data was collected at Site 3 in August 2010, for initial investigation. After that, the field data collection for each site (Site 1-4) was conducted on two weekdays during December 2010-January 2011. The collection time periods include 07:00-09:00 hrs, 11:00-
13:00 hrs, and 16:00-18:00 hrs. These cover the morning peak, off-peak, and afternoon peak periods.

Most data collection in this research concerns with the time duration of interested traffic events. For instance, headway is the time difference between vehicle arrivals at a designate point on the road. Therefore, the most important required data is the time stamps of all related traffic movement events. The time stamping utilizes the Microsoft Excel’s formula and macro. The VBA command lets the Excel show the current computer time on the adjacent cell when a data is input in a cell. A form, representing different traffic events by different buttons, is used for data collection in laboratory. When an observer clicks a button on the form to record traffic movement event, the data will be input in the worksheet together with the time stamp. The further data analyses are based on the recorded time stamps, utilizing the spreadsheet formula.

The u-turn vehicles can be classified, based on vehicle characteristics and performance, into 3 main types including passenger car, motorcycle, and heavy vehicle (truck, bus, etc.). The u-turn vehicles in this study focus on only the passenger car, which includes all kinds of car with passenger car equivalent (PCE) of 1.

1.5 Dissertation Organization

This dissertation is divided into 6 chapters. Chapter 1 discusses background, problem statement, research objectives, and scope and limitations. Chapter 2 reviews all relevant literatures regarding to the gap acceptance theory, critical headway determination, waiting time, and traffic control. The relevant u-turn studies are also reviewed.

The u-turn behavior study results are summarized in Chapter 3. The chapter begins with the effect of u-turn waiting time on accepted gap size. The critical headway is also determined to compare with the accepted gap size to check on u-turn safety. The chapter further investigates the factors affecting the u-turn decision by binary logistic regression analysis. The decision model of u-turn movement based on the significant factors is also presented.

The u-turn capacity study is summarized in Chapter 4. This includes the evaluation of the gap acceptance capacity model by comparing with the field capacity. An adjustment method has been proposed based on the balancing of v/c. The proposed method is also validated by the field capacity. The required data for estimating the u-turn capacity by the proposed method are also discussed in detail.

The u-turn control study is summarized in Chapter 5. The chapter discusses the formulation of u-turn waiting time function. The effects of u-turn traffic characteristics on waiting time are evaluated. The u-turn control warrant guideline is also presented. The chapter also evaluates the effect of police control on u-turn discharge flow. This can represent the performance of the traffic control at u-turn. Finally, Chapter 6 discusses the conclusions, recommendations and future works of this study.
CHAPTER 2 LITERATURE REVIEW

2.1 Gap Acceptance Concept

The traffic movement with unsignalized control is mostly based on the gap acceptance concept. The vehicles will find their acceptable gap of the main traffic stream to enter into the main traffic stream. This maneuver depends on the arrival rate of the traffic in the main street and the acceptable time gap. The random arrival process of the light traffic streams is assumed to be Poisson distribution as shown in Equation 2.1 (ITE, 2008).

\[ P(n) = \frac{\lambda^n e^{-\lambda t}}{n!} \quad \text{Equation 2.1} \]

where,
- \( P(n) \) = probability of observing \( n \) vehicle in time interval \( t \)
- \( \lambda \) = average vehicle arrival rate (veh/s)

The gap in the traffic stream means no vehicle to be observed (\( n=0 \)). The probability of gaps greater than or equal to \( t \) seconds for the random arrival process (assumed Poisson distribution) can be determined as shown in Equation 2.2 (ITE, 2008).

\[ P(0) = P(gap \geq t) = e^{-\lambda t} = e^{-\frac{q}{3600}} \quad \text{Equation 2.2} \]

where,
- \( q \) = flow rate (veh/hr; vph)

Therefore, the number of \( t \) second gaps available in the main traffic stream for the vehicle in the minor street to enter can be computed as \( P(gap \geq t) \times (q-1) \) gaps, or simplified as \( P(gap \geq t) \times (q) \), when \( q \) is high.

However, for the heavy traffic condition during peak periods, the traffic volume may reach a saturation flow rate. The vehicle headways become more uniform because there are fewer opportunities to increase speed and change lanes. A normal distribution, instead of Poisson distribution for light traffic random arrival rate, can be applied in the gap analysis. In the real world practice, the arrival rate might not be random nor uniform as discussed above. Some other distributions that have been applied in the past for intermediate volume conditions include the Pearson Type III, Gamma, Erlang, and shifted negative exponential (ITE, 2008).

Most studies on the gap acceptance behavior relate to the TWSC intersections. The length of delayed time, the conflicting traffic flow rate, and the directional movement of the subject vehicle affect the size of the accepted gap (Kyte et al., 1991). The mean accepted gap decreases as the queue time or service time increases; the longer waiting time, the greater driver’s frustration and the better driver capability in estimation the size of an accepted gap. Pollatschek et al. (2002) also assumes, when developed the decision model for gap acceptance behavior, that the longer a driver waits, the more the driver is willing to accept risk. A decision model for gap acceptance at intersections has been developed based on a risk-reward loop process. The result shows that the entry onto the main road occurs when the benefit from entry is greater than the associated risk (Pollatschek et al., 2002).

The different conditions (combination of driver, vehicle, traffic, and environment conditions) result in the different gap acceptance behavior at TWSC intersections. Dissanayake et al. (2002) studies on the gap acceptance capabilities of different driver age
groups under different light conditions, daytime and nighttime, at TWSC intersections. Three driver age groups (old, middle, and young) were considered in accordance with two maneuvers (left-turn and through) for each light condition. The analysis results show the significant differences (95% confidence interval) in the gap acceptance capabilities among three driver age groups under both day and night light conditions. The study by Yan et al. (2007) compliments the same conclusion. The older drivers tend to select the larger gaps than the gaps selected by the younger and middle-age drivers. The male drivers tend to accept smaller gaps than female drivers. In addition, the drivers are more likely to accept smaller gap when the speed of major-road vehicles are higher. This implies that the drivers judge the gap based on the distance or position of vehicles on the major road rather than speed.

Yan and Radwan (2007) studies on the effects of restricted sight distances on driver behaviors during unprotected left-turn phase at signalized intersections. The u-turn gap acceptance behavior at signalized intersections was also included. The logistic regression model was used to determine the critical gap of u-turn movement due to limited observation with no-queued condition. The results show that the sight distance problem affects the gap size.

There are only a few studies on gap acceptance characteristics of u-turn junction. One study shows that the combination of gap size and acceleration of priority vehicle gives the best and most consistent definition of the probability of gap acceptance of u-turning vehicles (Ebisawa et al., 2001). The speed of the priority vehicle and wait time of u-turning vehicle are among the insignificant factors. However, the result is based on the data collected at one site on one day. The presence of crosswalk and minor road’s merging flow may affect the u-turn gap acceptance characteristics. Another study, by Kaysi and Abbany (2007), focuses on the aggressive driver behavior. In the gap acceptance process, there might be some aggressive drivers who force their vehicles into the major stream, making the conflicting vehicles to slow down or stop. Three major factors, affecting the probability of such a forcing maneuver, include driver age, car performance, and average speed on the major road. The driver’s total waiting time, while waiting for an acceptable gap, is of little significance in incurring the forcing behavior.

2.2 Unsignalized Intersection Capacity Analysis

The capacity analysis is mostly based on the well-known Highway Capacity Manual (HCM). The current edition is the 5th edition, published since year 2010, known as HCM 2010. The capacity of a system element is defined as “the maximum sustainable hourly flow rate at which persons or vehicles reasonably can be expected to traverse a point or a uniform section of a lane or roadway during a given time period under prevailing roadway, environmental, traffic, and control conditions” (TRB, 2010). From the last phase, if the prevailing conditions change, the capacity of the facility will change.

Elefteriadou (2004) reviews the historical perspective of HCM since the first edition in 1950 until the previous edition in 2000. The finding is that the definition and value of highway capacity have evolved over time. The recent capacity refers to the term “possible capacity”. There is variability in the maximum sustained flows observed due to different of microscopic characteristics, including spacing, time headway, and speed of each vehicle. The three basis components of the traffic system, which are vehicle, driver, and roadway, and their interactions affect a facility’s capacity. For the two-way stop-controlled (TWSC)
intersections and roundabouts, the drivers on minor approach must evaluate the gap size in the conflicting traffic and judge whether they can safely enter. Therefore, the capacity of such system depends on availability of gaps, gap acceptance characteristics, follow-up time of queued vehicles, and utilization of gaps; or so called “gap acceptance process” as mentioned in the previous section.

In the previous HCM 2000, the Chapter 17 includes all 3 types of unsignalized intersections, i.e. two-way stop-controlled (TWSC), all-way stop-controlled (AWSC), and roundabout (TRB, 2000). The current HCM 2010 separates those three facilities into Chapter 19-21. The major street u-turn movement is included in Chapter 19: TWSC intersections. The u-turn maneuver at midblock median opening can be analyzed as a simplified TWSC unsignalized intersection, while the through traffic is the priority movement and the u-turn traffic is the lower rank stop-controlled movement. However, the u-turn movement has a smaller turning radius and slower turning speed than other turning movements (Liu et al., 2008a).

The model for TWSC intersections is based on gap acceptance theory and a relative rigid view of priority of various vehicular and pedestrian movements in the intersection. The HCM methodology for the capacity analysis of TWSC intersection is shown in Figure 2.1.

![TWSC intersection methodology](TRB, 2010, Exhibit 19-4)

Figure 2.1 TWSC intersection methodology (TRB, 2010, Exhibit 19-4)
The explanatory variables in the model for the potential capacity estimation include the conflicting flow rate, the critical headway, and the follow-up headway as shown in Equation 2.3 (TRB, 2010, Equation 19-32). This model is derived based on the gap acceptance theory. It assumes the Poisson random arrival process of conflicting vehicles; in other words, the negative exponential distribution of conflicting headway.

\[ c_{p,x} = v_{c,x} \frac{e^{-v_{c,x}t_{c,x}/3600}}{1-e^{-v_{c,x}t_{c,x}/3600}} \]  

Equation 2.3

where,

- \( c_{p,x} \) = potential capacity of movement \( x \) (vph)
- \( v_{c,x} \) = conflicting flow rate for movement \( x \) (vph)
- \( t_{c,x} \) = critical headway for minor movement \( x \) (s)
- \( t_{f,x} \) = follow-up headway for minor movement \( x \) (s)

The critical headway and the follow-up headway are the main capacity characteristics of the minor movements. Based on the HCM 2010, the critical headway is defined as the minimum time interval in the major-street traffic stream that allows intersection entry for one minor-street vehicle. Thus, the driver’s critical headway is the minimum headway that would be acceptable. A particular driver would reject any headways less than the critical headway and would accept headways greater than or equal to the critical headway. Estimation of critical headway can be made on the basis of observations of the largest rejected and smallest accepted headways for a given intersection. On the other hand, the follow-up headway is defined as the time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major-street headway, under a condition of continuous queuing on the minor street. The follow-up headway is the headway that defines the saturation flow rate for the approach if there were no conflicting vehicles (TRB, 2010).

The potential capacity of a movement assumes the following ideal conditions:

- Traffic from nearby intersections does not back up into the subject intersection.
- A separate lane is provided for the exclusive use of each minor-street movement.
- An upstream signal does not affect the arrival pattern of the major-street traffic.
- No other movements impede the subject movement.

Therefore, the potential capacity must be adjusted to account for the real world conditions and the movement capacity will be calculated consequently. The adjustments include impedance effects of higher priority movements, effects of shared-lane operation, effects of upstream signals & platoon flow, effects of two-stage gap acceptance, and effects of flared approaches. Some adjustment computations are quite cumbersome to implement by hand, and use of software is required (Roess et al., 2004).

Brilon and Wu (2001) proposes an innovation method, i.e. conflict technique, for capacity estimation of the TWSC intersections. The capacity is calculated in accordance with the time consumed by all conflicting traffic components, with the constraint of 3,600 seconds in an hour. The probability that the conflict area is not blocked defines the capacity of the minor movement. The new method is flexible and can deal with share lanes, short lanes, and flared entries. In addition, the method incorporates pedestrians and bicyclists according to their priority rankings. The situation of limited priority and priority reversal can also be reasonably represented by this technique. Brilon and Miltner (2005) illustrates the application of the conflict technique in detail. The conflict matrices based on the real observation of priority were developed. The method was validated with the field capacity.
The conflict technique provides the realistic estimated capacity and agrees with the results from the gap acceptance method. Because of its simplicity and flexibility, Li et al. (2009) applies this conflict technique in the mixed traffic conditions in China where there are a lot of pedestrians and bicyclists. The results indicate that the effect of pedestrian and bicycle movements on the capacities of vehicular movements cannot be ignored. Prasetijo et al. (2011) also proposes this method to use in the mixed traffic condition in Indonesia. The results from the conflict technique correspond with the results from the Indonesia Highway Capacity Manual (IHCM).

2.3 Critical and Follow-up Headways Determination

The gap acceptance model is based on the two main parameters, critical gap and follow-up time (TRB, 2000). Based on the current issue of HCM, those terms are changed to critical headway and follow-up headway (TRB, 2010). The new terms can concisely represent the real world data collection. The data collection for real “gap” needs twice efforts of the collection of “headway”. However, the differences between those values are negligible. Therefore, the terms “gap” and “headway” are interchangeable throughout this dissertation. Some previous works may mention about “gap”, while the present meaning is “headway”. The critical headway cannot be measured directly from the field. The observed accepted headway is larger than the actual driver’s critical headway. On the other hand, the follow-up headway can be obtained from the field by measuring the continuous movement headway of the queued minor street vehicles, entering during the same major traffic gap.

Brilon et al. (1999) summarizes many different methods to estimate the critical gap. The estimation technique for saturated condition is based on Siegloch’s method, which apply a linear regression analysis. The method yields both the critical gap and the follow-up time. For unsaturated condition, there are many techniques to estimate the critical gap. Brilon et al. (1999) also evaluates those methods and recommends the maximum likelihood method and Hewitt’s method for practical application. Tian et al. (1999) describes in detail how to implement the maximum likelihood method to estimate the critical gap. The background of the method includes theory and equations for numerical calculation. A driver’s critical gap is more than his/her largest rejected gap and less than his/her accepted gap. It is assumed that driver behavior is both homogeneous and consistent. The method also assumes a log-normal distribution for the critical gap. Tian et al. (1999) also comprehensively discusses the definition of gap events based on various practical traffic conditions. The gap events should reasonably reflect the gap acceptance characteristics. The maximum likelihood method is the most common and is recommended for use in guidelines. For the u-turn capacity calculation, the Siegloch’s method and the maximum likelihood method yield the close results of the critical gaps of u-turn (Liu et al., 2009).

Wu (2006) proposes a new method for estimating critical gap based on the equilibrium of probabilities of the rejected and accepted gaps. The method is established macroscopically using the cumulative distribution of the rejected and accepted gaps to determine the cumulative distribution of the critical gaps. The probability frequencies of the estimated critical gaps can be calculated from the cumulative distribution. The mean critical gap can be determined by the summation of the product of frequencies and class means. The proposed new method does not require presumptions regarding the distribution function of critical gaps and driver behavior. The advantages of this new method over the maximum likelihood method are no presumption and ease. The calculation can be carried out using common spreadsheet softwares, with no iteration. Using the same datasets, the resulting critical gaps from new method are similar to the results from the maximum likelihood method.
method. According to the resulting probability density function estimated from the new method, the log-normal distribution is suitable for critical gaps. However, Wu (2012) uses regression analysis to calibrate two distribution functions, i.e. log-normal and Weibull, to the empirical distribution of critical gaps from the new method. The result shows that the Weibull distribution is better for representing the distribution of critical gaps, although the differences are not significant.

Vasconcelos et al. (2012) estimates the critical and follow-up headways at roundabout in Portugal. This task applied several estimation methods, including Siegloch, Raff, Wu, maximum likelihood, and logit. Actually, only the Siegloch’s method, which is applied for the saturated condition, yields both the critical and follow-up headways. The other methods yield only the critical headway. The estimated values by different methods are different. Therefore, the selection of method affects the gap acceptance parameters estimation, then capacity. The study shows that the Wu and maximum likelihood methods produce very similar estimates. This is in line with the findings from Wu (2006). After comparing to the reference values from other countries, it can be concluded that those parameters require local calibration because there are differences in driving style and behavior. Bunker (2012) compares the maximum likelihood method with other three methods, i.e. Average Central Gap (ACG), Strength Weighted Control Gap (SWCG), and Mode Central Gap (MCG). The research used simulation model to generate the data of maximum rejected and accepted gaps for each driver. The results affirm that the maximum likelihood method provides the best fit to simulation mean critical gaps across a broad range of conditions. The MCG is the second best; however, it has superior computational simplicity and efficiency. The study tries to promote the use of MCG, but still requires a series of further research.

Guo and Lin (2011) proposes four new calculation models of the critical gap. The main concept is to include the probability distribution function of rejected and accepted gaps. The calculation considers the accepted and rejected coefficients or proportions. Unlike many former methods, the proposed new methods calculate the critical gap and capacity in theoretically manner. Therefore, these new methods are too complicated for practical calculation and should be further simplified.

Some past studies utilize the waiting time to estimate the critical gap. Polus et al. (2003) developed a function of waiting time to estimate the critical gap. After knowing the critical gap, they also proposed the exponential relationship between circulating flow and entry capacity at roundabout. Polus et al. (2005) also evaluated the effect of waiting time on critical gaps at roundabout by a binary logit model.

2.4 U-turn as Alternatives to Direct Left-turn

The traffic studies of the u-turn facilities, especially in the USA, mostly focus on the study, in terms of operation and safety effects, of the right-turns followed by u-turns (RTUT) movement for the replacement of direct left-turns (DLT) movement. These studies are to investigate the advantages and disadvantages of the installation of non-traversable medians on multilane highways. The studies were also to convince the opposing business and property owners who lose the opportunity to allow direct left-turns to/from their developments.

As left-turn (in the right-side driving rule) creates a number of conflicts, there have been efforts to reduce this type of movement at the intersection. The well-known intersection treatment for the left-turn elimination at the signalized intersection is median u-turn
crossovers, which is commonly used in Michigan (FHWA, 2004). In this treatment with median u-turn crossovers located on the major road, the left-turning vehicle from the major road has to cross the signalized intersection, make a u-turn at the provided directional u-turn facility, and eventually make a right-turn to complete the indirect left-turn maneuver. The left-turning vehicle from the minor road has to turn right onto the major road and make a u-turn. Dorothy et al. (1997) studies the operational aspects of various Michigan designs and finds that the indirect left-turning designs and signalized crossovers are superior to other designs in most cases. The direct left-turn design has proportionally higher delay than other designs.

In the last decade, many states in USA installed non-traversable medians on multi-lane highways at the two-way stop-controlled unsignalized intersection. The direct left-turn (DLT) maneuver from driveway has been prohibited and replaced by right-turn followed by u-turn (RTUT) maneuvers. The u-turn can be made either at the next median opening or adjacent signalized intersection. Zhou et al. (2002) studies the operation effects of this treatment and develops delay and travel-time models of both DLT and RTUT movements as a function of traffic flow rates on the major and minor streets. Liu et al. (2007a) compares vehicle delay and travel time for (a) DLT, (b) RTUT at downstream signalized intersections, and (c) RTUT at downstream median openings in advance of signalized intersection. The delay of (c) movement is the least, followed by the delay of (a) movement, while the delay of (b) movement is the longest. The study also shows that more driver prefer making RTUT at median openings rather than at signalized intersections. For the RTUT at the downstream signalized intersection, the increased u-turn may affect the saturation flow rate of the signalized intersection. The study by Carter et al. (2005) shows a 1.8% saturation flow rate loss in the left-turn lane for every 10% increase in u-turn percentage and an additional 1.5% loss for every 10% u-turn if the u-turning movement is opposed by protected right-turn overlap from the cross street.

The location of u-turn median openings has a great impact on the operations of u-turns. Zhou et al. (2003) develops a working model to guide the location of u-turn median openings by minimizing the average delay for u-turn movements. The separation distance between driveway and downstream u-turn location significantly impacts the running time for RTUT (Liu et al., 2007a). Besides the operational aspects, Liu et al. (2008c) studies the safety effects of separation distance and finds that it significantly impacts the safety of the street segments between driveway and u-turn location. A 10% increase in separation distance will result in a 4.5% decrease in target crashes and a 3.3% decrease in total crashes.

The provision of midblock median openings for u-turn between intersection can reduce the number of turning maneuvers at the adjacent intersections. Since the conflict points are less, the accident rates at midblock median openings are lower than at three- or four-leg median openings (NCHRP, 2004). However, the u-turning vehicles may delay full-speed conflicting through traffic. In addition, narrow medians may not provide enough space for larger vehicles to negotiate a u-turn maneuver. To accommodate the u-turn maneuver at median opening, the width of median nose (refer to median at u-turn location) and receiving driveway should be wide enough. For the design vehicle “P” (passenger car with length of 5.7 m), AASHTO recommends the median nose width of 9 m, 5 m, and 2 m for the u-turn maneuver from inner lane to inner lane, to outer lane, and to shoulder lane, respectively, for 4-lane divided highway (AASHTO, 2001).
Recently, the u-turns are widely used in many countries as the alternative to DLT at busy intersections. The performance of such u-turn facilities had been investigated. Combinido and Lim (2010) develops a u-turn traffic flow model, applying cellular automata (CA) simulation, to demonstrate the traffic flow condition at a u-turn junction. The result shows that the effectiveness of u-turn facility is limited to low traffic density and minimal lane changing maneuvers. Pirdavani et al. (2011) proposes a new u-turn design with protected u-turn movement. The new design is compared, in term of travel time, with conventional signalized intersection. The comparison shows that this new u-turn facility generally produces lower travel time, especially when the traffic flow is high.

