An Estimation Method of Exclusive Left-Turn Lane Capacity Considering the Bi-directional Flow of Crossing Pedestrians at Signalized Intersections

Doctoral Dissertation

Submitted in Partial Fulfillment of the
Requirements for the Degree of
Doctor of Engineering

By

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August, 2021

Abstract

Capacity is one of the most important indices for performance evaluation and signal timing design procedures of signalized intersections. Most of the time, traffic engineers are assigning concurrent signal timing design for crossing pedestrians and left turning vehicles (left hand traffic system) and facilitate the smooth traffic movement at signalized crosswalks by minimizing delay. Thus, pedestrian flow will be significant conflicting stream having major impact on the left turn lane capacity. The influence of crossing pedestrians on left turning vehicles is highly dependent on the individual or pedestrian platoon movement behaviour. Because, pedestrian crossing time can be highly influenced by variation of pedestrian platoon movement, traffic signal settings, crosswalk length and bi-directional interactions of pedestrians. The longest pedestrian-vehicle interaction time interval between crossing pedestrians and left turning vehicles is happening when the platoon of bi-directional waiting pedestrians crosses the crosswalk, at this time interval left turning vehicles are blocked by crossing pedestrians significantly. Consequently, the left turn lane capacity may considerably fluctuate depending on the composition of the bi-directional pedestrian demand. The duration of time that pedestrians occupy the shared space is highly dependent on the position of pedestrians with increasing elapsed time. In addition to the presence of crossing pedestrians the layout of the crosswalk has an impact on the left turning vehicles and pedestrian's movement.

However, in existing methods, the crossing pedestrian impact is estimated through an adjustment factor by considering the number of pedestrians as a factor to investigate the influence of pedestrians on left turning vehicles. Meanwhile, those estimation procedures do not express the realistic pedestrian-vehicle and bi-directional pedestrian interaction situations, for instance, without considering the bi-directional pedestrian flow interaction, waiting pedestrian platoon effect and change in geometric layout of the crosswalk. Therefore, this study aims to propose an estimation method of exclusive left turn lane capacity by empirical observation of different signalized crosswalks that can appropriately reflect the Japanese situations considering the bi-directional crossing pedestrians and crosswalk layout factors, i.e. bi-directional pedestrian demand, signal timing, crosswalk setback distance, stop line set back distance and crosswalk length. The result of this study is expected to apply for planning and operational stage analysis of signalized crosswalks regarding the treatment of pedestrians and crosswalk location issues in future.

For the development of the capacity estimation procedure, the conceptual settings are outlined in **Chapter 3** when bi-directional crossing pedestrians and left turning vehicles use the signalized crosswalks under concurrent signal phasing situation. And then, under different bi-directional pedestrian demand conditions the left turning vehicles discharge flow situations are conjectured. In general, the green time can be divided into five time intervals based on the basic bi-directional pedestrian flow characteristics. The five intervals have six critical time thresholds which determine the boundary of the time intervals. Finally, by combining the conceptual bi-directional pedestrian flow condition and left turning vehicles discharge flow within the available green time intervals the hypothetical capacity estimation equation is proposed.

To generalize the components of the proposed capacity estimation procedure, first the bi-directional pedestrian characteristics at the signalized crosswalks are observed and then modeled in **Chapter 4**. Bi-directional crossing pedestrian position distribution along the crosswalk at

higher and lower demand conditions are empirically observed at different signalized crosswalks. Furthermore, pedestrian presence probability is modeled by modifying existing time dependent model by considering crosswalk length, bi-directional pedestrian flow and signal timing settings. The model result implied that on longer crosswalks, pedestrian platoon tends to disperse. Besides, the model captured the platoon dispersion effect in the situation when the high demand of opposing pedestrian flow impedes on the subject direction pedestrian flow. Subsequently, by applying the pedestrian presence probability model the estimation procedure and the generalized model of waiting pedestrian's presence-time at the conflict-area of the crosswalk is defined. The pedestrian presence-time model can be applied to understand the influence of waiting pedestrians on left turning vehicles.

After the crossing pedestrian flow analysis, in **Chapter 5** the left turning vehicles discharge flow is observed. The observed discharge flow rate distribution at different signalized intersections showed that the position and slope of the distribution curve are different because of the variance in the number of arriving pedestrians per cycle and directional composition of arriving pedestrians. Subsequently, the discharge flow rate is modeled by selecting the significant factors i.e. influence of bi-directional arriving pedestrian presence and geometric layout conditions (crosswalk length, crosswalk and stop-line setback distances). The discharge flow rate model revealed that, when higher number of pedestrian's presence in the conflict-area the possibility of left turning vehicles to pass the conflict-area will be lower meanwhile the impact of far-side arriving pedestrian's presence is more significant than pedestrians entering from near-side direction. In addition, when the stop line is far from the intersection the arriving time of the left turning vehicle to the subject crosswalk will be long, due to that there will be delay to pass the conflict-area and unused green time may increase. On the other hand, when the subject crosswalk position is near to the intersection the left turning vehicles discharge flow rate show a decreasing trend. The reason is related with sight distance, left turning vehicles can easily notice the position and volume of crossing pedestrians at the onset of green and they will approach with lower turning speed, due to that the waiting time of turning vehicles before they pass the shared space will increase and consequently total number of turning vehicles within the available green time may decrease.

Chapter 6 outlined the validation and sensitivity analysis of the proposed method. The sensitivity of the proposed method is checked with different ranges of the five main components for the capacity estimation, i.e. signal timing settings, bi-directional pedestrian demand combinations, crosswalk length, crosswalk and stop-line setback distances. In addition, the proposed left turn lane capacity estimation method is compared with observed capacity at saturated cycles of signalized crosswalks and with selected existing left turn lane capacity estimation methods. The result reveled that; the proposed method can realistically capture the influence of crossing pedestrians and variation in crosswalk layout in the left turn lane capacity estimation procedures.

Application of the proposed method as an adjustment factors is also discussed in **Chapter 6**.A simplified equation is proposed to estimate the adjustment factors by considering the influence of bi-directional crossing pedestrians and variation in layout of the crosswalk at signalized crosswalks. The adjustment factors are useful for practitioners in the operational performance