2.5 Safety of U-turn at Median Opening

For the safety of u-turn at unsignalized median openings, Levinson et al. (2005) summarizes the research findings from the NCHRP Project 17-21 that the accident frequencies is very limited. The unsignalized median openings experience only an average of 0.41 and 0.20 u-turn-plus-left-turn accidents per median opening per year for urban and rural arterial corridors, respectively. Therefore, there is no indication that u-turns at unsignalized median openings are a major safety concern. Pirinccioglu et al. (2006) studies the safety effects of RTUT at signalized intersections and unsignalized median openings in terms of traffic conflict analysis comparing with DLT movements. The DLT movements generates 100% and 10% more conflicts per hour than RTUT movements at signalized intersections and unsignalized median openings, respectively. When the effects of traffic volumes are taken into consideration, RTUT movements have a 5% higher and 62% lower conflict rate than DLT movement at signalized intersections and unsignalized median openings, respectively. Severity comparison indicates that RTUT-related conflicts are less severe than DLT-related conflicts for all cases.

In Thailand, the Bureau of Highway Safety collects and analyzes the data of traffic accident on national highways, under the responsible of the Department of Highways (DOH). The accident location data are also included in the accident report form. The locations are classified into major six categories, including (1) horizontal alignment, (2) vertical alignment, (3) intersection, (4) median opening, (5) highway access, and (6) other special characteristics. Based on the annual report, there are a total of 10,607 and 11,013 accidents occurred on national highways in 2011 and 2012, respectively (DOH, 2011; DOH, 2012). The main cause of accident is the speed limit violation. not much comparing to other highway facilities. Nevertheless, safety improvement measures should be implemented at any vulnerable locations.

Table 2.1 presents the number of accidents on Thailand’s national highway in 2011 and 2012, categorized by accident locations. Most of the accidents occurred at the straight road section. The proportion of accidents at each location is similar in both years. The number of accident at median opening is 701 or 6.4% of all traffic accidents. The frequency of accident at median opening is approximately 2 times per day. In term of severity, around 9.6% of all accidents at median opening involve the fatality. The number of death and injured people caused by the accidents at median opening are 5.7% and 6.4% of all traffic accidents, respectively (DOH, 2012).

Although the u-turn movement at median opening seems to be risky, the studies in USA and the statistics in Thailand do not indicate the major safety concern. Due to the less conflict points, the RTUT movements are encouraged to be provided, in place of the DLT movements at intersections. The number of accidents at median opening is not much
comparing to other highway facilities. Nevertheless, safety improvement measures should be implemented at any vulnerable locations.

<table>
<thead>
<tr>
<th>Accident Location</th>
<th>Year 2011 Amount</th>
<th>Year 2011 Percentage</th>
<th>Year 2012 Amount</th>
<th>Year 2012 Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight</td>
<td>6,751</td>
<td>64%</td>
<td>7,054</td>
<td>64%</td>
</tr>
<tr>
<td>Curve &amp; Slope</td>
<td>1,357</td>
<td>13%</td>
<td>1,493</td>
<td>14%</td>
</tr>
<tr>
<td>Intersections</td>
<td>971</td>
<td>9%</td>
<td>1,033</td>
<td>9%</td>
</tr>
<tr>
<td>Median Opening</td>
<td>690</td>
<td>7%</td>
<td>701</td>
<td>6%</td>
</tr>
<tr>
<td>Highway Access</td>
<td>194</td>
<td>2%</td>
<td>163</td>
<td>1%</td>
</tr>
<tr>
<td>Others</td>
<td>644</td>
<td>6%</td>
<td>569</td>
<td>5%</td>
</tr>
<tr>
<td>Total</td>
<td>10,607</td>
<td>100%</td>
<td>11,013</td>
<td>100%</td>
</tr>
</tbody>
</table>

Source: Bureau of Highway Safety, Department of Highways, Thailand

2.6 U-turn Capacity Estimation

The capacity of u-turn at median openings was first studied in Jordan by Al-Masaeid (1999). Al-Masaeid (1999) conducts a correlation analysis to identify the factors that affect u-turn capacity. The result shows that u-turn capacity has strong correlation with the conflicting traffic flow and the average total delay. Al-Masaeid (1999) also develops a u-turn capacity model by regression analysis. The linear equation between u-turn capacity and conflicting traffic flow provides the better fit to the collected data than the exponential form. For the relationship between the average total delay and conflicting traffic flow, the exponential form is found to have the better fit. From the data collection, the maximum u-turn capacity is around 900 pcu/hr with around 100 pcu/hr of conflicting traffic flow. However, the linear capacity equation indicates that the maximum u-turn capacity would be 799 pcu/hr when there is no conflicting traffic flow. To compare with the HCM 1994 gap acceptance approach, the critical gap and move-up time models are also developed. The critical gap strongly correlates with the average total delay and conflicting traffic speed. The move-up time relates to the average total delay. The empirical u-turn capacity model, developed by regression analysis, is compared with the gap acceptance model. The comparison shows that the gap acceptance model provides reasonable results.

The recently effort to estimate the capacity of u-turn movement is undertaken by Liu et al. (2008a). The critical gap is estimated from the study of headway acceptance characteristic of u-turning vehicle at unsignalized intersections by Liu et al. (2007b). Data collection is based on 4-lane highways with various median widths. Liu et al. (2008a) utilizes the gap acceptance model, as provided in the HCM 2000 for TWSC intersections (Harders model), to estimate the potential capacity of u-turn movement. The result reveals that the width of median at the openings greatly impacts the determination of critical gap, consequently the capacity of u-turn movement; the wide medians result in the higher capacity than the narrow medians. Liu et al. (2008a) also concludes that the gap acceptance model provides reasonable capacity estimation for u-turn movement at median openings comparing to the collected field data. Liu et al. (2008b) further investigates the adjustments on the potential capacity as recommended by HCM 2000, to estimate the u-turn capacity. These include (a) the impedance effects of the minor street right-turn movement, (b) the conflicting traffic volume, and (c) the shared-lane capacity of the major street exclusive lane. For impedance effects, the impedance factor in HCM can reasonably reflect the effects of minor street
right-turn movement on the capacity of u-turn movement. The conflicting traffic volumes for u-turn movements equal the major street through traffic in the conflicting direction plus the major street right-turn traffic volume. However, the HCM shared-lane capacity model overestimates the capacity of the major street exclusive left-turn lane with mixed u-turn traffic.

One year later, Liu et al. (2009) conducts the similar capacity study of u-turns at unsignalized median openings on six-lane streets. The critical and follow-up headways of u-turn on six-lane highway are lower than those on four-lane highway. The conflicting traffic volume for six-lane streets equals 2.2 times the average opposing major street traffic volume in each lane. The results show that the gap acceptance model can also reasonably estimate the capacity of u-turn on six-lane streets, although the mean absolute percent error (MAPE) is quite high. Based on the above research, the capacity estimation methods providing for TWSC intersection can be applied to estimating u-turn capacity at unsignalized median openings. Therefore, it can imply that the behavior of the u-turning vehicle is similar to that of the minor-street vehicle at the TWSC intersection. The latest HCM 2010 adopts the resulting parameters (e.g. critical and follow-up headways) from the above studies in the capacity estimation methodology for u-turn movements.

### 2.7 Waiting Time Estimation

The related past studies about the waiting time estimation concentrate on the two-way stop-controlled (TWSC) intersections. Kyte et al. (1991) studies the delay characteristics of TWSC intersections in both macroscopic and microscopic analysis. The result shows that the waiting time is affected by the traffic flow rate on the conflicting approaches. The linear relationship has been developed by regression analysis. The waiting time increases as the conflicting flow rate increases. Madanat et al. (1994) develops the waiting time function by probability theory. The expected waiting time at the stop line is the product of the average size of rejected gaps and the expected number of rejected gaps. The process of rejecting sequential gaps can be expressed as a geometric distribution. The gap size is assumed to be negative exponential distributed in the study. Al-Omari and Benekohal (1999) develops the linear waiting time models for unsaturated TWSC intersections by empirical approach. The separate waiting time models are also developed for different turning movements; right, left, and through. The statistical test unveils that there is no significant differences between the three models.

Chandra et al. (2009) analyzes the waiting time at uncontrolled intersections in mixed traffic conditions by microscopic approach. The microscopic analysis considers each individual subject vehicle. The conflicting flow rate as seen by the particular subject vehicle is the number of observed conflicting vehicles divided by the observation time. Some advantages of this method are that data is not lost by aggregation and the data points are increased. It also reflects the real conflicting flow rate the subject vehicle faces when waiting for an acceptable gap. The function of waiting time was in the exponential form.

### 2.8 Intersection Traffic Control

When considering the traffic control signal needs studies, the agencies and engineers usually refer to the Manual on Uniform Traffic Control Devices (MUTCD) by FHWA (2009). The manual contains nine warrants in recommending the traffic signal control at intersection. The first three warrants relate to vehicular traffic volume; eight-hour, four-hour, and peak hour. Chilukuri and Laval (2012) evaluates those three warrants based on
the control delay, which is determined by the methodology described in the latest Highway Capacity Manual (HCM) by TRB (2010). The results show that the current eight-hour volume warrant in MUTCD has lower volume than the study’s estimated value. On the contrary, the current warrants for four-hour volume and peak hour volume are higher than the study’s estimated values.

The saturation flow rate is the basic input for the analysis of the signalized intersection. The HCM indicates that the saturation flow rate is reached after the fourth to sixth queuing passenger car crosses the stop line when the signal turns green (TRB, 2010). However, a study of discharge headway model shows that the minimum headway is not achieved until the eighth or higher queue positions (Bonneson, 1992). Moreover, Lin and Thomas (2005) discovers that queue discharge headways tend to undergo compression for a considerable time as more vehicles in the queue are discharged. The discharge rates keep rising even after the 15th queued vehicle entering the junction. He et al. (2009) studies the characteristics of signalized u-turn movements at median openings and intersections in Kunming, China. The results indicate that the saturation headways can be measured from the eighth queuing vehicle.

Instead of providing the physical traffic signal, the human control is sometimes utilized. Traffic police occasionally controls traffic operation; in case of special events, evacuation, and incidents (Wojtowicz and Wallace, 2010). In developing countries, traffic police are often utilized to control the traffic operation during peak periods. Al-Madani (2003) evaluates the efficiency of a police controlled roundabout comparing to a pre-timed traffic signal controlled intersection, in term of dynamic delay. The police is necessary to be provided to control the roundabout during peak periods. This gives better chances for mass crossings of traffic and better control on queue length. The relationship between delay and queue length as the vehicle joins the queue has been developed. The comparison result shows that the roundabout cause less delay when the queue length less than 80 meters and cause greater delay as the 80 meters queue length criterion is exceeded.
CHAPTER 3 FACTORS AFFECTING U-TURN DECISION

3.1 Introduction

U-turn movement at an unsignalized midblock median opening is based on the gap acceptance behavior. When a vehicle arrives at a median opening, each u-turning vehicle’s driver waits for the acceptable gap of the conflicting through traffic at the stop line. When the driver finds the sufficiently large gap, the driver makes a u-turn. This phenomenon is similar to the movement of the minor stream traffic at a two-way stop-controlled (TWSC) intersection. The u-turn movement at the midblock median opening is less complicated in term of traffic components comparing to the movement at the TWSC intersection. However, the u-turn movement is more complicated in terms of the maneuver mechanism. Since the u-turn movement is complex and may lead to safety concerns, the factors affecting the u-turn decision were investigated in this chapter.

This chapter starts with the preliminary study on the behavior of the u-turn vehicle. This is to initially investigate the effect of waiting time on u-turn drivers’ gap acceptance behavior. After that, all the relevant factors to the u-turn decision were taken into consideration. The waiting time of u-turn vehicle was reinvestigated herein to affirm its significance. The effect of queue time was also evaluated, whether or not it affects the u-turn decision.

The objectives of the study in this chapter are listed as below:

- to investigate the effect of waiting time on the accepted gap at a u-turn location,
- to determine the amount of waiting time that would lead to safety concern,
- to estimate the critical gap parameters for the u-turn movement,
- to evaluate the factors affecting the u-turn decision in terms of their statistical significance and level of influence, and
- to develop a u-turn decision model, in order to predict the decision under variety of factors.

The “waiting time” is defined as the time period that a u-turning vehicle arrives at the stop line, waits for the acceptable gap, and starts to make a u-turn. The “rejected gap/headway” is the time gap/headway between two consecutive conflicting vehicles that the u-turning vehicle declines to make a u-turn. The “accepted gap/headway” is the time gap/headway between two consecutive conflicting vehicles that the u-turning vehicle makes a u-turn. During the waiting time, the u-turning vehicle may face one or more rejected gaps, but only one accepted gap. Some vehicles may accept the first gap; therefore, no rejected gap. The terms “gap” and “headway” are interchangeable throughout this research as mentioned in Section 2.3.

The control measures, which are currently in use, at the midblock median openings include police control and traffic signal control. This is to accommodate the u-turning vehicles in the peak periods, when the conflicting traffic volume is high, in terms of capacity and safety. However, the effectiveness of these controls is still in doubt. There is a lack of research work in this field. This chapter also investigates and evaluates the necessity of the control measures for safer u-turn movement. To improve the traffic control strategies at or near the u-turn junctions, knowledge of affecting factors is essential. The developed u-turn decision model can be used to find some control criteria or threshold values. The model can also be used in the microscopic simulation modeling.
This chapter initially presented the effect of the waiting time on the accepted gap of the individual u-turn driver at the midblock median opening. All potential factors affecting the decision of the u-turn driver were also investigated. Those included u-turn drive age and gender, u-turn and conflicting vehicle type, u-turn queuing time and waiting time, conflicting speed and gap size. Of the total eight factors, five related to the u-turning traffic while the other three related to the conflicting traffic. A video camera was set to record the traffic movement at all u-turn locations. In addition, the field observer recorded the data that could not be identified by the video file, e.g. age and gender of u-turn driver. The other required data were extracted in the laboratory.

The data analysis employed the statistical analysis methods. The binary logistic regression technique was employed in the u-turn decision modeling. This technique also yields the significance level of each factor. Then these factors could be selected to be included in the model, based on the significance level. The analysis was conducted in an aggregate manner to represent the overall behavior of u-turning vehicles. It was not site-specific or time-specific; the decision was based on the conditions that the driver faced. In addition, the critical and follow-up headways were determined by the Siegloch’s method, due to its simplicity. The estimated critical headway was compared with the mean accepted gap to ensure the traffic safety of the current traffic operation.

The result showed the trend that the higher waiting time, the shorter accepted gap. The accepted gap for the drivers facing waiting time of more than 30 seconds was significant lower than the accepted gap for the drivers with waiting time up to 20 seconds. The critical gap and follow-up time were 4.3 seconds and 3.4 seconds, respectively. Considering all eight potential factors, the significant factors included gap size, conflicting speed, and waiting time, at the confidence interval of 95%. The combination of u-turning and conflicting vehicle types was also examined and found statistically insignificant. It could be noticeable that the queue time of the u-turning vehicle did not significantly affect the gap acceptance decision, but the wait time did. Eventually, it can be concluded that the control measures was needed for u-turn movement at midblock median openings.

The u-turning vehicles in this study focused only on passenger car category, which included all kinds of vehicle with passenger car equivalent (PCE) of 1. This included private car, taxi, and pick-up truck. The conflicting vehicles also include heavy vehicles (bus and large truck).

3.2 Methodology

3.2.1 Data Collection

The traffic data was collected at three midblock median opening u-turn facilities on Phetkasem Road, western Bangkok area, Thailand. The site locations and traffic condition were shown in Figure 3.1. The selected sites were located on a six-lane divided street with three lanes in each direction. The physical geometry characteristics of the three sites were similar. There was an exclusive u-turn pocket/bay for both directions at all sites. Most u-turning passenger cars encroached to the middle lane in order to complete the u-turn maneuver. Therefore, the vehicles on the middle and median lanes were treated as the conflicting vehicles. These u-turn locations were quite busy with the u-turn vehicles. The analyses were conducted in an aggregate manner so that the results could be applied in general, not site-specific.
Initially, the data were collected at the middle u-turn location (Site 3) for waiting time analysis. The collection periods included AM peak (07:00-09:00 hrs), off-peak (11:00-13:00 hrs), and PM peak (16:00-18:00 hrs). Only the video data was collected in the field. After the waiting time analysis, the more comprehensive data collection was planned. Additional data required the field observation at the same time as video recording. As it was difficult to quickly take note of those data in field, a sound recorder was used by the observer to record the useful information. At this stage, the data were collected for two days at all three sites in order to get sufficient sample data and cover most ranges of traffic conditions. The data from all periods were combined for decision analysis to get the average behavior of all u-turning vehicle samples. The data collection should be conducted under the normal traffic condition; with no accident, no special event, and normal weather condition.

Two kinds of data were collected in the field, i.e. video data and sound data. The video data was reviewed to collect the necessary data and useful information in the laboratory, for example, vehicle type, queuing time, waiting time, gap size, and speed. The speed was calculated based on distance over time, by collecting the travel time between a known fixed distance at each site. The sound data was recorded by another observer, who was near the u-turn location. The observer recorded the u-turn driver age and gender of some random vehicles by speaking to a sound recorder, instead of taking note on paper. The follow-up u-turning vehicles were not considered because they did not perform the ideal gap acceptance behavior. Since the driver age information was based on the observer’s perception, the age data was divided into rough three groups to minimize the human error. Those age groups could reasonably represent the different driving behaviors on gap decision.
The data extraction was based on the manual technique. A spreadsheet program, Microsoft Excel, was utilized to facilitate the data extracting process. When each vehicle passed the imaginary designate lines, the program recorded the time of such incidents by manual clicking the relevant buttons on the screen. The waiting time, queuing time, rejected and accepted gap sizes, number of vehicles accepting the same gap, and other useful information could be obtained by the calculation from the recorded time stamps.

3.2.2 Waiting Time Analysis

At the first stage, the waiting time analysis was conducted to evaluate its effect on the gap acceptance behavior of u-turning vehicle. The initial data from the Site 3 were used in this analysis. The data was screened and analyzed based on the statistical analysis methods. The data were analyzed on both individual data point basis and interval data basis. The descriptive statistic was calculated to explain the characteristics of data.

The statistical method for the comparison of means was based on the two sample t-test. These comparisons were conducted to validate the significance of the differences. The variance homogeneity test applied the Levene’s test. The parametric tests can be conducted when the data follows the normal distribution. For the normality test, this study employed the Shapiro-Wilk test (W-test). The confidence interval of the hypothesis testing for this study was 95%; in other words, level of significance ($\alpha$) was 0.05 or 5%.

3.2.3 Critical Gap Parameters Estimation

The critical gap/headway for u-turn was determined as a benchmark to compare with the accepted gap sizes in the field. If the mean accepted gap size were smaller than the critical gap, the traffic operation would not be safe and require some treatments. From the data collection process described earlier, all headways were recorded with the number of u-turn vehicles utilizing with that gaps. This data could be used to estimate the critical gap parameters, which included critical headway and follow-up headway. The estimation of critical headway parameters employed the Siegloch’s method, which is based on the regression analysis. The method requires the continuous queued condition on the minor street. So, only the traffic data during the saturated u-turn traffic condition was used to estimate the critical headway and follow-up headway.

Akcelik (2007) provides a good summary of the Siegloch’s method. The observation would be made during the time when there is at least one vehicle queuing in the minor street. The number of vehicles, $n$, entering each main stream gap of duration $t$ were recorded, including the zero acceptance ($n = 0$). For each of the gaps accepted by $n$ vehicles, the average of the accepted gaps $t$ was computed. Finally, the linear regression of the average gap values as a function of the number of vehicles was fitted. Typically, the gap size $t$, in seconds, was plotted on the X-axis while the number of vehicles $n$ was plotted on the Y-axis. The zero-gap parameter, $t_0$, was the X-axis intercept of the regression line. The slope of the regression line was the reciprocal of the follow-up time, $t_f$. The critical gap, $t_c$, can be calculated as Equation 3.1 below.

$$t_c = t_0 + 0.5 t_f$$  

Equation 3.1

where,

$t_c$ = critical headway (s)

$t_0$ = zero-gap parameter; intercept on X-axis (s)

$t_f$ = follow-up headway (s)
3.2.4 Gap Decision Analysis

The gap acceptance decision is the dichotomous problem, of which the possible solution will be either “yes” or “no”. The binary logistic regression analysis is widely used to model the occurrence of such an event. The output of the model is the probability of the event occurrence. In addition, the logistic regression also gives the statistical significance level of each variable.

The probability of the u-turn decision based on the explanatory variables \(x_1, x_2, \ldots, x_n\) can be modeled as:

\[
P(\text{accept}) = \frac{e^z}{1+e^z} = \frac{1}{1+e^{-z}}
\]

Equation 3.2

where,

\[P(\text{accept}) = \text{Probability of accepting the subjected gap/headway}\]

\[z = \beta_0 + \beta_1x_1 + \beta_2x_2 + \ldots + \beta_nx_n\]

\(\beta_0, \beta_1, \beta_2, \ldots, \beta_n\) are the parameters estimating from the logistic regression analysis.

The analysis started with all variables to determine the significance of each variable. The forward and backward stepwise analyses, based on likelihood ratio, were also conducted to validate the level of influence of variables. The level of significance was set at 0.05 for the variable entry and 0.10 for the variable removal in the stepwise analysis. The forward stepwise analysis results were used to evaluate the effect of significant variables. The cutting value for the decision of accepting the gap was set at the probability of 0.5. For validating the developed model, around 10% of all data was reserved for model validation. In other words, the model development utilized around 90% of all data.

3.3 Effect of Waiting Time on Accepted Gap

3.3.1 Data Screening

According to the recorded video reviews and field observations, some major traffic stream gaps were distorted, especially, when more than one u-turn vehicles entered the main stream at the same gap. The gaps were not the real gaps because the main traffic vehicles stopped at the median opening and allowed the continuous u-turn vehicles to complete their maneuvers. Therefore, the waiting time study included only the traffic data of the gap, which the only one vehicle could make u-turn at that gap.

In addition, the conflicting traffic should be undersaturated, which could reflect the pure gap acceptance behavior of u-turning vehicles. The traffic conditions in the PM Peak period could not be used for the gap acceptance analysis because the traffic condition in the major stream was in the saturated condition. The u-turn movement was controlled by traffic police. The traffic police acted as a traffic signal to prohibit the movement of the main traffic vehicles and allowed the u-turn vehicles to enter the main traffic stream.

3.3.2 Waiting Time and Accepted Gap Analysis

The waiting time and the accepted gap for each u-turn vehicle according to the above conditions were acquired for further analysis. The traffic data were collected from the recorded video in the AM Peak and off-peak periods. After ruled out all outliers, the total of 169 observations remained for the analysis. Figure 3.2 showed the scatter plots of all
data points. It can be noticed that the range of accepted gap was wide in the small waiting time. On the contrary, the range was narrow in the large waiting time, especially when the waiting time was more than 30 seconds. The wide variety of the small waiting time gap might be caused by the large amount of data points. However, the trend could be obviously seen that the higher waiting time, the smaller accepted gap.

The correlation test between waiting time and accepted gap have been conducted. The resulting Pearson correlation was -0.0661, which indicated a low correlation between waiting time and accepted gap. The negative value of Pearson correlation indicated that the higher waiting time, the lower accepted gap. For the one-tailed statistical significance, the negative correlation was significant at the 80% confidence interval (p-value = 0.1966 < 0.20 level of significance).

To practically investigate the relationships between waiting time and accepted gap, the data was rearranged in the interval of waiting time. The parametric statistical tests were applied, given that the data in each interval was normally distributed. The determination of grouping time interval considered the shortest period, which the number of data in each interval was not too few for further analysis. The number of intervals should be reasonable for real practice. According to the data characteristic of this study, the grouping time interval of 10 seconds was selected. Four intervals of waiting time were defined as follows;

- Group 1: waiting time up to 10 seconds,
- Group 2: waiting time between 11-20 seconds,
- Group 3: waiting time between 21-30 seconds, and
- Group 4: waiting time more than 30 seconds.

After grouping the raw data into the above four intervals, however, the distribution of data in each group failed to follow the normal distribution at the 95% confidence interval. The normality test was based on the Shapiro-Wilk test (W test), which was the principle analysis of the data arrangement in the statistical consideration. The normal distribution of the data was needed. The data could not be statistically analyzed unless the data distribution followed the normal distribution.
In order to reduce the data scattering, the average accepted gap for the same waiting time was calculated and used as the new raw data (red points in Figure 3.2). Therefore, the number of data points was declined. There was only one value of accepted gap for each waiting time value. The average accepted gap data was rearranged again into the previously described four intervals. The data in each group passed the normality test at the 95% confidence interval (p-value > 0.05). Table 3.1 showed the statistic numerical summaries of the accepted gap. The range and mean values were also illustrated as in Figure 3.3.

<table>
<thead>
<tr>
<th>Group</th>
<th>Waiting Time (s)</th>
<th>Mean (s)</th>
<th>SD (s)</th>
<th>Min (s)</th>
<th>Q1 (s)</th>
<th>Q2 (s)</th>
<th>Q3 (s)</th>
<th>Max (s)</th>
<th>n</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1-10</td>
<td>6.25</td>
<td>0.88</td>
<td>5.00</td>
<td>5.54</td>
<td>6.19</td>
<td>6.96</td>
<td>7.50</td>
<td>10</td>
</tr>
<tr>
<td>2</td>
<td>11-20</td>
<td>6.17</td>
<td>0.85</td>
<td>4.50</td>
<td>5.88</td>
<td>6.07</td>
<td>6.73</td>
<td>7.50</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>21-30</td>
<td>5.75</td>
<td>0.82</td>
<td>4.67</td>
<td>5.00</td>
<td>6.00</td>
<td>6.33</td>
<td>7.00</td>
<td>9</td>
</tr>
<tr>
<td>4</td>
<td>more than 30</td>
<td>5.21</td>
<td>0.82</td>
<td>4.00</td>
<td>5.00</td>
<td>5.00</td>
<td>6.00</td>
<td>7.00</td>
<td>18</td>
</tr>
</tbody>
</table>

Note: SD = Standard Deviation, Min/Max = Minimum/Maximum, Q1/Q2/Q3 = 1st/2nd/3rd Quartile, n = No. of Data

The results showed that the mean accepted time was slightly decreased while the waiting time increased, especially when the waiting time is more than 30 seconds. Some drivers accepted the gap as short as four seconds for their u-turn maneuvers. This might lead to the unnecessary traffic accidents. When facing the longer waiting time, some drivers accepted the gaps just equal or shorter than the previously rejected gaps. This situation could be found according to the raw data from data collection processes.

The statistical test was conducted to verify the differences of the mean accepted gaps in all waiting time intervals. The hypothesis testing, based on the mean accepted gap of the last group (waiting time more than 30 seconds), was conducted. The hypothesis of the statistical test was explained below;

Null Hypothesis \( H_0: \mu_i = \mu_4 \)
Alternative Hypothesis \( H_1: \mu_i > \mu_4 \)
where \( i = 1, 2, 3 \) represents the waiting time interval groups
The two-sample t-test was conducted for such comparison purpose. Beforehand, the Levene’s test was also necessary to check the differences in sample variances between the groups. Therefore, the corrected two-sample t-test was conducted properly. The results of the two-sample t-test, at the 95% confidence interval, were summarized as in Table 3.2.

Table 3.2 Comparisons of two means based on the means of Group 4 (> 30 seconds)

<table>
<thead>
<tr>
<th>Waiting Time Group</th>
<th>Group 1 (1-10 seconds)</th>
<th>Group 2 (11-20 seconds)</th>
<th>Group 3 (21-30 seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Difference in Variances</td>
<td>Insignificant</td>
<td>Insignificant</td>
<td>Insignificant</td>
</tr>
<tr>
<td>(Levene’s Test)</td>
<td>(Equal Variance)</td>
<td>(Equal Variance)</td>
<td>(Equal Variance)</td>
</tr>
<tr>
<td>Difference in Means</td>
<td>Significant</td>
<td>Significant</td>
<td>Insignificant*</td>
</tr>
<tr>
<td>(One-tailed t-Test)</td>
<td>(p-value = 0.0021)</td>
<td>(p-value = 0.0036)</td>
<td>(p-value = 0.0597)</td>
</tr>
</tbody>
</table>

Note: The results were based on the level of significance (α) of 5%.
* The difference in means was significant at 10% level of significance.

The statistical test confirmed that the mean accepted gap for the drivers facing the waiting time more than 30 seconds was significantly less than the mean accepted gap for the waiting time up to 20 seconds at the 95% confidence interval. There was no remarkable difference in the mean accepted gaps, at the 95% confidence interval, between the waiting time more than 30 seconds and the waiting time in the range of 21-30 seconds. However, the difference could be statistically significant at the 90% confidence interval, according to the p-value.

3.3.3 Critical Headway Determination

The critical gap parameters were estimated by the Siegloch’s method. Since the collected data included both queued and non-queued conditions of the u-turn traffic, the data must be screened out to reflect only the continuous queued condition. Only the data from the AM Peak periods, which had continuous u-turn queue, were selected for the critical gap parameters determination. The total number of 1,236 gaps for continuous u-turn queue condition were acquired for the analysis, of which 1,096 gaps were rejected (no u-turn vehicle use such gaps). Figure 3.4 illustrated the scatter plots of gap sizes and number of vehicles making u-turn at the same gaps.
Since the number of observations for the number of vehicles more than 5 vehicles making u-turn at the same gap is quite few (6, 8, and 9 vehicles – 1 observation for each; 7 vehicles – 3 observations), the analysis was established on the number of vehicles up to 5 vehicles only. The average gap size for each amount of vehicles was calculated and the linear regression line was also drawn as shown in Figure 3.5.

\[ y = 0.29x - 0.75 \]
\[ R^2 = 0.99 \]

Figure 3.5 Critical gap parameter estimation based on regression technique

From the regression line, the zero-gap parameter, \( t_0 \), equaled to 2.6 seconds (the X-axis intercept). The follow-up headway, \( t_f \), was 3.4 seconds (the reciprocal of the slope). The critical headway, \( t_c \), for the u-turning vehicle at midblock median openings was 4.3 seconds. It should be noted that this regression line was created in order to determine critical gap parameters only. There was no logical meaning when either one of the two variables was less than zero. In addition, the number of vehicles would be a non-negative integer.

3.4 Significant Factors Identification

3.4.1 Considered Factors

The factors that can be simply measured and realized in the field were considered in this study. This would demonstrate and represent the decision of the drivers of u-turning vehicles. The acceleration of the conflicting vehicle was not included because it is very difficult to be recognized by the u-turning vehicle when making a decision.

Eight variables were included in this study:

1. u-turn driver age group, dividing into three groups as young, middle, and old.
2. u-turn driver gender, male or female.
3. u-turning vehicle type, dividing into car (sedan, sport utility vehicle, and van), taxi, and pick-up, according to their different driving behaviors.
4. queue time, which is the time duration that the u-turning vehicle starts to join the queue until it moves to the front of the queue.
5. wait time at the front position of the queue.
6. gap size, referring to time headway of conflicting traffic in this study. This is to facilitate the data collection. As changes in the latest HCM 2010, the term “headway” replaces “gap” for the gap acceptance parameters to estimate the potential capacity of TWSC intersections.

7. speed of the conflicting vehicle.

8. conflicting vehicle type, dividing into car (sedan, sport utility vehicle, and van), taxi, pick-up, and heavy vehicle (bus and large truck).

The variables 1-5 related to the u-turn traffic while the remaining related to the conflicting traffic. In addition, the combination of u-turning vehicle type and conflicting vehicle type were also of interest. It is assumed that all u-turning vehicles have similar capability to recognize gap and speed of conflicting vehicles.

3.4.2 Data Preparation

The recorded video data were reviewed to collect the usable data and information. Some data could not be used in the analysis because the traffic movement did not represent the ideal gap acceptance behavior. For example, the data during the following situations could not be used:

- the conflicting traffic was congested,
- the conflicting vehicle stopped for u-turn, and
- the u-turn movement was controlled by police.

After the data reviewing, a total of 337 u-turning vehicles were extracted from the sound and video data. Each vehicle faced one or many rejected gaps and accepted only one gap. The gap size was known as the most influential factor of gap acceptance process. Definitely, the driver would reject small gaps and accept large enough gap. However, 22 u-turning vehicles accepted the gap smaller than their maximum rejected gap. This analysis was selective in order to explain the reasons why a driver rejected a large gap or accepted a small gap. To weight each driver the same, a pair of largest rejected gap and accepted gap were included in the analysis. Therefore, 674 cases were collected (337 vehicles, 2 gaps each). The conflicting speed data was collected after identifying the vehicles of interest.

The cases with gap less than or equal 2 seconds were screened out since no u-turning vehicle could take such a small headway of conflicting vehicles for u-turn maneuver. On the other end, the cases with gap more than or equal 10 seconds were also screened out because all u-turning vehicles could take such a large gap. As a result, the further analysis included a total of 593 cases of the u-turn decision, of which the random 530 cases were used for model development and the remaining 63 cases were used for model evaluation.

3.4.3 Correlation Analysis

From all 593 data cases, the correlation analysis was conducted. The Pearson correlation coefficient was shown in Table 3.3. It could be noticed that the u-turn decision had a high correlation with gap size. The correlations with conflicting speed and wait time were in the fair level. The result indicated a potential relationship among u-turn decision, gap size, conflicting speed, and wait time. This was in line with the result from the waiting time analysis. The more waiting time would frustrate the drivers to accept the next available gap. In other words, it would increase the probability of accepting gap or making a u-turn decision.
Table 3.3 Pearson correlation coefficient between all variables

<table>
<thead>
<tr>
<th>Variables</th>
<th>Gender</th>
<th>Age</th>
<th>U-turn Vehicle Type</th>
<th>Queue Time</th>
<th>Wait Time</th>
<th>Gap Size</th>
<th>Conflicting Speed</th>
<th>Conflicting Vehicle Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-turn Decision</td>
<td>-0.02</td>
<td>0.00</td>
<td>0.00</td>
<td>-0.03</td>
<td>0.35</td>
<td>0.69</td>
<td>-0.36</td>
<td>-0.03</td>
</tr>
<tr>
<td>Gender</td>
<td>-0.06</td>
<td>-0.18</td>
<td>-0.04</td>
<td>0.03</td>
<td>0.02</td>
<td>0.02</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>Age</td>
<td>0.05</td>
<td>0.05</td>
<td>-0.07</td>
<td>0.05</td>
<td>0.10</td>
<td>0.07</td>
<td>0.03</td>
<td>0.03</td>
</tr>
<tr>
<td>U-turn Vehicle Type</td>
<td>-0.07</td>
<td>-0.02</td>
<td>0.02</td>
<td>0.03</td>
<td>0.12</td>
<td>0.01</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Queue Time</td>
<td>-0.03</td>
<td>-0.01</td>
<td>0.12</td>
<td>0.01</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wait Time</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gap Size</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conflicting Speed</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.4.4 Variable Selection

The logistic regression analysis was conducted by entering all considered variables in order to evaluate their significance levels. The p-value of each variable was shown in Table 3.4. When the p-value is less than the preset significance level, the null hypothesis is rejected. The result is said to be statistically significant. In the analysis, the null hypothesis is no difference on having the variable in model. On the other words, the variable with higher p-value has less effect on the u-turn decision.

Table 3.4 Significance test of all variables

<table>
<thead>
<tr>
<th>Variable</th>
<th>Wald’s $\chi^2$</th>
<th>df</th>
<th>p-value</th>
<th>Sig. Order</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gender</td>
<td>0.84</td>
<td>1</td>
<td>0.36</td>
<td>6</td>
</tr>
<tr>
<td>Age</td>
<td>1.41</td>
<td>2</td>
<td>0.49</td>
<td>7</td>
</tr>
<tr>
<td>U-turn Veh. Type</td>
<td>1.08</td>
<td>2</td>
<td>0.58</td>
<td>8</td>
</tr>
<tr>
<td>Queue Time</td>
<td>0.00</td>
<td>1</td>
<td>0.99</td>
<td>9</td>
</tr>
<tr>
<td>Wait Time</td>
<td>23.15</td>
<td>1</td>
<td>0.00</td>
<td>3</td>
</tr>
<tr>
<td>Gap Size</td>
<td>113.82</td>
<td>1</td>
<td>0.00</td>
<td>1</td>
</tr>
<tr>
<td>Conflicting Speed</td>
<td>36.69</td>
<td>1</td>
<td>0.00</td>
<td>2</td>
</tr>
<tr>
<td>Conflict Veh. Type</td>
<td>4.87</td>
<td>3</td>
<td>0.18</td>
<td>4</td>
</tr>
<tr>
<td>U-turn Veh. Type × Conflict Veh. Type</td>
<td>8.14</td>
<td>6</td>
<td>0.23</td>
<td>5</td>
</tr>
</tbody>
</table>

Goodness-of-fit Test

$\chi^2$ | df | p-value
---|----|---------|
Hosmer and Lemeshow | 13.51 | 8 | 0.10 |

Note: df=degree of freedom, Veh.=Vehicle, Sig.=Significance

The influence of variables to the decision in the descending order was as following: gap size > conflicting speed > wait time > conflicting vehicle type > combination of vehicle type > gender > age > u-turning vehicle type > queue time. Only three variables were statistically significant, at the significance level of 0.05, including gap size, conflicting speed, and wait time. The combination of u-turning and conflicting vehicle type was also insignificant.
3.4.5 Effect of Significant Variables

In the forward stepwise logistic regression analysis process, each significant variable was entered into the model according to its p-value. Table 3.5 presents the results of the stepwise analysis, including the variables at each step and parameters estimation. The effect of each variable, entering at each step, was described in the following sub-sections. The correlation among all variables was also analyzed to check whether those variables should include in the decision model. Normally, the high correlation coefficient between any pair of variables indicates that one of the two variables should be taken out from the model for reliable prediction.

<table>
<thead>
<tr>
<th>Step 1</th>
<th>Variable</th>
<th>B</th>
<th>S.E.</th>
<th>Wald’s $\chi^2$</th>
<th>df</th>
<th>p-value</th>
<th>Exp(B)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Constant</td>
<td>-6.129</td>
<td>0.502</td>
<td>149.15</td>
<td>1</td>
<td>0.00</td>
<td>0.002</td>
</tr>
<tr>
<td></td>
<td>Gap Size</td>
<td>1.221</td>
<td>0.101</td>
<td>145.09</td>
<td>1</td>
<td>0.00</td>
<td>3.391</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Step 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Constant</td>
<td>-3.107</td>
<td>0.627</td>
<td>24.56</td>
<td>1</td>
<td>0.00</td>
<td>0.045</td>
</tr>
<tr>
<td></td>
<td>Gap Size</td>
<td>1.270</td>
<td>0.111</td>
<td>131.51</td>
<td>1</td>
<td>0.00</td>
<td>3.560</td>
</tr>
<tr>
<td></td>
<td>Conflicting Speed</td>
<td>-0.064</td>
<td>0.010</td>
<td>41.14</td>
<td>1</td>
<td>0.00</td>
<td>0.938</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Step 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Constant</td>
<td>-3.802</td>
<td>0.665</td>
<td>32.66</td>
<td>1</td>
<td>0.00</td>
<td>0.022</td>
</tr>
<tr>
<td></td>
<td>Gap Size</td>
<td>1.235</td>
<td>0.112</td>
<td>122.59</td>
<td>1</td>
<td>0.00</td>
<td>3.439</td>
</tr>
<tr>
<td></td>
<td>Conflicting Speed</td>
<td>-0.062</td>
<td>0.010</td>
<td>35.48</td>
<td>1</td>
<td>0.00</td>
<td>0.940</td>
</tr>
<tr>
<td></td>
<td>Wait Time</td>
<td>0.043</td>
<td>0.010</td>
<td>19.83</td>
<td>1</td>
<td>0.00</td>
<td>1.044</td>
</tr>
</tbody>
</table>

Note: B=Estimated Parameter, S.E.=Standard Error, df=degree of freedom

a) Gap size

The relationship between the gap size and probability of accepting the gap was shown in Figure 3.6. At the $P(\text{accept})$ of 0.5, the gap size was 5.0 seconds. This implied that when a u-turning vehicle faced a gap size of greater than or equal to 5.0 seconds, the vehicle would accept the gap and make u-turn.

![Figure 3.6 Effect of gap size on u-turn decision](image-url)
b) Speed of conflicting vehicle

The speed of the conflicting vehicle affected the gap acceptance behavior. As shown in Figure 3.7, the higher speed, the lower probability of accepting the gap for the same gap size. At the P(accept) of 0.5, the gap size was 4.0, 5.0, and 6.0 seconds for the conflicting vehicle’s speed of 30, 50, and 70 km/hr, respectively.

![Figure 3.7](image)

Figure 3.7 Effect of conflicting vehicle speed on u-turn decision

c) Wait time

The wait time of the u-turning vehicle also affected the gap acceptance behavior. As shown in Figure 3.8, the longer wait time, the higher probability of accepting the gap for the same gap size. At the P(accept) of 0.5, the gap size was 5.6, 5.0, 4.5 and 4.0 seconds for the wait time of 0, 15, 30, and 45 seconds, respectively. These results were based on the assumed constant conflicting speed of 50 km/hr.

![Figure 3.8](image)

Figure 3.8 Effect of wait time on u-turn decision
d) Correlation among variables

The correlation matrix from the forward stepwise analysis was shown in Table 3.6. The gap size and conflicting speed seemed to have some negative relationship, but not so high. Therefore, it was better to leave both variables in the model to yield the more accurate prediction results. Nevertheless, the wait time seemed to have no correlation with gap size and conflicting speed.

<table>
<thead>
<tr>
<th>Step 1</th>
<th>Gap Size</th>
<th>-0.970</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 2</td>
<td>Gap Size</td>
<td>-0.616</td>
</tr>
<tr>
<td></td>
<td>Conflicting Speed</td>
<td>-0.550</td>
</tr>
<tr>
<td></td>
<td>Speed</td>
<td>-0.288</td>
</tr>
<tr>
<td>Step 3</td>
<td>Gap Size</td>
<td>-0.598</td>
</tr>
<tr>
<td></td>
<td>Conflicting Speed</td>
<td>-0.520</td>
</tr>
<tr>
<td></td>
<td>Wait Time</td>
<td>-0.268</td>
</tr>
<tr>
<td></td>
<td>Speed</td>
<td>0.069</td>
</tr>
<tr>
<td></td>
<td>Speed</td>
<td>-0.040</td>
</tr>
</tbody>
</table>

3.5 U-turn Decision Model

To predict the u-turn decision at the confidence interval of 95%, three variables were included in the model formulation. The developed u-turn decision model was shown as follows:

\[
P(\text{accept}) = \frac{1}{1 + e^{3.802 - 1.235t_g + 0.062s_c - 0.043t_w}}
\]

Equation 3.3

where,

- \( P(\text{accept}) \) = Probability of accepting the subjected gap/headway (in a range of 0 to 1)
- \( t_g \) = gap size (s)
- \( s_c \) = conflicting speed (km/hr)
- \( t_w \) = wait time (s)

The model performance was shown in Table 3.7. The model yielded the Nagelkerke \( R^2 \) of 0.69. The Hosmer and Lemeshow test also indicated the goodness-of-fit of the observed and predicted events (p-value > 0.05). The classification table showing the percentage correctness of the model prediction was shown in Table 3.8 for both the cases that were used and not used in the model development process. The validation result showed that the developed model could predict the u-turn decision well with the percentage correctness of more than 85%.

<table>
<thead>
<tr>
<th>Goodness-of-fit Test</th>
<th>( \chi^2 )</th>
<th>df</th>
<th>p-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hosmer and Lemeshow</td>
<td>7.78</td>
<td>8</td>
<td>0.46</td>
</tr>
</tbody>
</table>

Model \( R^2 \)

| Cox & Snell \( R^2 \)      | 0.52 |
| Nagelkerke \( R^2 \)       | 0.69 |
### Table 3.8 Classification table showing the prediction result

<table>
<thead>
<tr>
<th></th>
<th>Predicted Cases for model development</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rejected</td>
<td>Accepted</td>
<td>% Correct</td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rejected</td>
<td>239</td>
<td>38</td>
<td>86.28</td>
<td></td>
</tr>
<tr>
<td>Accepted</td>
<td>39</td>
<td>214</td>
<td>84.58</td>
<td></td>
</tr>
<tr>
<td>Overall Percentage</td>
<td></td>
<td></td>
<td>85.47</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th>Predicted Cases for model validation</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rejected</td>
<td>Accepted</td>
<td>% Correct</td>
<td></td>
</tr>
<tr>
<td>Observed</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rejected</td>
<td>25</td>
<td>4</td>
<td>86.21</td>
<td></td>
</tr>
<tr>
<td>Accepted</td>
<td>4</td>
<td>30</td>
<td>88.24</td>
<td></td>
</tr>
<tr>
<td>Overall Percentage</td>
<td></td>
<td></td>
<td>87.30</td>
<td></td>
</tr>
</tbody>
</table>

#### 3.6 Discussions

This chapter initially investigated the effect of waiting time, one of the main factors affecting the gap acceptance behavior, of u-turning vehicles at midblock median openings. The results showed that the waiting time affected the accepted gap. However, based on the collected data, the negative correlation was very low and insignificant. After grouping the collected data into intervals, the trend of the relationship was clearer. The effect of waiting time was further investigated together with other potential factors in the logistic regression analysis. The results confirmed the significance of waiting time on u-turn decision, even though the gap size and conflicting speed had more influence.

The initial result of this study agrees with the previous research on the gap acceptance at the TWSC intersections. Kyte et al. (1991) studies the capacity and delay characteristics and found that the length of delayed time affects the size of accepted gap. Pollatschek et al. (2002) also mentions that the duration of the wait affects the risk tolerance of the driver. They conclude that the longer waiting time, the smaller size of accepted gap. In addition to the previous research, this study focused on the u-turn facility, which is the different transport facility but has similar gap acceptance process. This study also yielded the threshold value of waiting time, i.e. more than 30 seconds, that might frustrate the driver to accept the very short gap.

The findings from this chapter could illustrate the distinctive characteristics of u-turn gap acceptance behavior. For TWSC intersection, the mean accepted gap tends to decrease as the queue time or wait time increases (Kyte et al., 1991). However, the analysis in this chapter found that the queue time was not statistically significant for the u-turn gap acceptance decision. The u-turn vehicles, when staying in queue, could see the conflicting traffic stream and realize the traffic situation. Therefore, the u-turn drivers would not take the delay in queue to decide whether or not accepting the gap. In other words, the waiting of u-turn drivers might be memoryless, when changing position to the front of queue. Nevertheless, it also depended on the nature of the driver population in the area.

Unlike the past research (Ebisawa et al., 2001), the results of this study showed that the conflicting speed and wait time also influenced the u-turn decision. The past study indicated the significance of wait time in the morning and noon periods, but not in the afternoon period. Thus, the wait time was not selected due to its inconsistency. Nevertheless, the wait time might be significant in the afternoon period on the other days. In addition, the different site characteristics might differ the analysis results. The presence
of crosswalk might cause the insignificance of conflicting speed in u-turn decision. Instead, the u-turn vehicle might consider whether the conflicting vehicle was accelerating or decelerating. However, it is quite difficult for u-turn drivers to recognize the acceleration level of the conflicting vehicles.

The critical gap parameters would differ from place to place. In USA, the critical headway for u-turn movement on six-lane streets is 5.6 seconds while the follow-up headway is 2.3 seconds (Liu et al., 2009; TRB, 2010). For a case study of a u-turn site in Bangkok, the critical headway was 4.3 seconds while the follow-up headway was 3.4 seconds. The speed of the major stream traffic might be one of the primary causes of the difference. It seems that the speed of major stream traffic in USA is higher than the speed in Bangkok. The u-turning vehicle facing the higher speed traffic stream required the larger gap and shorter follow-up time. For the current case study, comparing to the other movements at TWSC intersection, the relatively low critical headway of u-turn movement might be caused by the better sight distance. On the other hand, the relatively high follow-up headway of u-turn movement might be caused by the slower movement mechanism.

It was concerned whether the long waiting time would cause traffic accident at the u-turn facility. The mean accepted gap of the long waiting time group (more than 30 seconds) was statistically compared with the estimated critical headway. The results showed that the mean accepted gap of the long waiting time group (= 5.2 s) was significantly larger than the critical headway (= 4.3 s), at the 95% confidence interval. This could ensure the traffic safety of the u-turn movement at the median opening. However, based on the Figure 3.8, when the wait time was as high as 45 seconds, the driver would accept the gap as low as 4.0 seconds, which was smaller than the critical headway. This might lead to the traffic accident, caused by the forcing u-turn movement. Traffic operation treatments are required to prevent such a long time waiting of u-turn vehicles.

Some factors affecting the u-turn gap acceptance and aggressive behaviors were different. Conflicting speed was the same factor affecting both behaviors. The aggressive behavior is mainly the outcome of attitudinal characters of driver, not dependent on driver’s wait time (Kaysi and Abbany, 2007). Most aggressive drivers (90%) conducted the forcing maneuver after waiting not more than 10 seconds. However, based on the result of the present study, the longer waiting time would contribute to the unsafe movement, implying that the driver himself is not an aggressive driver but could induce an aggressive behavior. On the contrary, the real aggressive driver would conduct the forcing maneuver at the beginning of his waiting, without considering the wait time.

The conclusions from the study in this chapter could be listed as follows:

- the larger waiting time, the smaller accepted gap for the u-turning vehicles at midblock median openings,
- the waiting time of more than 30 seconds would frustrate the drivers to accept the significant smaller gap,
- gap size, conflicting speed, and wait time significantly affected the u-turn decision at 95% confidence interval, respectively,
- u-turn driver age and gender, vehicle type of both u-turning and conflicting traffic, and queue time did not significantly influence the decision at 95% confidence interval,
- u-turn decision model based on gap size, speed and wait time could predict well with the percentage correctness of more than 85%.
The factors affecting the u-turn decision at midblock median opening were addressed in this chapter. Each significant factor could be investigated in the further studies. The wait time of u-turn vehicle may have relationships with conflicting traffic characteristics. The further studies will provide a clear understanding on the traffic operation at the u-turn junction, leading to the better traffic management. Since the u-turn movement was complex and risky, the appropriate control and management of the u-turn traffic was highly recommended for improving the operation and safety of the u-turn movement. The safety concerns would be arisen when the u-turning vehicles faced the long waiting time and behaved to accept extremely unsafe gap. Knowing these behaviors can help the traffic engineer to mitigate the problem at the real causes.
CHAPTER 4  CAPACITY ESTIMATION OF U-TURN JUNCTION

4.1  Introduction

There are a lot of midblock u-turn facilities on urban arterials in the developing countries’ cities. These midblock u-turn junctions interrupt the through traffic movement. After arriving the midblock median opening, the u-turn vehicles wait for the large enough gap and make u-turn maneuver. There are interactions between through and u-turn traffic streams. When the through traffic volume increases, it lessen the chances for the u-turn traffic to move. The reduction of traffic volume in one stream could increase the movement capacity in the other stream. The u-turn vehicles affect the through traffic movement in the opposite direction when they move. Those u-turn vehicles also affect the through traffic movement in the same direction when they stop and create queue longer than the pocket lane length.

The u-turning vehicles act in the same manner as the vehicles in the minor approaches in the TWSC intersection. The recent Highway Capacity Manual (HCM 2010) includes the major-street u-turn movements in the methodology for two-way stop-controlled (TWSC) intersections (TRB, 2010). The gap-acceptance theory defines the method for capacity estimation. Three basic elements are gap availability, gap usefulness, and relative priority of subjected movements. The potential capacity equation assumes random arrival process of vehicles on the major street. The model also assumes consistent and homogeneous driving behavior. Liu et al. (2007b; 2008a; 2008b; 2009) have conducted series of research relating to capacity of u-turn at median opening. They estimates the parameters (critical headway and follow-up headway) of u-turn movements from the field data. They validates the capacity estimation from the model with the field capacity. The model provides reasonable capacity estimate for u-turn movement at median openings. The HCM 2010 utilizes the values of these parameters of u-turn movement for the capacity analysis in the US. Nevertheless, the critical headway and the follow-up headway need local calibration due to differences in driving style (Vasconcelos et al., 2012). Those parameters also vary according to physical geometry characteristics of the junction (Weinert, 2000).

The previous chapter estimated the critical gap parameters from the data collected at one u-turn site (Site 3) by Siegloch’s method. The critical headway was 4.3 seconds while the follow-up headway was 3.4 seconds. Based on the method proposed in HCM (Equation 2.3), the u-turn capacity as a function of conflicting traffic flow rate is shown in Equation 4.1. The relationship can be clearly illustrated in Figure 4.1.

\[
    c = v_c \frac{e^{-0.001195v_c}}{1 - e^{-0.000943v_c}}
\]

Equation 4.1

where,

- \( c \) = capacity of u-turn movement (vph)
- \( v_c \) = major stream conflicting flow rate (vph)

According to the graph, the u-turn movement can reach the maximum capacity of about 1,050 vph when there is no conflicting traffic. The capacity of u-turn is decreasing while the conflicting flow rate is increasing. The capacity is reduced to 500 vph, 220 vph, and 90 vph when the conflicting flow rate is 1,000 vph, 2,000 vph, and 3,000 vph, respectively. The capacity model is totally relied on the values of critical and follow-up headways. Those parameters are varied according to the selected site and the method of estimation, which was discussed in Section 2.3.
Figure 4.1 U-turn capacity model based on gap acceptance theory

The traffic flow rate at downstream of the u-turn is the summation of conflicting through vehicles and u-turn vehicles. Since the u-turn capacity depends on the conflicting traffic flow rate, the capacity at downstream is also same. Figure 4.2 shows the variation of traffic flow at downstream of u-turn, provided the same parameters as above (Equation 4.1).

Figure 4.2 Capacity at downstream of u-turn location

The model estimate can differ from field capacity. Kyte et al. (2003) lists the three main causes of difference, including (1) headway distribution of major stream, (2) usage of gaps of minor stream, and (3) driver behavior. The arrival of conflicting vehicles on urban arterial sometimes does not follow the random process. In other words, the headways are not negatively exponential distributed. This affects the availability of gaps for the u-turning vehicles. This chapter considered the headway distribution. The conflicting traffic headway distribution had been checked before conducting capacity estimation. Unlike crossroads, the u-turning drivers can easily recognize the gap of conflicting traffic because of the better
line-of-sight. The critical headway of u-turn movement is smaller than those of other movements on minor streets. The response of the u-turning vehicles to the gap may not be consistent. Sometimes the driver does not accept the first large enough gap. Sometimes the driver accepts the relatively small gap, which is not safe. The individual driver behavior affects the decision on the facing gap. For the gap acceptance at unsignalized intersections, Pollatschek et al. (2002) concluded that the longer waiting time, the smaller accepted gap.

The existing capacity model in HCM 2010 may not be applicable for the u-turn movements at midblock median openings on urban arterials. In addition, the conflicting through traffic stream is not always priority. It sometimes has to stop or decelerate to allow the forcing u-turn traffic movement. The traffic characteristics do not follow the concept of priority-controlled TWSC intersections. The capacity of the conflicting through traffic is also of interest. This chapter proposes a method to find capacity of u-turn as well as conflicting traffic movements at midblock u-turn junctions on urban arterials. The proposed method comprises two steps of calculation. Firstly, the potential u-turn capacity was estimated based on the gap acceptance theory, according to the known headway distributions. Secondly, the estimated u-turn capacity was adjusted, based on balancing of volume-to-capacity ratio (v/c) of both traffic streams. The results include the capacity of both u-turn traffic and conflicting traffic.

The objectives of the study in this chapter can be listed as follows;

- evaluate the u-turn capacity estimation based on gap acceptance theory;
- study the effect of conflicting headway distribution on u-turn capacity estimation by gap acceptance theory;
- propose the new methodology to estimate u-turn and conflicting capacity based on v/c balancing; and
- investigate the characteristics of capacity estimates by the new method.

The result showed that the gap acceptance capacity might overestimate or underestimate the field capacity, depending on the types of conflicting headway distribution. The proposed method could adjust the estimated capacity to be closer to the measured field capacity. In addition, the proposed method considered the interactions between the two traffic streams, which were in line with the real world traffic operation in the urban environment.

4.2 Methodology

4.2.1 U-turn Potential Capacity

The potential capacity equations were derived by the gap acceptance concept. When a u-turn vehicle faces a gap of conflicting traffic, the driver would recognize gap size and compare with his/her critical gap ($t_c$). The driver does not make a u-turn if the gap size is less than the critical gap. The driver makes a u-turn when the gap size equals to the critical gap or more. For the queued u-turn movement, the followed u-turn vehicles require the lesser critical gap, which is called follow-up headway ($t_f$). So, if the gap size is between $t_c$ and $t_c + t_f$, only one vehicle can make u-turn. If the gap size is between $t_c + t_f$ and $t_c + 2t_f$, two vehicles can make u-turn. If the gap size is between $t_c + 2t_f$ and $t_c + 3t_f$, three vehicles can make u-turn and so on. The potential capacity is the summation of the total u-turn vehicles, according to the above explanation, as shown in Equation 4.2. To estimate the potential capacity, the gap size distribution, the critical gap, and the follow-up headway must be
known. Since the gap data requires too much effort of data collection, the headway is used instead.

\[ c_{pu} = \sum_{n=1}^{\infty} \left( v_c \times \left[ P(h > t_c + (n-1)t_f) - P(h > t_c + nt_f) \right] \times n \right) \]

Equation 4.2

where,

\[ c_{pu} = \text{u-turn potential capacity} \]
\[ v_c = \text{conflicting traffic flow rate} \]
\[ P(h > t) = \text{probability that the headway is larger than t} \]
\[ t_c = \text{critical headway} \]
\[ t_f = \text{follow-up headway} \]
\[ n = \text{number of u-turn vehicles in the same headway; } n = 1, 2, 3, \ldots \]

When one knows the headway distribution of the conflicting traffic stream, one can determine the probability that the headway is larger than a specific value. The second term in Equation 4.2 represents the probability that the headway is between \( t_c + (n-1)t_f \) and \( t_c + nt_f \), which allows \( n \) vehicles to make u-turn. When the vehicles arrive in random, the headway distribution is the negative exponential distribution. When the traffic volume is high, the movement of one vehicle affects or is affected by other vehicles. The vehicle arrival is not random anymore. The Erlang distribution can explain the traffic condition in the intermediate state, which lies between the random and constant headway states (May, 1990). The Erlang distribution can also represent the headway of traffic on multi-lane highway, where the headway on each lane is negatively exponential distributed. The headway probability density function of Erlang distribution is shown in Equation 4.3 (Salter and Hounsell, 1996).

\[ f(t) = \left( \frac{qK}{(K-1)!} \right) t^{K-1} e^{-qt} \]

Equation 4.3

where,

\[ q = \text{traffic flow rate} \]
\[ K = \text{shape factor; } K = 1, 2, 3 \]

When the shape factor (K) equals to 1, it is the negative exponential distribution, which represents the random arrival process. Therefore, the Erlang distribution can cover a wide range of traffic conditions, by varying its shape factor. For this research, we considered the shape factor of 1, 2, and 3 because the traffic flow rate on an urban arterial is not extremely high. The distribution of the headway data was determined by the Chi-square (\( \chi^2 \)) goodness-of-fit method.

The probability that the headway is larger than \( t \), for each value of shape factor, is shown in Equation 4.4. After substituting Equation 4.4 into Equation 4.2, the potential capacity equations are derived and shown in Equation 4.5. The details of derivation for Equation 4.4 and Equation 4.5 are shown in Appendix.

For \( K=1 \), \( P(h > t) = e^{-v_c t} \)

For \( K=2 \), \( P(h > t) = e^{-2v_c t} (1 + 2v_c t) \)

For \( K=3 \), \( P(h > t) = e^{-3v_c t} \left[ 1 + 3v_c t + \frac{(3v_c t)^2}{2} \right] \)

Equation 4.4
For $K=1$, $c_{pu} = \frac{v_c e^{-v_c t_c}}{1-e^{-v_c t_f}}$  \[\text{Equation 4.5}\]

For $K=2$, $c_{pu} = \frac{v_c e^{-2v_c t_c}}{1-e^{-2v_c t_f}} \left[1+2v_f t_c +2v_f t_f - \frac{e^{-2v_f t_f}}{1-e^{-2v_f t_f}} \right]$  

For $K=3$, $c_{pu} = \frac{v_c e^{-3v_c t_c}}{1-e^{-3v_c t_f}} \left[1+3v_f t_c +2v_f t_f - \frac{e^{-3v_f t_f}}{1-e^{-3v_f t_f}} \right] + \frac{(3v_f t_f)^2}{2} \left[ e^{-3v_f t_f} (1+e^{-3v_f t_f}) \right] - \frac{(1-e^{-3v_f t_f})^2}{2}$

where,
- $c_{pu}$ = u-turn potential capacity (veh/s)
- $v_c$ = conflicting traffic flow rate (veh/s)
- $t_c$ = critical headway (s)
- $t_f$ = follow-up headway (s)

The critical headway was determined by the maximum likelihood method (Tian et al., 1999). It assumes that a driver’s critical headway is between his largest rejected headway and his accepted headway. The method also assumes a log-normal distribution for the critical headways. The log-likelihood of a sample of $n$ drivers having an accepted headway and a largest rejected headway of $(a_i, r_i)$ is given in Equation 4.6. After maximizing the log-likelihood function, the mean critical headway and its variance can be calculated from the mean and variance of the distribution of the logarithms of the individual driver’s critical headways, as shown in Equation 4.7. On the other hand, the follow-up headway was determined directly from the field data, according to the definition provided in HCM 2010.

$$L = \sum_{i=1}^{n} \ln[F(y_i) - F(x_i)]$$  \[\text{Equation 4.6}\]

where,
- $L$ = logarithm of the likelihood function
- $y_i$ = logarithm of the accepted headway of the $i$th driver = $\ln(a_i)$
- $x_i$ = logarithm of the largest rejected headway of the $i$th driver = $\ln(r_i)$
- $F(\cdot)$ = cumulative distribution function of the normal distribution
- $t_c = e^{\mu+0.5\sigma^2}$ \[\text{Equation 4.7}\]
- $s^2 = t_c^2 (e^{\sigma^2} - 1)$

where,
- $t_c$ = mean critical headway
- $s^2$ = variance of the critical headway
- $\mu$ = mean of the distribution of the logarithms of the individual driver’s critical headways
- $\sigma^2$ = variance of the distribution of the logarithms of the individual driver’s critical headways

4.2.2 Capacity Adjustment

The traffic condition on an urban arterial tends to reach an equilibrium situation. Considering interactions of both traffic streams, the traffic intensity of u-turn traffic seems to equal to the traffic intensity of conflicting traffic. The volume-to-capacity ratio ($v/c$) was used as the measurement of traffic intensity in this research. By balancing the $v/c$, one could estimate the capacity for both u-turn and conflicting traffic directly.
The capacity estimation could be described in steps as follows:

Step 1: collect the traffic volume data of conflicting traffic ($v_c$) and u-turn traffic ($v_u$).

Step 2: calculate the u-turn potential capacity ($c_{pu}$) by Equation 4.5, as described in the previous section.

Step 3: calculate the conflicting potential capacity ($c_{pc}$) by inversing the headway; $c_{pc} = 3600/h_i$, where $h_i$ is the mean rejected headway of the conflicting traffic.

Step 4: adjust the potential capacities by increasing the capacity of one stream (u-turn or conflict) and decreasing the capacity of the other stream (conflict or u-turn) so that the $v/c$ of both traffic streams are equal; $v_c/c_c = v_u/c_u$, where $c_c$ and $c_u$ are the resulting capacities of conflicting and u-turn traffic, respectively.

The adjustment followed the fact that the seconds consumed by one traffic stream were replaced by the other traffic stream. For instance, when the $v/c$ of u-turn traffic was higher than the $v/c$ of conflicting traffic, we had to increase the capacity of u-turn traffic and decrease the capacity of conflicting traffic. The amount of conflicting capacity reduction was converted into time consumed by such reduction, and that time would be used by u-turn movement to increase u-turn capacity. The follow-up headway ($t_f$) of u-turn movement, representing the continuous u-turn, was used for converting time to amount of vehicle movement. For the conflicting traffic, the imaginary headway ($h_i$), calculated from u-turn potential capacity, was used instead of measured headway ($h_i$). The relationship was derived based on the fact that the total seconds consumed in an hour must be equal 3600, i.e. $v_c \times h_i + c_{pu} \times t_f = 3600$. So, the imaginary headway could be calculated as $h_i = (3600 - c_{pu} \times t_f)/v_c$.

4.2.3 Validation

The estimated u-turn capacity was validated by the field capacity. The field capacity estimation followed the method as described in NCHRP (1996). In this research, the data analysis was based on 5-minute intervals. Since the u-turn traffic during the intervals was undersaturated, the field capacity was estimated by Equation 4.8 (Kyte et al., 1991). The service time ($t_s$) and move-up time ($t_{mv}$) could be measured from the field observation.

$$c_f = \frac{3600}{t_s + t_{mv}} \quad \text{Equation 4.8}$$

where,

- $c_f$ = u-turn field capacity (vph; veh/hr)
- $t_s$ = average service delay, i.e. waiting time at the stop line (s)
- $t_{mv}$ = average move-up time from the second position to the stop line (s)

The capacity estimation was evaluated by the value of mean absolute percentage error (MAPE) as shown in Equation 4.9. The low MAPE indicated that the estimated capacity could well predict the field capacity.

$$\text{MAPE} = \frac{1}{n} \sum_{i=1}^{u} \left| \frac{c_u - c_f}{c_f} \right| \quad \text{Equation 4.9}$$
where,

\[ n \] = number of intervals

\[ c_u^i \] = estimated u-turn capacity at time interval \( i \) (vph)

\[ c_f^i \] = field u-turn capacity at time interval \( i \) (vph)

### 4.3 Input Data Preparation

#### 4.3.1 Traffic Count Data

To illustrate the application of the proposed method, the traffic data were collected at a u-turn midblock median opening on an urban arterial in western Bangkok, Thailand (Site 2 in Figure 1.5). The road, at the u-turn junction, has three through lanes in each direction with an exclusive u-turn lane on both directions. Most u-turning vehicles encroach to the middle lane in order to complete the u-turn maneuver. Since the traffic condition at the site is busy during the peak periods, the data collection is difficult during those periods. In addition, the policeman controls the u-turn movement when the traffic is congested. The data collection was conducted during off-peak period (11:00-13:00 hrs) on two days. A digital camera was set up on the pedestrian bridge to record the traffic movements. The recorded video files were reviewed in the laboratory to extract the required data for further analysis.

Since the data was analyzed in 5-minute intervals, a total of 48 intervals were considered. The data acquisition was based on the timestamp of all movement events. The required data were determined by the calculation from the recorded time. The traffic volume came from the vehicle count in each interval. Because the traffic count is in the 5-minute intervals, the flow rate in each interval can be calculated by multiplying with 12 to get the hourly volume. The headway was the time difference of the passing of two consecutive vehicles on all conflicting lanes. The waiting time was the difference between the departure time and the arrival time. Table 4.1 summarizes the required data for each interval, including u-turn traffic volume \( (v_u) \), conflicting traffic volume \( (v_c) \), and conflicting traffic headway \( (h_c) \). The field capacity was also indirectly collected for validation. The u-turn field capacity in the undersaturated traffic condition was calculated from the average service time and average move-up time, as described in Equation 4.8. The service time \( (t_s) \), move-up time \( (t_{mv}) \), and u-turn field capacity are also shown in the Table 4.1.

The field u-turn capacity in the undersaturated traffic condition was calculated from the average service time and average move-up time as described in Equation 4.8. The u-turn service time was the average of u-turn waiting time in each interval, i.e. total waiting time of u-turn vehicle divided by total u-turn vehicles. On the other hand, the u-turn move-up time was averaging from all non-stop u-turn vehicles. The results were also shown in the Table 4.1. The scatter plots showing relationship between the field capacity and conflicting flow rate was illustrated in Figure 4.3. The collected data was in the limited ranges; 800-1,500 vph of conflicting flow rate and 200-500 vph of field capacity. Nevertheless, the figure shows the trend that the larger conflicting flow rate, the smaller field capacity. This is in line with the gap acceptance capacity model as shown in Figure 4.1.
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* The headway distribution was not fitted with the test Erlang distributions at the significance level of 0.05.
4.3.2 Gap Acceptance Parameters

The critical headway and the follow-up headway were determined for the whole 2-hours period on each day. To estimate the critical headway by maximum likelihood method, a pair of largest rejected headway and accepted headway for each u-turning vehicle is required. The largest rejected headway of a specific vehicle must be smaller than its accepted headway. The maximization of the log-likelihood function could be determined by the Microsoft Excel’s Solver. The mean critical headway and its variance could be calculated from the mean and variance of the distribution, according to Equation 4.7. On the other hand, the follow-up headway was averaged from all continuous u-turn events.

Table 4.2 presents the calculated distribution parameters from maximum likelihood method and the values and variances of critical headway and follow-up headway for each day. Although the data was collected from the same site, the critical headway and the follow-up headway on both days were not the same. In general, the traffic flow is different from day to day. Both gap acceptance parameters on the first day were larger than those on the second day. This implied the quicker u-turn movement on the second day of data collection. It could be verified by the larger u-turn volume on the second day even though the conflicting volume was larger. Normally, when the conflicting volume is larger, the u-turn volume is expected to be smaller according to the gap acceptance theory; less chance to find the acceptable gap. However, the u-turn volume on the second day was not smaller than that on the first day. This is consistent with the finding that the values of gap acceptance parameters were lower on the second day. The differences in driver behaviors might be caused by different driver population and different traffic condition on both days.

<table>
<thead>
<tr>
<th>Day</th>
<th>Distribution Parameter</th>
<th>Critical Headway</th>
<th>Follow-up Headway</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\mu$</td>
<td>$\sigma^2$</td>
<td>$t_c$</td>
</tr>
<tr>
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<td>1.57</td>
<td>0.21</td>
<td>4.9</td>
</tr>
<tr>
<td>2</td>
<td>1.53</td>
<td>0.16</td>
<td>4.7</td>
</tr>
</tbody>
</table>
4.3.3 Conflicting Headway Distribution

The type of conflicting headway distribution affects the u-turn capacity estimation as shown in Equation 4.5. Since the analysis period was set at 5-minute interval, the arrival headways of conflicting vehicles were collected for each 5-minute interval. In reality, the traffic arrival pattern changed from time to time. Checking the headway distribution in each interval could help improve the accuracy of capacity estimation.

The u-turn maneuver requires road space. The u-turn movement conflicts with the through movement on more than one lane. This study considered the conflicting through vehicles on median and middle lanes because most u-turn vehicles utilize those two lanes for their movements. The superimposed headways of the vehicles on median and middle lanes, as the u-turn vehicle faces, were considered as the conflicting headways. The parallel conflicting vehicles, coming at the same instance, were counted as one conflicting vehicle.

The headway distribution was determined by the Chi-square goodness-of-fit test with the significance level of 0.05. The test distributions included Erlang-1 (K=1; negative exponential), Erlang-2 (K=2), and Erlang-3 (K=3). The resulting fitted distribution for each interval is also shown in the Table 4.1. Of the total 48 intervals, 24 followed the Erlang-1, 21 followed the Erlang-2, 2 followed the Erlang-3, and the remaining 1 interval could not be fitted with the test distributions. Figure 4.4 categorized the conflicting headway distribution of the collected data points, based on the conflicting flow rate.

![Figure 4.4 Data categorized by conflicting headway distribution](image)

Normally, the headway distribution could be predicted by the traffic volume. According to Erlang distribution property, it is expected that the shape factor (K) increases as the traffic volume increases. However, based on the data collected in this study, the traffic flow rate could not predict the headway distribution. Instead, the arrival pattern in the traffic stream determines the headway distribution.
4.4 Capacity Estimation

4.4.1 Calculation Example

This section illustrates an example of capacity estimation from the collected data. From the data in Table 4.1 and Table 4.2, the input data for interval 1 were \( v_c = 984 \) vph, \( v_u = 300 \) vph, \( t_c = 4.9 \) s, \( t_f = 3.0 \) s, \( h_c = 2.5 \) s, and conflicting headway followed negative exponential distribution (Erlang distribution with \( K = 1 \)). The calculation could be conducted as below.

\[
c_{pu} = \left( \frac{984}{3600} \right) e^{-\left(\frac{984}{3600}(4.9)\right)} = 0.128 \text{ veh/s} \times 3600 \text{ s/hr} = 461 \text{ vph}
\]

\[
c_{pc} = \frac{3600}{2.5} = 1440 \text{ vph}
\]

\[
\frac{300}{461} = 0.65 < \frac{984}{c_{pc}} = 0.68
\]

To balance \( v_c \), decreased \( c_{pu} \) to \( c_u \) and increased \( c_{pc} \) to \( c_c \). The rate of adjustment followed the ratio of \( h_i \) and \( t_f \); \( \Delta c_u / \Delta c_c = -h_i / t_f \). Then solved to find the value of \( \Delta c_u \) and \( \Delta c_c \) to equalize \( v_c \).

\[
h_i = \frac{3600-461 \times 3.0}{984} = 2.3 \text{ s}
\]

\[
\frac{\Delta c_u}{\Delta c_c} = \frac{-16}{21} = -\frac{h_i}{t_f} = \frac{2.3}{3.0}
\]

\[
\frac{300}{461-16} = 0.67 = \frac{984}{c_{pc}} = 0.67
\]

So, \( c_u = 445 \) vph and \( c_c = 1461 \) vph. In addition, to determine the u-turn field capacity, the input data for interval 1 were \( t_s = 5.7 \) s and \( t_{mv} = 2.7 \) s.

\[
c_f = \frac{3600}{5.7+2.7} = 429 \text{ vph}
\]

The absolute percentage error of estimation in interval 1 could be calculated as:

\[
\left| \frac{c_u' - c_f}{c_f} \right| = \frac{445 - 429}{429} = 0.04 \text{ or } 4\%
\]

4.4.2 Estimated Capacity

Based on the input collected data and methodology above, the u-turn potential capacity and adjusted capacity by balancing \( v_c \) were estimated and shown in Table 4.3. The calculation process also gave the capacity of the conflicting traffic. The inputs for u-turn potential capacity included the conflicting headway distribution (to select which model to be used), the conflicting flow rate, the critical headway, and the follow-up headway. The u-turn flow rate was not necessary. This research considered the conflicting headway distribution in
order to improve the capacity estimation because it affected the gap availability. For the proposed capacity adjustment method, the additional inputs included the u-turn flow rate and conflicting traffic headway (to estimate the conflicting traffic capacity). The concept of v/c balancing required both the volume and capacity information. It seemed that the proposed method required more data; however, the method also yielded the conflicting traffic capacity according to the interaction between the two traffic streams.

The following sections discuss the characteristics of the result, the validation of the proposed method, and the developed capacity curves for practical application.

### Table 4.3 Estimated capacity by balancing v/c

<table>
<thead>
<tr>
<th>Interval</th>
<th>Flow Rate (vph)</th>
<th>U-turn Potential Capacity (vph)</th>
<th>Estimated Capacity</th>
<th>Interval</th>
<th>Flow Rate (vph)</th>
<th>U-turn Potential Capacity (vph)</th>
<th>Estimated Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( v_u )</td>
<td>( v_c )</td>
<td>( c_{mu} )</td>
<td>( c_{mc} )</td>
<td></td>
<td>( v_u )</td>
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<td>353</td>
<td>270</td>
<td>1758</td>
<td>48</td>
<td>336</td>
</tr>
</tbody>
</table>

* The capacity information was not available since the headway distribution was unknown.

### 4.4.3 Effect of Headway Distribution

The u-turn potential capacity, calculated based on gap acceptance theory, was firstly compared with field capacity. As the u-turn traffic movement from field observation was not saturated, the calculation of field capacity followed the concept of service time in an hour. The summation of average service time and move-up time is regarded as the average time that a minor stream vehicle is served by a traffic lane (see Equation 4.8). This is to evaluate the performance of the gap acceptance model in estimating the u-turn field capacity, before the adjustment by the proposed method. The comparison is shown in Figure 4.5. The results showed that the headway distribution of conflicting traffic affected the estimation as follows:
• when the headway followed negative exponential (Erlang-1) distribution, the gap acceptance model overestimated the field capacity;
• when the headway followed Erlang-2 distribution, the gap acceptance model underestimated the field capacity;
• when the headway followed Erlang-3 distribution, the number of samples were too few to conclude anything; however, based on the two available data points, the gap acceptance model predicted well.

![Graph](image)

Figure 4.5 Headway distribution and u-turn potential capacity

The above results showed clear sign about the effect of different conflicting headway distribution, except for the Erlang-3 distribution. The only two data points followed the Erlang-3 distribution. They were taken out from the further analysis. The u-turn capacity estimation by the gap acceptance model needed some adjustments. The capacity estimation when headway distribution followed negative exponential needed to decrease. The capacity estimation when headway distribution followed Erlang-2 needed to increase.

According to the past research (Kyte et al., 1991; Pollatschek et al., 2002), when the drivers wait longer, they tend to accept smaller gaps. In other words, the behavior of drivers affects their decision. This behavior could explain the above results. When the headways follow negative exponential distribution (random arrival process), there are more chances or higher probability to have large gaps for u-turn. The drivers feel relax and may not take the first large enough gap for u-turn maneuver. Instead, they are willing to wait for next large gaps. Therefore, the measured field capacity values are less than the theoretical estimation values. On the other hand, when the headways follow Erlang-2 distribution, there are less chances or lower probability to have large gaps for u-turn. The drivers feel difficult to make u-turn. They do not want to miss the first available large gap. They may behave more aggressive to take the smaller headway than their critical headway. So, the measured field capacity values are more than the theoretical estimation values. Nevertheless, the gap acceptance model assumes the consistent and homogenous driver behavior.
4.4.4 Validation of the Proposed Method

The validation of the proposed method is shown in Figure 4.6. This figure compares the field capacity with the estimated capacity by the gap acceptance model and the proposed methodology. The results showed that the proposed method, based on balancing v/c, yielded the better result than the gap acceptance model in term of lower MAPE value. The proposed method could improve the u-turn capacity estimation.

The previous research about u-turn capacity on six-lane streets by Liu et al. (2009) also refers to the MAPE when validating the capacity model. In their study, the model yields the MAPE of 17.8%. The MAPE is higher than their previous study on four-lane highway, which yields the MAPE of 11.3% (Liu et al., 2008a), but still acceptable considering that the capacity data were collected based on 5-minute interval. They explained that the MAPE is expected to decrease if the larger time intervals are used. Normally, the data analysis in a larger aggregate time period could decrease the data dispersion. They concluded that the model can be applied for the u-turn capacity estimation on six-lane streets.

![Figure 4.6 Comparison of u-turn potential capacity and adjusted capacity](image)

The traffic operation in urban area tends to reach the equilibrium. The drivers on one stream may care about the other conflicting streams. According to the observation at the u-turn junction, when the u-turn traffic has more queue or waited for longer time, the u-turn traffic tends to be more aggressive to make u-turn. At the same time, the conflicting through traffic tends to be willing to stop and allow the u-turn traffic to go. In theory, the through traffic should get priority over the u-turn traffic all the time. However, the major traffic does not always get priority in urban environment. Therefore, the concept of balancing two traffic streams could be valid at u-turn junctions on urban arterials. In this research, the traffic intensity in term of v/c was used to represent the traffic condition in each traffic stream. The v/c balancing of the two traffic streams could provide the better results, comparing to the traditional gap acceptance models.
4.5 Capacity Curves

To investigate the characteristics of the new method, two sets of trial capacity estimations were conducted. These trial estimations could illustrate the effects of different traffic volumes and different headway distribution on the calculated capacity values. This is to get overview properties in the real application. The calculation assumed some inputs and varied the traffic volume of both traffic streams as follows:

- constant critical headway \((t_c)\) of 5.0 seconds
- constant follow-up headway \((t_f)\) of 3.0 seconds
- constant conflicting traffic headway \((h_c)\) of 2.5 seconds
- u-turn traffic volume from 100 to 500 vph, with 100 vph increment
- conflicting traffic volume from 800 to 1600 vph, with 200 vph increment
- conflicting headway distribution of negative exponential and Erlang-2

Table 4.4 shows the results of capacity estimation based on the above assumptions by the proposed method of balancing v/c. To illustrate the characteristics of the results, the capacity curves, which were drawn based on the values on the table, for negative exponential and Erlang-2 conflicting headway distributions are shown in Figure 4.7 and Figure 4.8, respectively.

<table>
<thead>
<tr>
<th>Conflicting volume (vph)</th>
<th>U-turn volume (vph)</th>
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<th>200</th>
<th>300</th>
<th>400</th>
<th>500</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>(c_u)</td>
<td>(c_c)</td>
<td>(c_u)</td>
<td>(c_c)</td>
<td>(c_u)</td>
<td>(c_c)</td>
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</table>

Note: \(c_u\) = u-turn capacity (vph), \(c_c\) = conflicting capacity (vph)
According to the developed capacity curves, the effects of traffic volume and conflicting headway distribution on the estimated capacity based on v/c balancing are discussed in this section. Based on the proposed method, capacity estimates depended on traffic volume of both streams, representing their interactions. The different value of traffic volume in one traffic stream affected the capacities of both traffic streams. The higher u-turn traffic volume resulted in the higher u-turn capacity and the lower conflicting capacity. The
higher conflicting traffic volume brought about the lower u-turn capacity, but the lower conflicting capacity. The influential level of conflicting traffic volume on conflicting capacity decreased when the u-turn traffic volume increased.

Type of conflicting headway distribution did not affect the shape of the curves. Based on the estimated values, the capacities of Erlang-2 distribution was lower than those of negative exponential distribution (around 10%). The difference was higher when the conflicting volume is higher. For some traffic planning tasks which require a rough estimation, one could assume the negative exponential distribution to simplify the calculation, with a possibility of about 10% overestimated. It could be concluded that, for capacity determination, the Erlang-2 headway distribution was not much different from the general assumption of Poisson random arrival process or negative exponential headway distribution.

The u-turn capacity estimation by balancing v/c is different from the gap acceptance model in nature. For gap acceptance model, the conflicting volume affects the u-turn capacity but the u-turn volume does not. The model is developed based on the theoretical approach with ideal condition assumptions. In the real world, the traffic operation may not follow the premise conditions. The driver behaviors and the interaction between traffic streams could change the traffic condition. The proposed method in this chapter took into account the traffic interactions in the capacity estimation process. The observed real situation on urban arterials derived the assumption of v/c balancing.

4.6 Discussions

Since there are a lot of u-turn facilities at the midblock median openings on urban arterials, the reliable capacity analysis method is crucial for traffic planning and management. This chapter illustrated the application of the methodology described in the HCM 2010 for u-turn capacity estimation in developing countries, where the nature of driving would be different from the United States. The HCM 2010 applies the gap acceptance theory and assumes the negative exponential conflicting headway distribution. This chapter covered more types of headway distributions for a more accurate estimation. The result showed that the gap acceptance models seemed not so reliable and needed adjustment. The gap acceptance capacity might overestimate or underestimate the field capacity, depending on the types of conflicting headway distribution. The adjustment by inputting the local driving manner characteristics could improve the capacity estimation. The concept of balancing v/c was applied to determine the u-turn traffic capacity as well as the conflicting traffic capacity. The proposed method adjusted the potential capacity and resulted in the estimated balanced capacities.

The findings from the study in this chapter could lead to the following conclusions:

- Traffic operation in real world did not perfectly follow the gap acceptance model. Driver behavior and traffic interaction could affect the traffic operation.
- Capacity estimation by gap acceptance model might systematically overestimate or underestimate the field capacity.
- Balancing volume-to-capacity ratio could illustrate the traffic operation on urban streets.
- Interaction between traffic streams affected field capacity.
- The Erlang-2 headway distribution did not yield much different capacity estimates comparing to the popular negative exponential distribution.
This is a good example that practitioners should consider the local calibration when doing the transport/traffic planning and analysis based on the authorized manuals or handbooks from other countries. This study provided an improved estimation method for the capacity of u-turn and conflicting traffic streams on urban arterials. The traffic capacity is the basic useful information, guiding the engineers and planners to develop the appropriate traffic design and management strategies.

One of the limitations of this study was the amount of data collection. The results relied only on the data collected at a specific site. More data collection on other sites could confirm the results from this study. Since the conflicting capacity is a by-product from the calculation process, the validation from field observation is recommended for future study. Further studies could also focus on the expansion of this adjustment method to other locations and/or other traffic facilities such as all-way stop-controlled (AWSC) intersection, where the traffic interactions are normally observed.
CHAPTER 5 U-TURN TRAFFIC CONTROL

5.1 Introduction

There are a lot of u-turn facilities at the midblock median opening on urban arterials in developing countries. The u-turn movement at uncontrolled midblock median opening is based on the gap acceptance process. The behavior of u-turn movement is similar to the movement of the minor street traffic at the unsignalized intersections. The u-turn movement has a better sight distance, but needs more time and larger space. After the u-turn vehicles arrive at the u-turn junction, they wait for an acceptable gap in the main through traffic stream and complete the u-turn maneuver. However, when the conflicting traffic volume is high, it is very difficult to find a large enough gap for the u-turn movement. The police will sometimes present and control the u-turn junctions. This is to facilitate the u-turn traffic movement, avoid through traffic blockage by u-turn queue, and prevent unsafe forcing u-turn behavior.

The previous chapter shows that the longer time the driver waits at the stop line, the smaller gap the driver accepts. The waiting time of more than 30 seconds will frustrate the drivers to accept the significant smaller gap, which may lead to traffic safety problem. Factors affecting the u-turn decision have been investigated. The significant factors include gap size, conflicting speed, and waiting time. The queuing time does not significantly affect the u-turn decision. Because the waiting time significantly affects the decision of u-turn’s drivers, this chapter develops the function to estimate the waiting time and set the warrant to control the u-turn movement for safety purpose. The effect of police control at u-turn is also evaluated.

Three types of control strategies have been utilized at the u-turn facilities at the midblock median opening, including (1) no control, (2) police/human control, and (3) signal control. The goal of u-turn junction control is to maintain the traffic flow and safety. There are many traffic situations that need control, for instance, when the traffic volume on either main or u-turn traffic is high. As the u-turn movement is a non-priority traffic hierarchy, the waiting of the drivers affect their decisions. The longer waiting time the drivers face, the higher chance the drivers take a risk and accept a short gap.

Typically, the delay represents the service quality of the unsignalized intersection. The delay time comprises two parts, which are queuing time and waiting time. The queuing time is the excess time spending in line of queue. On the other hand, the waiting time is the time spending at the front position of the queue or at the stop line. For u-turn traffic, the result from the previous chapter shows that the waiting time significantly affects the u-turn decision while the queuing time does not. So, this chapter focuses on the estimation of waiting time of the u-turn vehicle when facing the through traffic stream. When the responsible agencies’ officers know a threshold value of unsafe waiting time, they can determine the u-turn control warrant in term of conflicting traffic volume.

The “waiting time” in this chapter is defined as the theoretical waiting time or service time at the front position of the queue. It does not include the queuing time and the distorted time due to undesirable driver’s behaviors. For example, the driver may wait shorter when he/she make a forcing u-turn. To avoid that unsafe behavior, the ideal waiting time when facing the through traffic stream should be considered. It is difficult to apply the empirical approach by field data collection because those data include undesirable driver’s behaviors, both cautious and aggressive.
Currently, the authorized formula to calculate the ideal waiting time is not available. This chapter applied the spreadsheet simulation to calculate the waiting time. A number of random sets of time-series gap sizes were generated according to the known distribution functions, representing the headway of the through traffic stream. The waiting time of a driver can be determined by accumulating all the rejected gap sizes, assuming a value of critical headway. This method is a simplified simulation without using any commercial traffic simulation software, which are costly and complex to use. This method can utilize the common spreadsheet software, i.e., Microsoft Excel.

The performance of traffic control at u-turn is still in doubt. In this study, the u-turn control by traffic police is also examined according to data availability. This chapter evaluates the effect of police control at the median opening u-turn on its discharge flow, comparing to the normal traffic operation when there is no police control. To conduct a fair comparison, the effect of conflicting traffic volume was neglected. The discharge flow was calculated based on the inverse of the discharge headway. In case of no police control, the headway of u-turning traffic was measured in the situation that conflicting traffic stopped for u-turn or no conflicting traffic.

The objectives of this chapter can be listed as follows:

- to formulate the u-turn waiting time model as a function of conflicting traffic volume,
- to develop a control warrant for u-turn traffic in order to enhance the flow and safety of traffic operation at the u-turn junction,
- to study the departure headway characteristics of continuous u-turn movement in case of with and without control,
- to evaluate the effect of police control at median opening on u-turn discharge flow, and
- to investigate the effect of median width on u-turn discharge flow and on u-turn maneuver.

The results showed that whether one u-turn vehicle or more utilizing the same gap had no effect on the waiting time of the vehicle at the front of queue. The relationship between waiting time as a dependent variable and conflicting flow rate as an independent variable was in the exponential form. The influence of distribution type was much higher than the effect of critical headway value. The u-turn traffic volume did not affect the estimation of the waiting time, theoretically. The police control could stabilize the u-turn movement and increase the u-turn discharge flow. The discharge flow decreased as the median width increased. However, the too narrow median reduced the discharge flow rate and reduced the efficiency of police control at u-turn.

5.2 Methodology

5.2.1 Waiting Time Determination

a) Basic Concept

The proposed method considers the arrival process of the conflicting traffic stream in the chronological order. When conflicting traffic vehicles arrive, they create gaps or headways. The terms “gap” and “headway” are interchangeable in this research even though the theoretical definitions are different. The latest HCM has replaced the term “gap” with “headway”. This might represent the practical data collection process: record headway
rather than gap. The waiting time is the summation of the rejected gap sizes before accepting the large enough gap. Therefore, the determination of the waiting time requires the chronological order of conflicting vehicle gaps. The critical gap/headway information is also required to determine which gaps are acceptable. The example of the gap size of conflicting traffic versus time of the day is shown in Figure 5.1.

![Figure 5.1 Decision based on chronological order gap events](image)

This chapter utilized the spreadsheet software (i.e. Microsoft Excel) to generate the random numbers, representing the conflicting vehicle gaps. The random numbers are generated based on the premise probability distributions. The waiting time for each u-turn can be estimated by the summation of the rejected gaps. The waiting time for the entire hour can be calculated by the arithmetic mean. The follow-up u-turn vehicles do not wait at the stop line and are excluded from the waiting time calculation.

b) Assumptions

The simulation requires some assumptions about the gap/headway distribution. When the vehicles arrive in random, the headway distribution is the negative exponential distribution. When the traffic volume is high, the movement of one vehicle affects or is affected by other vehicles. The vehicle arrival is not random anymore. The Erlang distribution can explain the traffic condition in the intermediate state, which lies between the random and constant headway states (May, 1990). When the shape factor (K) of the Erlang distribution equals to 1, it is the negative exponential distribution, which represents the random arrival process. Therefore, the Erlang distribution can cover a wide range of traffic conditions, by varying its shape factor.

In accordance with the works in Chapter 4, this chapter considered two cases of conflicting headway distributions, including negative exponential and Erlang-2 (K=2) distributions. On the other hand, the u-turn arrival was assumed to be random; its headway followed negative exponential distribution. The critical headway, which is the threshold the vehicles decide to make u-turn, was assumed to follow the log-normal distribution. The standard deviation of the distribution function was assumed to be 20% of the mean.
c) **Simulation Scenarios**

The simulations varied the conflicting traffic volume from 200 to 2,400 vehicles per hour (vph) with the increment of 200 vph. The critical headway was varied from 3 to 6 seconds with the increment of 0.5 seconds. The simulation scenarios were described in detail as below.

**Scenario 1: Queued u-turn, constant critical headway**

The initial scenario was the simplest one. The u-turn traffic was in a queued condition and waiting for u-turn all the time; therefore, all rejected gaps were considered. A constant critical headway value was applied for all drivers.

**Scenario 2: Random u-turn, constant critical headway, 1 u-turn/headway**

The second scenario generated the random arrival of u-turn vehicle. The u-turn vehicle arrival was combined with the conflicting traffic arrival. Each u-turn vehicle has waited since its arrival time until the time that the acceptable gap happens. Only the rejected gaps when u-turn vehicle exists were counted as waiting time. In this scenario, the u-turn traffic volume was considered in generating the random u-turn arrival. The constant critical headway value was applied for all drivers. This scenario assumed no follow-up u-turn. The following u-turn vehicle will wait for the next acceptable gap to make u-turn.

**Scenario 3: Random u-turn, constant critical headway, follow-up u-turn**

This scenario was similar to scenario 2 but allowed the follow-up u-turn maneuver when the gaps were large enough. So, the value of follow-up headway was also required for the simulation.

The above three scenarios assumed the critical gap to be consistent for all drivers. In the real world situation, the accepted gap size is varied by the driver behavior. Some cautious drivers reject the mean critical headway, while some aggressive drivers accept the smaller headway. Therefore, after selecting the best practical scenario from above, the simulation will include the random properties of the critical headway.

The required number of random seeds for each conflicting traffic volume fulfilled the following criteria: (1) a minimum of 15 random seeds and increase in the increment of 5 random seeds; (2) the percentage error from mean not larger than 5%; and (3) the value of error not larger than 1. However, for the random generation of critical headway (log-normal), the number of seeds was set at 30 seeds, which was sufficient according to the test run. The calculation of prediction error utilized the level of significance of 0.05.

5.2.2 **Waiting Time Function and Control Warrant**

The simulations above resulted in the ideal waiting time for each conflicting traffic volume level. For each value of critical headway and conflicting headway distribution, the relationship between waiting time and conflicting traffic volume could be determined by the regression analysis. The waiting time could be estimated as a function of conflicting traffic volume. The parameters could be estimated by the regression analysis.

This waiting time is the theoretical waiting time of the u-turn vehicle when facing the conflicting traffic. When the drivers wait for more than a threshold value, they may conduct a forcing u-turn maneuver, which leads to traffic safety concern at the u-turn junction. The u-turn control is needed in this situation. By knowing the waiting time
threshold value, we can determine the conflicting traffic volume that needs control by the inverse of the waiting time function.

5.2.3 Performance of Control at U-turn

a) Analysis Approach

Generally, the u-turn capacity model is based on gap acceptance process. The capacity of u-turn depends on the volume of conflicting traffic volume, headway distribution, and u-turn gap acceptance characteristics. The higher conflicting traffic volume, the lower u-turn capacity due to less chances for entering to the through traffic stream. The smaller value of either critical headway or follow-up headway for u-turn, the greater u-turn capacity.

This chapter intends to evaluate the performance of u-turn control. The direct measurement can be conducted by comparing the characteristics of continuous u-turn movement between when there is a control and no control. The comparison of u-turn movement at the same location obviously shows the effect of u-turn control. As police is often utilized during peak periods, the effect of such police control is evaluated. The police control represents the traffic control at u-turn. The behavior of individual police is neglected in this study.

The u-turn continuous movement is similar to the queue discharge phenomenon at a signalized junction. The effects of conflicting traffic characteristics are neglected for comparison purpose. The discharge headways of u-turn movement in both situations are determined and compared. The discharge headway is the movement headway when the subjected movement is in queued condition. In case of police control, it is the headway of the u-turn movement when the police blocks the conflicting traffic. The police plays the same role as the traffic signal. In case of no police control, it can be measured in the situations that the conflicting traffic stop and allow the queued u-turn traffic to enter into the traffic stream continuously. The discharge flow rate of the u-turn movement, when there is no conflicting traffic, is determined by the inverse of the discharge headway;

\[ S = \frac{3600}{h} \]  

Equation 5.1

where,

- \( S \) = discharge flow rate (vph)
- \( h \) = discharge headway (s)

The analysis of headway data determines the discharge headway. Firstly, the pattern of discharge headway has been plotted to see when the minimum discharge headway can be reached. If the discharge headway pattern followed the traditional trend, the discharge headway would be calculated by averaging the headway data starting from the stable queue position. However, the facilities under study are not really controlled by traffic signal. The departure headway may not be the same pattern as found at the signalized junctions. In that case, the data screening by observation is required in order to rule out the undesirable driver behaviors, such as too quick or too slow movements. In addition, to avoid the possible error from the inconsistent driver behavior, outliers were excluded from the data analysis. In this study, the outlier is any data point that is at least 1.5 times interquartile ranges (IQR) less than the lower quartile (Q1) or greater than the upper quartile (Q3). In other words, the data points used for analysis were between Q1-1.5*IQR and Q3+1.5*IQR.
The remaining headway data analysis applied the basic statistical process. The descriptive statistics were calculated to explain the characteristics of data. The statistics got from the data sets included mean or average, standard error of mean, standard deviation, median, minimum, and maximum.

In this study, the discharge headway and flow rate of u-turn movement are compared between control strategies at each site. In addition, the comparisons of the results from all sites are conducted to check whether there is an effect of different median widths.

**b) Data Collection**

To fulfill the objectives, the selected u-turn sites should be operated for both with and without police control. In addition, the major through traffic should frequently stop to allow u-turn traffic to move, in order to create the similar movement phenomenon as when police controls the u-turn for comparison purpose. The physical geometry characteristics of all sites should also be similar. The site locations should be in the same area to get the similar driver behavior, i.e. same driver population.

Consider the above criteria, four midblock median openings were selected for data collection. Those sites are located on the same six-lane divided street (three lanes in each direction) in western Bangkok, Thailand. The roadway section at one site has been expanded to eight-lane divided roadway (four lanes in each direction). There is an exclusive storage lane for u-turn in both directions at 3 sites while the site with 4 lanes in each direction has no u-turn bay. Therefore, the lane arrangement at u-turn section at all sites are similar; one u-turn lane and three through lanes in each direction. The width of median nose and receiving roadway is sufficient for u-turn maneuver at all sites. The site location and typical u-turn layout were shown in Figure 5.2.

![Figure 5.2 Location and typical layout of the selected u-turn sites](image)

The field data collection was conducted by video recording. A video camera was set on the nearby pedestrian bridge at each site to record the traffic movement during morning peak, afternoon peak, and off-peak periods. The video data was collected on two days at most.
sites, except Site 3 that required another two days video data to get more and sufficient data for further analysis.

The video data were reviewed to collect useful information in the laboratory. The first step was to identify the periods of continuous u-turn movement when the through traffic stopped, for both police control and no police control. After that, the time headway data were collected for all applicable periods. The headway data were analyzed separately for each site at each control condition.

5.3 Waiting Time from Simulation

5.3.1 First Scenario

The results of the first scenario were shown in Table 5.1 and Figure 5.3. The larger conflicting traffic volume makes the u-turn vehicles to wait longer. The smaller critical headway means the higher possibility to enter into the main traffic. So, the waiting time is lower when the critical headway is smaller.

Table 5.1 Waiting time for each conflicting traffic volume and critical headway

<table>
<thead>
<tr>
<th>Conflicting Volume (vph)</th>
<th>Waiting time (s) for each critical headway (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Negative Exponential Distribution</td>
</tr>
<tr>
<td>200</td>
<td>1.7</td>
</tr>
<tr>
<td>400</td>
<td>2.0</td>
</tr>
<tr>
<td>600</td>
<td>2.3</td>
</tr>
<tr>
<td>800</td>
<td>2.7</td>
</tr>
<tr>
<td>1000</td>
<td>3.0</td>
</tr>
<tr>
<td>1200</td>
<td>3.5</td>
</tr>
<tr>
<td>1400</td>
<td>3.9</td>
</tr>
<tr>
<td>1600</td>
<td>4.5</td>
</tr>
<tr>
<td>1800</td>
<td>5.1</td>
</tr>
<tr>
<td>2000</td>
<td>5.9</td>
</tr>
<tr>
<td>2200</td>
<td>6.9</td>
</tr>
<tr>
<td>2400</td>
<td>7.7</td>
</tr>
</tbody>
</table>

* The value of prediction error is more than 1; however, the percentage error is less than 1%.

![Figure 5.3 Relationships between waiting time and conflicting traffic volume](image-url)
As illustrated in Figure 5.3, the relationships between the waiting time and conflicting traffic volume were in exponential form. The type of headway distributions had much effect on the waiting time of the u-turn vehicle. The Erlang-2 headway distribution represents the more platoon of traffic stream, making the u-turn vehicles to wait longer until they could find the acceptable gap. The waiting time difference between the two distributions was high when the conflicting traffic volume was high. This implied that when the conflicting traffic volume was high, the type of headway distribution had much more effect than the value of critical headway. Therefore, the selection of headway distribution should be carefully considered.

5.3.2 Effect of U-turn Traffic Characteristics

To get the more realistic results, the u-turn traffic volume was considered in the simulations. The scenario 2 and 3 varied the amount of u-turn traffic to evaluate its effect on the waiting time estimation. The difference was the condition of follow-up u-turn on the same conflicting gap. The scenario 3 allowed the follow-up u-turn by assuming the follow-up headway.

The comparison of the simulation results was shown in Table 5.2. The results showed that the u-turn traffic volume did not affect the waiting time estimation, especially when the conflicting traffic followed negative exponential distribution. For Erlang-2 distribution, there were some differences in the value of mean waiting time when varying the u-turn traffic volume; however, the differences were very small (not larger than 0.5 seconds). On the other hand, the follow-up u-turn condition also did not affect the estimated waiting time. The simulation results of scenario 2 and scenario 3 were quite the same for all levels of conflicting and u-turn volumes.

<table>
<thead>
<tr>
<th>Conflicting Volume (vph)</th>
<th>U-turn Volume (vph)</th>
<th>Scenario</th>
<th>Negative Exponential</th>
<th>Erlang-2 Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Mean Waiting time</td>
<td>Standard Deviation</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>50</td>
<td>2</td>
<td>3.2</td>
<td>0.18</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>3.2</td>
<td>0.14</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>2</td>
<td>3.2</td>
<td>0.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>3.2</td>
<td>0.18</td>
</tr>
<tr>
<td>1000</td>
<td>50</td>
<td>2</td>
<td>7.8</td>
<td>0.58</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.8</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>2</td>
<td>7.9</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.9</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>2</td>
<td>7.9</td>
<td>0.59</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.8</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>2</td>
<td>7.9</td>
<td>0.59</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.9</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>2</td>
<td>7.9</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.9</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>500</td>
<td>2</td>
<td>7.9</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>7.9</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Note: The simulations assumed the critical headway of 5 s and the follow-up headway of 3.5 s.

As there was no effect of the follow-up u-turn, the scenarios 2 and 3 yielded the same results. Only the scenario 2 was simulated to reduce the additional input parameter, i.e. follow-up headway. The simulations applied a single value of u-turn traffic volume of 100.
vehicles per hour because the effect of the u-turn traffic volume was not much. The comparison of the results from scenario 1 and scenario 2, when the critical headway equals 5 seconds, was shown in Figure 5.4. The results showed that the differences were very small. Therefore, the scenario 1 was selected as the best practical scenario for the waiting time estimation in order to ease the simulation process. It could be concluded that the u-turn traffic characteristics (volume, queued or random, follow-up or not) did not affect the estimation of the waiting time.

![Figure 5.4 Comparison of the simulation results: scenario 1 vs scenario 2](image)

**Figure 5.4** Comparison of the simulation results: scenario 1 vs scenario 2

### 5.3.3 Effect of Random Critical Headway

The above analyses assumed that all drivers were consistent and homogeneous. The decision, whether accept or reject the individual gap, was based on a constant critical headway value. However, all drivers are different in the real world. Moreover, the decision of a specific driver on each gap is also random. The driver might reject the large headway but accept the same headway size later. This section considered the random nature of the critical headway. The critical headway was assumed to be log-normal distributed with standard deviation of 0.20 of mean. Figure 5.5 showed the comparison of the waiting time in case of constant and random critical headway.

![Figure 5.5 Comparison of the simulation results: constant vs random critical headway](image)

**Figure 5.5** Comparison of the simulation results: constant vs random critical headway
When the critical headway was random, the waiting time was lower than that in case of constant critical headway for high conflicting traffic volume, which was in line with the real world situation. The drivers do not want to wait too long. The level of difference increased as the critical headway and conflicting traffic volume increased. At the other end, the waiting time was very little higher in case of random critical headway when the conflicting volume is low. Because there are high possibilities of large acceptable gaps, the driver may relax and does not accept the first large enough gap. Nevertheless, the waiting time seems not so different if the driver accepts the followed gap.

The results of all simulations in case of random critical headway were shown in Table 5.3 and illustrated in Figure 5.6. This waiting time estimation might be more realistic and could be used to develop the waiting time function.

### Table 5.3 Waiting time in case of random critical headway

<table>
<thead>
<tr>
<th>Conflicting Volume (vph)</th>
<th>Waiting time (s) for each critical headway (s)</th>
<th>Waiting time (s) for each critical headway (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Negative Exponential Distribution</td>
<td>Erlang-2 Distribution</td>
</tr>
<tr>
<td>--------------------------</td>
<td>---------------------------------------------</td>
<td>---------------------------------------------</td>
</tr>
<tr>
<td>200</td>
<td>1.9 2.2 2.6 3.0 3.4 3.8 4.2</td>
<td>2.1 2.5 3.0 3.4 3.8 4.3 4.8</td>
</tr>
<tr>
<td>400</td>
<td>2.1 2.5 3.0 3.5 4.1 4.7 5.4</td>
<td>2.3 2.8 3.3 3.8 4.4 5.1 5.8</td>
</tr>
<tr>
<td>600</td>
<td>2.3 2.9 3.6 4.3 5.1 5.9 6.9</td>
<td>2.5 3.1 3.9 4.7 5.6 6.7 8.0</td>
</tr>
<tr>
<td>800</td>
<td>2.7 3.4 4.2 5.2 6.2 7.4 8.8</td>
<td>2.9 3.7 4.7 6.0 7.4 9.2 11.3</td>
</tr>
<tr>
<td>1000</td>
<td>3.0 3.9 5.0 6.2 7.6 9.4 11.3</td>
<td>3.4 4.5 6.0 7.8 10.1 12.8 16.3</td>
</tr>
<tr>
<td>1200</td>
<td>3.4 4.6 5.9 7.5 9.4 11.7 14.3</td>
<td>4.0 5.6 7.6 10.2 13.6 17.9 23.3</td>
</tr>
<tr>
<td>1400</td>
<td>3.9 5.2 6.9 9.0 11.4 14.4 17.9</td>
<td>4.7 6.8 9.6 13.3 18.3 24.9 33.6</td>
</tr>
<tr>
<td>1600</td>
<td>4.4 6.0 8.1 10.7 13.8 17.6 22.4</td>
<td>5.7 8.4 12.2 17.6 24.9 34.8 48.6</td>
</tr>
<tr>
<td>1800</td>
<td>5.0 7.0 9.5 12.7 16.7 21.7 28.5</td>
<td>6.8 10.4 15.6 23.0 33.6 49.3 71.8</td>
</tr>
<tr>
<td>2000</td>
<td>5.6 8.0 11.1 15.1 20.2 27.0 36.1</td>
<td>8.0 12.7 20.0 30.4 46.2 70.3 105.1</td>
</tr>
<tr>
<td>2200</td>
<td>6.3 9.2 12.9 17.8 24.4 33.7 45.1</td>
<td>9.6 15.7 25.4 40.2 63.6 99.2 153.8</td>
</tr>
<tr>
<td>2400</td>
<td>7.1 10.5 15.0 21.1 29.5 41.5 58.1</td>
<td>11.5 19.5 32.4 53.4 87.3 140.8 227.7</td>
</tr>
</tbody>
</table>

**Figure 5.6** Waiting time estimation in case of random critical headway

### 5.4 Waiting Time Function and Control Warrant

#### 5.4.1 Waiting Time Function

According to Figure 5.6, the relationship between conflicting traffic volume and waiting time was in the exponential form as shown in Equation 5.2. The regression analysis
estimated the parameters \((a \text{ and } b)\) as shown in Table 5.4. The coefficient of determination \((R^2)\) showed the perfect representation of the functions.

\[
t_w = ae^{bq/1000}
\]

where,

- \(t_w\) = waiting time (s)
- \(q\) = conflicting traffic volume (vph)
- \(a, b\) = parameters from regression analysis

Table 5.4 Estimated parameters from regression analysis

<table>
<thead>
<tr>
<th>Parameter Critical Headway (s)</th>
<th>3</th>
<th>3.5</th>
<th>4</th>
<th>4.5</th>
<th>5</th>
<th>5.5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>1.6373</td>
<td>1.9101</td>
<td>2.2132</td>
<td>2.5170</td>
<td>2.8152</td>
<td>3.1027</td>
<td>3.3892</td>
</tr>
<tr>
<td>(b)</td>
<td>0.6151</td>
<td>0.7148</td>
<td>0.8049</td>
<td>0.8949</td>
<td>0.9871</td>
<td>1.0854</td>
<td>1.1841</td>
</tr>
<tr>
<td>(R^2)</td>
<td>0.9992</td>
<td>0.9996</td>
<td>0.9995</td>
<td>0.9995</td>
<td>0.9996</td>
<td>0.9997</td>
<td>0.9998</td>
</tr>
</tbody>
</table>

\(R^2\) values were very close to the perfect representation of the functions. The value of parameters \(a\) and \(b\) increased as the critical headway increased. To illustrate the relationship, the parameters and critical headway were plotted as shown in Figure 5.7. The relationships were in the linear form for both parameters and both headway distributions.

\[
a = 0.5888t_c - 0.1375
\]

\[
b = 0.1879t_c + 0.0526
\]

Equation 5.3

Equation 5.2
For Erlang-2 distribution,
\[ a = 0.4022t_c + 0.4329 \quad \text{Equation 5.4} \]
\[ b = 0.3354t_c - 0.2137 \]
where,
\[ t_c = \text{critical headway (s)} \]

5.4.2 Recommended U-turn Control Warrants

Apart from the waiting time estimation, the conflicting traffic volume warrant for controlling a u-turn junction was also of interest. The conflicting traffic volume for a specific waiting time could be determined by the inverse of the waiting time function, as shown in Equation 5.5.

\[ q = \frac{1000}{b} \ln \left( \frac{t_w}{a} \right) \quad \text{Equation 5.5} \]

where,
\[ q = \text{conflicting traffic volume (vph)} \]
\[ t = \text{waiting time (s)} \]
\[ a, b = \text{parameters as estimated from critical headway by Equation 5.3 or Equation 5.4} \]

For example, the conflicting traffic headway followed the negative exponential distribution and the critical headway for u-turn was 4.3 seconds. If the driver could not wait longer than 30 seconds, the u-turn junction should be controlled when the conflicting traffic volume is above 2,938 vph for safety purpose.

The control warrant determination depended on the critical headway value and conflicting headway distribution. Those traffic characteristics were country-specific and required local calibration (Vasconcelos et al., 2012). The HCM recommends the based critical headway for u-turn movement of 5.6 seconds for six-lane highway (TRB, 2010). However, the estimation by the maximum likelihood in Chapter 4 yielded the critical headway of 4.7-4.9 seconds for the case of Bangkok. Table 5.5 illustrated the u-turn control warrant based on conflicting traffic volume, according to the HCM’s based critical headway for USA and the value from this research in Bangkok. The higher value of 4.9 seconds was picked to compare with the USA’s case in order to notice the minimum differences.

<table>
<thead>
<tr>
<th>Waiting Time Threshold (s)</th>
<th>Conflicting Traffic Volume That Needs Control (vph)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Negative Exponential Distribution</td>
</tr>
<tr>
<td></td>
<td>( t_c = 5.6 \text{ s (USA)} )</td>
</tr>
<tr>
<td>30</td>
<td>2038</td>
</tr>
<tr>
<td>45</td>
<td>2405</td>
</tr>
<tr>
<td>60</td>
<td>2665</td>
</tr>
<tr>
<td>75</td>
<td>2867</td>
</tr>
<tr>
<td>90</td>
<td>3032</td>
</tr>
</tbody>
</table>

The type of conflicting headway distribution had much effect on the volume warrant. As discussed in Chapter 4, the two types of headway distribution had not much effect in
capacity estimation. But for waiting time, the type of headway distribution had significant effect; consequently, the control warrant was much different. As shown in Table 5.5, the calculated warrants in case of Erlang-2 distribution were about 70% of the warrants in case of negative exponential distribution, for each waiting time threshold. The u-turn control warrants in Bangkok were about 20% more than the control warrants in USA, according to the difference in driving behaviors. The control strategy depends on the policy of responsible agencies. The control may be initially implemented by human control (e.g., police or staff). When the number of control hours in a day is increased, the agencies may decide to install a traffic signal for u-turn control, instead of human control.

5.5 Effect of Police Control at U-turn

5.5.1 Discharge Headway Determination

From the collected headway data, the average headway for each queue position of the u-turn queue was illustrated in Figure 5.8 and Figure 5.9 for the movement without and with police control, respectively. The headway data started from the second queue position, i.e. the departure time difference between the first and second vehicles.

From Figure 5.8, the discharge headway patterns in case of no police control were dispersed. On the contrary, the stable headway could be noticed when the police controlled the u-turn movement as shown in Figure 5.9. The result confirmed that the police control acted in the similar way as a signal control did. The minimum discharge headway could be reached from the seventh or eighth queue position.
Considering the dispersion of the headway data in case of no police control, the minimum discharge headway could not be determined from this dataset. The video data were observed again to exclude the data points with undesirable driver behaviors. To maintain the same analysis basis, the data screening was also applied for the police control case. In addition, the u-turn queue did not contain a lot of u-turn vehicles. Excluding the beginning of the queue would lose the large amount of useful data. The discharge headway patterns after the data screening were improved as shown in Figure 5.10. This dataset was used for the outlier analysis before determining the average discharge headway. After excluding the outliers, the statistics of discharge headway at each site with each control condition were calculated and shown in Table 5.6.
Table 5.6 Descriptive statistics of discharge headway data

<table>
<thead>
<tr>
<th>Site</th>
<th>Median Nose Width (m)</th>
<th>Case</th>
<th>No. of Sample</th>
<th>Mean (s)</th>
<th>Standard Error of Mean (s)</th>
<th>Standard Deviation (s)</th>
<th>Median (s)</th>
<th>Minimum (s)</th>
<th>Maximum (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.8</td>
<td>No Police</td>
<td>544</td>
<td>2.84</td>
<td>0.02</td>
<td>0.43</td>
<td>2.9</td>
<td>1.5</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>With Police</td>
<td>823</td>
<td>2.59</td>
<td>0.02</td>
<td>0.44</td>
<td>2.6</td>
<td>1.5</td>
<td>3.6</td>
</tr>
<tr>
<td>2</td>
<td>3.2</td>
<td>No Police</td>
<td>553</td>
<td>2.64</td>
<td>0.02</td>
<td>0.45</td>
<td>2.6</td>
<td>1.4</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>With Police</td>
<td>373</td>
<td>2.39</td>
<td>0.02</td>
<td>0.39</td>
<td>2.4</td>
<td>1.4</td>
<td>3.3</td>
</tr>
<tr>
<td>3</td>
<td>2.7</td>
<td>No Police</td>
<td>389</td>
<td>2.64</td>
<td>0.02</td>
<td>0.47</td>
<td>2.6</td>
<td>1.5</td>
<td>3.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>With Police</td>
<td>334</td>
<td>2.49</td>
<td>0.02</td>
<td>0.42</td>
<td>2.4</td>
<td>1.3</td>
<td>3.7</td>
</tr>
<tr>
<td>4</td>
<td>3.8</td>
<td>No Police</td>
<td>830</td>
<td>2.67</td>
<td>0.01</td>
<td>0.38</td>
<td>2.7</td>
<td>1.5</td>
<td>3.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>With Police</td>
<td>569</td>
<td>2.41</td>
<td>0.02</td>
<td>0.40</td>
<td>2.4</td>
<td>1.2</td>
<td>3.3</td>
</tr>
</tbody>
</table>

5.5.2 Police Control Effect

As shown in the previous section, the police control could obviously stabilize the u-turn traffic movement. From the statistical analysis results, the comparison of average discharge headway at different median widths on different control strategies was illustrated in Figure 5.11. The mean headway values of u-turn movement in case of police control were significantly lower than those in case of no police control at all sites. The hypothesis test results were shown in Table 5.7. This could indicate that the police control at u-turn could increase the u-turn discharge flow, even comparing with the same situation that the conflicting through traffic stopped. When the u-turn junction was controlled by police, the drivers were more confident to make u-turn quickly. Based on the field observation, the policeman who controlled the junction sometimes rushed the u-turn movement by his hand sign as well. On the contrary, when the through traffic stopped without police, the u-turn drivers still concerned on the conflicting traffic action, whether to let u-turn go or not. The hesitation time, which is the difference between the discharge headway of both controls, was approximately 0.25 seconds for all sites except Site 3.

Figure 5.11 Effect of police control and median width
Table 5.7 Hypothesis test of the difference in means

<table>
<thead>
<tr>
<th>Site</th>
<th>Variance Test (F-test)</th>
<th>Mean Test (t-test)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H_0: \sigma^2_{\text{no police}} = \sigma^2_{\text{with police}}$</td>
<td>$H_0: \mu_{\text{no police}} = \mu_{\text{with police}}$</td>
</tr>
<tr>
<td></td>
<td>$H_1: \sigma^2_{\text{no police}} \neq \sigma^2_{\text{with police}}$</td>
<td>$H_1: \mu_{\text{no police}} &gt; \mu_{\text{with police}}$</td>
</tr>
<tr>
<td>1</td>
<td>$F = 1.03 &lt; F_{cr} = 1.14$</td>
<td>$t = 9.95 &gt; t_{cr} = 1.65$</td>
</tr>
<tr>
<td></td>
<td>Accept $H_0$</td>
<td>Reject $H_0$</td>
</tr>
<tr>
<td>2</td>
<td>$F = 1.33 &gt; F_{cr} = 1.17$</td>
<td>$t = 9.10 &gt; t_{cr} = 1.65$</td>
</tr>
<tr>
<td></td>
<td>Reject $H_0$</td>
<td>Reject $H_0$</td>
</tr>
<tr>
<td>3</td>
<td>$F = 1.25 &gt; F_{cr} = 1.19$</td>
<td>$t = 4.51 &gt; t_{cr} = 1.65$</td>
</tr>
<tr>
<td></td>
<td>Reject $H_0$</td>
<td>Reject $H_0$</td>
</tr>
<tr>
<td>4</td>
<td>$F = 1.08 &lt; F_{cr} = 1.13$</td>
<td>$t = 12.26 &gt; t_{cr} = 1.65$</td>
</tr>
<tr>
<td></td>
<td>Accept $H_0$</td>
<td>Reject $H_0$</td>
</tr>
</tbody>
</table>

Note: Significance level ($\alpha$) = 0.05

The behavior of driver is subjective, difficult to predict, and different from place to place. In some places, the u-turning traffic might move quicker in case of no police control than in case of police control. The reason is that when there is police control, the drivers do not need to rush to make u-turn while the drivers need to rush for their movement when there is no police control. Nevertheless, the results from this study confirmed that the u-turn traffic moved quicker with less headway when there was police control. The u-turn drivers were afraid they could not make u-turns at that interval because the policeman may stop the u-turn movement when finding large gap.

The amount of discharge flow rate increase due to police control was summarized in Table 5.8. The discharge flow was increased at approximately 10% at all sites except the site with the narrowest median (2.7 m), which the discharge flow increase was only 6%. The median width might affect the discharge flow increase since the u-turn maneuver required the sufficient driveway space.

Table 5.8 Discharge flow increase by police control

<table>
<thead>
<tr>
<th>Median Nose Width</th>
<th>Case</th>
<th>Discharge Flow Rate (veh/hr)</th>
<th>% Increase of Police Control</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.7 m</td>
<td>No Police</td>
<td>1,365</td>
<td>6%</td>
</tr>
<tr>
<td></td>
<td>With Police</td>
<td>1,447</td>
<td></td>
</tr>
<tr>
<td>3.2 m</td>
<td>No Police</td>
<td>1,365</td>
<td>11%</td>
</tr>
<tr>
<td></td>
<td>With Police</td>
<td>1,509</td>
<td></td>
</tr>
<tr>
<td>3.8 m</td>
<td>No Police</td>
<td>1,346</td>
<td>11%</td>
</tr>
<tr>
<td></td>
<td>With Police</td>
<td>1,491</td>
<td></td>
</tr>
<tr>
<td>5.8 m</td>
<td>No Police</td>
<td>1,270</td>
<td>9%</td>
</tr>
<tr>
<td></td>
<td>With Police</td>
<td>1,388</td>
<td></td>
</tr>
</tbody>
</table>

5.5.3 Median Width Effect

The effect of median width on discharge headway could be seen in Figure 5.11. Except the narrowest median of 2.7 m, the wider median nose leaded to the greater departure
headway, and consequently the lower discharge flow rate of u-turn movement. The raised median might be considered as an obstacle for u-turning vehicle. The drivers have to carefully control their vehicles when making a u-turn at a wide median. Therefore, the wider median nose at u-turn, the larger headway to follow the leading vehicle. Since the width of median nose and roadway was sufficient for u-turn maneuver at all sites, there was no limitation of space for u-turn maneuver.

In case of normal gap acceptance u-turn, on the contrary, the u-turning vehicle prefers the wide median to narrow median. Since the u-turn maneuver needs large space to turn back, the wide median can compensate the required space. The u-turning vehicles can turn into the inner lane (close to median). The conflicting traffic is only the through vehicles on that lane. Therefore, the less conflicting traffic, the easier to make u-turn and the higher u-turn capacity. For narrow median, the u-turning vehicles have to conflict with two or more traffic lanes, making more difficult to complete the u-turn maneuver.

The median nose width of 2.7 m seemed quite narrow for comfortable u-turn maneuver. The u-turning vehicles could not use smaller discharge headway for their movement. The turning radius might not be sufficient for normal u-turn maneuver and the drivers had to turn steering wheel tightly. So, there would be a minimum median nose width for comfortable u-turn maneuver. Based on the result and comparison of 4 sites, the median nose width of 3.2 m could be the comfortable minimum width. However, the more site investigation should be conducted to find the exact relationships. The sites having median nose wider than 2.7 m and narrower than 3.2 m should be observed for discharge headway data collection.

For comfortable median width, the police control could improve the discharge flow at the same level (about 10%). Nevertheless, the contribution of police control was less at the site with uncomfortably narrow median.

5.6 Discussions

This chapter estimated the theoretical waiting time of the u-turn vehicle based on the stochastic random gap generation according to the known probability distributions by the spreadsheet simulation. The study developed the waiting time function, as a function of conflicting traffic volume and critical headway. The waiting time of u-turn vehicle is important because the driver may take a risk after waiting for a long time. Eventually, the control warrants could be determined by the inverse of the waiting time function. The effect of police control and median nose width on u-turn discharge flow had been evaluated by collecting the discharge headway at different sites.

The results showed that the relationship between waiting time as a dependent variable and conflicting flow rate as an independent variable was in the exponential form. The parameters of the functions could be estimated from the critical headway. The lower value of critical headway yielded the lower waiting time. The Erlang-2 headway distribution made the waiting time much higher than the negative exponential headway distribution. The type of headway distribution was quite sensitive. So, the headway distribution should be carefully selected when the waiting time was determined.

The u-turn traffic characteristics (volume, queued or random, follow-up or not) did not affect the waiting time estimation. The simple simulation by assuming the queued u-turn traffic yielded the indifferent results. When considering the randomness of critical
headway, the waiting time was lower than that in case of constant critical headway. The waiting time function was developed based on the random critical headway as it could be more realistic. The control warrant in term of the conflicting traffic volume for u-turn movement is essential to maintain the safe and smooth traffic operation.

This waiting time function was developed based on theoretical simulation. Therefore, it could be also applied to other highway facilities, which follows gap acceptance process, such as two-way stop-controlled intersection and roundabout. It should be kept in mind that the waiting time in this research was the waiting time at the front of queue only. It did not include the queuing time or represent any distorted time due to undesirable drivers’ behaviors.

The evaluation of police control at u-turn has been conducted according to the data collection at the selected sites. The comparison between the cases of with and without police control was based on the collected discharge headway. All sites had various sizes of median nose, which also affected the movement headway. The conclusions could be listed as follows:

- the police control could stabilize the u-turn movement and acted similar to the traffic signal;
- the discharge headway was less when police controlled the traffic operation at u-turn;
- the police control could increase the discharge flow rate of about 10%;
- the discharge flow rate was less at the sites with the wider median;
- the median nose width at u-turn should be wider than a critical minimum width for comfortable u-turn maneuver; and
- With too narrow median, the discharge flow rate was less and so was the efficiency of police control.

For design purpose, if a u-turn at midblock median opening were designed to be fully controlled by a traffic signal, the median width would be minimum but still provide comfortable u-turn maneuver. On the contrary, for uncontrolled u-turn junction, the median should be as wide as possible to give more chances for gap acceptance u-turn due to less interference to the conflicting traffic stream.

The limitation of this research was the number of study sites. The more data collection at other u-turn sites having median nose width in a range of 2.7-3.2 meters would be useful in order to determine the minimum preferable median width. The comparison between police-controlled and signal-controlled u-turn movement could also be studied in future. In addition, the swept path analysis should be conducted to get better understanding of the u-turn maneuver in traffic engineering point of view.
CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The u-turn at the midblock median opening on urban arterial exists to accommodate the travel to/from the places on the opposite direction. The traffic operation of u-turn vehicles interrupts the priority high speed through traffic. The u-turn maneuver is risky and may cause the accident. The situation requires the in-depth study of the u-turn traffic movement in order to understand its behaviors and mitigate its adverse potential impacts. The overall conclusions of this research, together with the highlighted findings, are summarized in Figure 6.1.

The goal of this study is to introduce a u-turn control to improve capacity and safety of the traffic operation at the u-turn location. To create a practical control measure, ones should know the behavior of the u-turn traffic movement. So, the u-turn behavior study is required to identify the influential factors affecting the u-turn decision. The only significant factors relating to the u-turn traffic is the waiting time of the driver. The longer waiting time at the front position of the queue induces the unsafe u-turn movement by accepting the smaller gap. The queue time when the driver waits in the line of queue has insignificant effect on u-turn decision. The question now is how to control the waiting time of the u-turning traffic, not to be longer than the driver’s endurance limit. The simple and direct answer is to let u-turn vehicle go by stopping the through traffic. Of course, the blockage of through traffic causes delay to the major traffic, which is expected to get priority all the time. However, the real world situation illustrates the forcing u-turn movement when the driver waits too long. No matter the u-turn at midblock is control or not, the u-turn vehicles have to go. Controlling the traffic movement at such midblock opening can help improve the traffic operation at the u-turn junction, in terms of traffic flow and safety.

Figure 6.1 Overall conclusions of the research
The capacity of the traffic movement is important for traffic engineering practice. The method to estimate the capacity of u-turn at midblock median opening should be addressed so that the related authorities could plan and manage the u-turn locations properly. Based on the published literature, the previous u-turn studies confirm that the HCM’s methodology can reasonably estimate the u-turn capacity. The method is based on gap acceptance model and assumes the Poisson arrival process of conflicting traffic. This study also utilizes the gap acceptance theory, but further applying the more specific headway distribution for the more accurate estimation. The results are not desirable. The model capacity overestimates the field capacity when the headway follows one type of distribution, but underestimates when the headway follows another distribution form. This study proposes an adjustment method based on v/c balancing in order to revise the capacity estimates. The proposed method yields the closer capacity estimates to the field capacity. The proposed method can be applied for the u-turn capacity estimation in an urban environment, where the traffic interactions are crucial. In practice, the engineer can assume the random arrival process of conflicting traffic, since the resulting capacity estimates are not much different. The study also affirms that the traffic operation in urban area tends to reach the equilibrium.

Developing the control warrant for u-turn requires the waiting time function. This waiting time should be the ideal waiting time that the driver faces in the gap acceptance process. The empirical approach, which utilizing the data collected at the site, could not provide the theoretical waiting time results because the drivers may behave inconsistently. For the purpose of setting the control warrant, this study employs the spreadsheet simulation to estimate the waiting time. The simulation has been conducted in various scenarios. The simplest scenario can provide the reasonable waiting time estimates, comparing to the results from the more complex scenarios. Therefore, the simulation task can neglect the effect of u-turn traffic characteristics. The concerned factors for waiting time estimation include the conflicting volume, conflicting headway distribution, and u-turn critical headway. Unlike the capacity, the waiting time is much affected by the conflicting headway distribution. The resulting waiting time function can be used to determine the control warrant, providing that the unsafe waiting time threshold is known. This is to guide the agencies’ staff to provide a control at u-turn location, when the conflicting traffic volume reaches the control warrant. The control can be implemented by human or signal, depending on the policy of responsible agencies.

Currently, traffic police is utilized to control the u-turn movement, especially during the peak periods. The police act as a traffic signal to stop the through traffic and allow u-turn traffic to go. However, there is a lack of study about the effectiveness of such control. This study also evaluates the performance of police control, comparing to the u-turn traffic operation without control. It is difficult to compare the traffic operation in different basis, i.e. controlled and uncontrolled. Therefore, the discharge headway of continuous u-turn movement in the two conditions are collected and compared. The departure headway is lower when there is a police control. The driver needs not to hesitate on the action of through traffic. The police control can increase the u-turn discharge flow rate of around 10%. The median width also affects the u-turn movement headway. The median is a physical obstacle for u-turn continuous movement. If a u-turn at midblock median opening is designed to be fully controlled by a traffic signal, the median width should be minimized, providing that the u-turn maneuver is still comfortable.

This study covers the necessary tasks for the improvement of the u-turn traffic operation at midblock median opening. The waiting time is found significant to the u-turn decision. The control warrant is based on the waiting time. The waiting time function is developed based
on the estimated waiting time by simulation. The u-turn junction should be controlled according to the conflicting traffic warrant. The effect of police control is also evaluated. In addition, this study evaluates the existing guideline to estimate the u-turn capacity and proposes an adjustment method to improve the estimation.

6.2 Recommendations

The results of this study should be implemented in the real world. The u-turn at midblock median opening should be controlled when the conflicting traffic volume is high. The appropriate level of conflicting traffic volume to control depends on the followings:

- the waiting time threshold
- the critical headway
- the type of conflicting headway distribution

As the driver behaviors may differ from place to place, the above parameters/values should be determined based on the same driver population. The hurriedness level in the central business area (CBD) is different from that in the suburb area. The local traffic agencies, which are responsible for the traffic control, should collect the necessary data and/or statistics to calculate the above values. Then the control warrant in term of conflicting traffic volume can be determined for all u-turn locations. Based on the traffic flow statistics, the responsible agencies can know when to dispatch their staff or police to control the u-turns properly. In addition, the staff/police allocation plan can be monitored and additional staff may be mobilized during peak periods. On the other hand, the traffic signal installation plan can also be implemented appropriately.

For the case study of Bangkok in this research, the waiting time threshold is 30 seconds. Assuming the conflicting vehicle arrival is random, the u-turn site should be controlled when the conflicting traffic volume is over 2,500 vph, according to Table 5.5.

6.3 Future Works

The future research can focus on the capacity and safety improvement of u-turn by other measures. Because the gap size and conflicting speed also influence the u-turn decision, the mitigation measures to control those factors can also improve u-turn operation. For instance, if the conflicting speed is controlled by speed limit or traffic calming, it will increase the chance for u-turn. However, it may cause unnecessary delay to the major through traffic. The further study is required for solving those issues.

To minimize the disturbance of u-turn traffic to the through traffic, an alternative u-turn design should be reviewed and studied. This design requires the more right-of-way because it provides space for u-turn traffic, without disturbing the through traffic. The detail of the u-turn movement swept path should be investigated for such design. The realization in the urban area is still in question, but challenging to put an effort in. The optimal interval between u-turn facilities is also interesting; in order to provide the smooth traffic operation on urban arterials.

Some u-turn facilities at midblock median openings are controlled by traffic signal. The future work can focus on the operation study of such u-turn movement. The performance of u-turn control by traffic signal and police can be compared. This is to evaluate the effectiveness of each type of control. The result can be used as the guideline for the traffic control at u-turn in the future.
REFERENCES


APPENDIX

GAP ACCEPTANCE CAPACITY MODEL DERIVATION

The gap acceptance capacity model can be represented as shown in Equation 4.2, as

\[
c_{ps} = \sum_{n=1}^{\infty} \left\{ v_c \times \left[ P(h > t_c + (n-1)\tau_t) - P(h > t_c + n\tau_t) \right] \times n \right\}
\]

The probability density function of Erlang distribution is shown in Equation 4.3, as

\[
f(t) = \frac{(qK)^K}{(K-1)!} e^{-qt} e^{-qt} ; \text{ in this case traffic flow rate } q \text{ is treated as conflicting flow rate } v_c.
\]

**Gap Acceptance Capacity in case of K=1**

\[
\int f(t)dt = \int (v_c e^{-v_c t}) dt = v_c \left( e^{-v_c t} - \frac{e^{-v_c t}}{v_c} \right) = -e^{-v_c t}
\]

\[
P(h \leq t) = -e^{-v_c t} \bigg|_{t=0}^{t=0} = -e^{-v_c t} - (-e^{0}) = 1 - e^{-v_c t}
\]

\[
P(h > t) = 1 - P(h \leq t) = 1 - (1 - e^{-v_c t}) = e^{-v_c t}
\]

\[
c_{ps} = \sum_{n=1}^{\infty} \left\{ v_c \times \left[ e^{-v_c (t_c + (n-1)\tau_t)} - e^{-v_c (t_c + n\tau_t)} \right] \times n \right\}
\]

\[
= v_c \left[ e^{-v_c (t_c)} - e^{-v_c (t_c + \tau_t)} \right] + v_c \left[ e^{-v_c (t_c + \tau_t)} - e^{-v_c (t_c + 2\tau_t)} \right] + v_c \left[ e^{-v_c (t_c + 2\tau_t)} - e^{-v_c (t_c + 3\tau_t)} \right] + \ldots
\]

\[
= v_c e^{-v_c (t_c)} \left[ 1 + e^{-v_c \tau_t} + e^{-2v_c \tau_t} + e^{-3v_c \tau_t} + \ldots \right]
\]

\[
= v_c e^{-v_c (t_c)} \left[ \frac{1}{1 - e^{-v_c \tau_t}} \right]
\]

\[
c_{ps} = v_c \frac{e^{-v_c (t_c)}}{1 - e^{-v_c \tau_t}}
\]

**Gap Acceptance Capacity in case of K=2**

\[
\int f(t)dt = \int (4v_c^2 t e^{-2v_c t}) dt = 4v_c^2 \int t e^{-2v_c t} dt = 4v_c^2 \left[ t \left( e^{-2v_c t} \right) - \frac{e^{-2v_c t}}{-2v_c} \right] - \frac{e^{-2v_c t}}{-2v_c} dt = 4v_c^2 \left[ t \left( e^{-2v_c t} \right) - \frac{e^{-2v_c t}}{-2v_c} \right]
\]

\[
= -2v_c^2 t e^{-2v_c t} + e^{-2v_c t}
\]

\[
P(h \leq t) = e^{-2v_c t} \left[ 1 + 2v_c t \right] \bigg|_{t=0}^{t=0} = 1 - e^{-2v_c t} (1 + 2v_c t)
\]

\[
P(h > t) = 1 - P(h \leq t) = 1 - (1 - e^{-2v_c t} (1 + 2v_c t)) = e^{-2v_c t} (1 + 2v_c t)
\]
\[ c_{pu} = \sum_{n=1}^{\infty} \left\{ v_x \left[ e^{-2v_c(t_c+(n-1)t_f)} \left( 1+2v_c(t_c+(n-1)t_f) \right) - e^{-2v_c(t_c+nt_f)} \left( 1+2v_c(t_c+nt_f) \right) \right] \right\} x n \]

\[ = \left\{ v_x \left[ e^{-2v_c(t_c)+(n-1)t_f} \left( 1+2v_c(t_c+(n-1)t_f) \right) - e^{-2v_c(t_c+nt_f)} \left( 1+2v_c(t_c+nt_f) \right) \right] \right\} x 1 + \]

\[ \left\{ v_x \left[ e^{-2v_c(t_c),(n-1)t_f} \left( 1+2v_c(t_c+(n-1)t_f) \right) - e^{-2v_c(t_c+2t_f)} \left( 1+2v_c(t_c+2t_f) \right) \right] \right\} x 2 + \]

\[ \cdots \]

\[ = v_x \left[ e^{-2v_c(t_c)+(1+2v_c(t_c+t_f))} + e^{-2v_c(t_c+t_f)} \left( 1+2v_c(t_c+t_f) \right) \right] + e^{-2v_c(t_c+(1+2v_c(t_c+2t_f)))} + e^{-2v_c(t_c+(1+2v_c(t_c+2t_f)))} + \]

\[ \vdots \]

\[ = v_x \left[ e^{-2v_c(t_c)} \left( 1+2v_c(t_c+t_f) \right) + e^{-4v_c(t_c)} \left( 1+2v_c(t_c+2t_f) \right) + \right] + e^{-4v_c(t_c)} + \]

\[ \cdots \]

\[ = v_x \left[ e^{-2v_c(t_c)} \left( 1+2v_c(t_c+t_f) \right) + e^{-4v_c(t_c)} \left( 1+2v_c(t_c+2t_f) \right) + \right] + e^{-4v_c(t_c)} + \]

\[ \cdots \]

\[ = v_x \left[ \left( 1-e^{-2v_c(t_f)} \right) + 2v_c \left( 1-e^{-2v_c(t_f)} \right) + 2v_c \left( 1-e^{-2v_c(t_f)} \right) \right] \]

\[ c_{pu} = \frac{v_x e^{-2v_c(t_c+t_f)}}{1+2v_c(t_c+t_f)} + \frac{2v_c e^{-2v_c(t_c+t_f)}}{1-e^{-2v_c(t_f)}} + \frac{2v_c e^{-2v_c(t_c+t_f)}}{1-e^{-2v_c(t_f)}} \]

Gap Acceptance Capacity in case of K=3

\[ f(t)dt = \left( \frac{27}{2} \right) \left( t^2 e^{-3v_c x} \right) dt = \frac{27}{2} \int t^2 e^{-3v_c x} dt = \frac{27}{2} v_c^3 \left[ t^2 \left( \frac{e^{-3v_c x}}{-3v_c} \right) \right] - e^{-3v_c x} \frac{2 t}{3v_c} dt \]

\[ = \frac{27}{2} v_c^3 \left[ t^2 \left( \frac{e^{-3v_c x}}{-3v_c} \right) \right] + \frac{2 t}{3v_c} \left[ e^{-3v_c x} \right] \]

\[ = \frac{27}{2} v_c^3 \left[ t^2 \left( \frac{e^{-3v_c x}}{-3v_c} \right) + \frac{2 t}{3v_c} \left[ e^{-3v_c x} \right] \right] \]

\[ = -e^{-3v_c x} \left[ \frac{27}{2} v_c^3 \frac{t^2}{3v_c} + 27 \frac{t}{3v_c} + 27 \frac{t^2}{3v_c} + \frac{2}{3v_c} - 9v_c^2 \right] \]

\[ = -e^{-3v_c x} \left[ \frac{27 v_c^3 t^2}{2} + 27 \frac{t}{3v_c} + \frac{2}{3v_c} - 9v_c^2 \right] \]

\[ = e^{-3v_c x} \left[ \frac{9v_c^2 t^2}{2} + 3v_c t + 1 \right] \]

\[ = e^{-3v_c x} \left[ 1+3v_c t + \frac{(3v_c t)^2}{2} \right] \]

\[ P(h\leq t) = 1 - e^{-3v_c x} \left[ 1+3v_c t + \frac{(3v_c t)^2}{2} \right] = 1 - e^{-3v_c x} \left[ 1+3v_c t + \frac{(3v_c t)^2}{2} \right] \]

\[ P(h> t) = 1 - P(h\leq t) = 1 - \left( 1- e^{-3v_c x} \left[ 1+3v_c t + \frac{(3v_c t)^2}{2} \right] \right) = e^{-3v_c x} \left[ 1+3v_c t + \frac{(3v_c t)^2}{2} \right] \]
\[ c_{pu} = \sum_{n=1}^{\infty} v_c \times \left[ e^{-3v_c(t_e+nnt_{f})} \left( 1 + 3v_c(t_e + (n-1)t_{f}) + \frac{(3v_c(t_e + (n-1)t_{f}))^2}{2} \right) \right] \times n \]

\[ = \left[ v_c \times \left[ e^{-3v_c(t_e+nnt_{f})} \left( 1 + 3v_c(t_e + (n-1)t_{f}) + \frac{(3v_c(t_e + (n-1)t_{f}))^2}{2} \right) \right] \times 1 \right] + \left[ v_c \times \left[ e^{-3v_c(t_e+t_{f})} \left( 1 + 3v_c(t_e + t_{f}) + \frac{(3v_c(t_e + t_{f}))^2}{2} \right) \right] \times 2 \right] + ... \]

\[ = v_c e^{-3v_c t_e} \left[ 1 + 3v_c t_e + \frac{(3v_c t_e)^2}{2} + e^{-3v_c t_{f}} \left( 1 + 3v_c t_{f} + \frac{(3v_c t_{f})^2}{2} \right) + e^{-6v_c t_{f}} \left( 1 + ... \right) + ... \right] \]

\[ = v_c e^{-3v_c t_e} \left[ 1 + 3v_c t_e + \frac{(3v_c t_e)^2}{2} + e^{-3v_c t_{f}} \left( 1 + 3v_c t_{f} + \frac{(3v_c t_{f})^2}{2} \right) + e^{-6v_c t_{f}} \left( 1 + ... \right) + ... \right] \]

\[ = v_c e^{-3v_c t_e} \left[ 1 + 3v_c t_e + \frac{(3v_c t_e)^2}{2} + e^{-3v_c t_{f}} \left( 1 + 3v_c t_{f} + \frac{(3v_c t_{f})^2}{2} \right) + e^{-6v_c t_{f}} \left( 1 + ... \right) + ... \right] \]

\[ = v_c e^{-3v_c t_e} \left[ 1 + 3v_c t_e + \frac{(3v_c t_e)^2}{2} + 3v_c t_e e^{-3v_c t_{f}} + \frac{(3v_c(t_e + t_{f}))^2}{2} + e^{-3v_c t_{f}} + ... \right] \]

\[ = v_c e^{-3v_c t_e} \left[ 1 + 3v_c t_e + \frac{(3v_c t_e)^2}{2} + 3v_c t_e e^{-3v_c t_{f}} + \frac{(3v_c(t_e + t_{f}))^2}{2} + e^{-3v_c t_{f}} + ... \right] \]

\[ = v_c e^{-3v_c t_e} \left[ 1 + 3v_c t_e + \frac{(3v_c t_e)^2}{2} + 3v_c t_e e^{-3v_c t_{f}} + \frac{(3v_c(t_e + t_{f}))^2}{2} + e^{-3v_c t_{f}} + ... \right] \]

\[ = v_c e^{-3v_c t_e} \left[ \frac{1}{1-e^{-3v_c t_{f}}} + 3v_c t_e e^{-3v_c t_{f}} + \frac{1}{1-e^{-3v_c t_{f}}} + 3v_c t_e e^{-3v_c t_{f}} + \frac{1}{1-e^{-3v_c t_{f}}} + A \right] \]

\[ A = \left( \frac{3v_c(t_e + t_{f})}{2} \right)^2 e^{-3v_c t_{f}} + \left( \frac{3v_c(t_e + 2t_{f})}{2} \right)^2 e^{-6v_c t_{f}} + \left( \frac{3v_c(t_e + 3t_{f})}{2} \right)^2 e^{-9v_c t_{f}} + ... \]

\[ = e^{-3v_c t_{f}} \left[ \frac{(3v_c(t_e + t_{f}))^2}{2} e^{-3v_c t_{f}} + \frac{(3v_c(t_e + 2t_{f}))^2}{2} e^{-6v_c t_{f}} + \frac{(3v_c(t_e + 3t_{f}))^2}{2} e^{-9v_c t_{f}} + ... \right] \]

\[ = \frac{e^{-3v_c t_{f}}}{2} \left[ (3v_c t_e)^2 + 2(3v_c)^2 t_e t_{f} + (3v_c t_f)^2 + e^{-3v_c t_{f}} (3v_c t_e)^2 + e^{-3v_c t_{f}} (3v_c t_f)^2 + \frac{4(3v_c)^2 t_e t_{f} + e^{-3v_c t_{f}} 4(3v_c t_f)^2}{2} + ... \right] \]

\[ = \frac{e^{-3v_c t_{f}}}{2} \left[ (3v_c t_e)^2 + 2(3v_c)^2 t_e t_{f} + (3v_c t_f)^2 + e^{-3v_c t_{f}} (3v_c t_e)^2 + e^{-3v_c t_{f}} (3v_c t_f)^2 + \frac{4(3v_c)^2 t_e t_{f} + e^{-3v_c t_{f}} 4(3v_c t_f)^2}{2} + ... \right] \]

\[ = \frac{e^{-3v_c t_{f}}}{2} \left[ (3v_c t_e)^2 + (1 + e^{-3v_c t_{f}} + ... + 2(3v_c)^2 t_e t_{f} (1 + e^{-3v_c t_{f}} + ... + 3e^{-6v_c t_{f}} + ... + 9(3v_c t_f)^2 + ... \right] \]

\[ = \frac{e^{-3v_c t_{f}}}{2} \left[ (3v_c t_e)^2 + (1 + e^{-3v_c t_{f}} + ... + 2(3v_c)^2 t_e t_{f} (1 + e^{-3v_c t_{f}} + ... + 3e^{-6v_c t_{f}} + ... \right] \]

\[ A = \frac{e^{-3v_c t_{f}}}{2} \left[ \frac{1}{1-e^{-3v_c t_{f}}} + 3v_c t_e e^{-3v_c t_{f}} + \frac{1}{1-e^{-3v_c t_{f}}} + 3v_c t_e e^{-3v_c t_{f}} + \frac{1}{1-e^{-3v_c t_{f}}} + A \right] \]

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Substitute $A$ into equation (i),

\[
c_{pu} = v_v e^{-3v_v t_f} \left[ \frac{1}{2} \left( 1 + 3v_v t_e + \frac{(3v_v t_f)^2}{2} + e^{-3v_v t_f} \left( \frac{1}{1-e^{-3v_v t_f}} + 3v_v t_f e^{-3v_v t_f} \right) \right) \right] e^{-3v_v t_f} \frac{1}{2} \left( 1 - e^{-3v_v t_f} \right) + \frac{1}{1-e^{-3v_v t_f}} \left( \frac{1}{2} \left( 1 + 3v_v t_e + \frac{(3v_v t_f)^2}{2} + e^{-3v_v t_f} \left( \frac{1}{1-e^{-3v_v t_f}} + 3v_v t_f e^{-3v_v t_f} \right) \right) \right) \right]
\]

\[
= v_v e^{-3v_v t_f} \left[ \frac{1}{2} \left( 1 + 3v_v t_e + \frac{(3v_v t_f)^2}{2} + e^{-3v_v t_f} \left( \frac{1}{1-e^{-3v_v t_f}} + 3v_v t_f e^{-3v_v t_f} \right) \right) \right]
\]

\[
= v_v e^{-3v_v t_f} \left[ \frac{1}{1-e^{-3v_v t_f}} \left( 1 + 3v_v t_e + \frac{(3v_v t_f)^2}{2} \right) + e^{-3v_v t_f} \left( \frac{1}{1-e^{-3v_v t_f}} + 3v_v t_f e^{-3v_v t_f} \right) \right]
\]

\[
= v_v e^{-3v_v t_f} \left[ \frac{1}{1-e^{-3v_v t_f}} \left( 1 + 3v_v t_e + \frac{(3v_v t_f)^2}{2} \right) + e^{-3v_v t_f} \left( \frac{1}{1-e^{-3v_v t_f}} + 3v_v t_f e^{-3v_v t_f} \right) \right]
\]

\[
= v_v e^{-3v_v t_f} \left[ \frac{1}{1-e^{-3v_v t_f}} \left( 1 + 3v_v t_e + \frac{(3v_v t_f)^2}{2} \right) + e^{-3v_v t_f} \left( \frac{1}{1-e^{-3v_v t_f}} + 3v_v t_f e^{-3v_v t_f} \right) \right]
\]

\[
= v_v e^{-3v_v t_f} \left[ \frac{1}{1-e^{-3v_v t_f}} \left( 1 + 3v_v t_e + \frac{(3v_v t_f)^2}{2} \right) + e^{-3v_v t_f} \left( \frac{1}{1-e^{-3v_v t_f}} + 3v_v t_f e^{-3v_v t_f} \right) \right]
\]

\[
c_{pu} = v_v e^{-3v_v t_f} \left[ \frac{1}{1-e^{-3v_v t_f}} \left( 1 + 3v_v t_e + \frac{(3v_v t_f)^2}{2} \right) + e^{-3v_v t_f} \left( \frac{1}{1-e^{-3v_v t_f}} + 3v_v t_f e^{-3v_v t_f} \right) \right]
\]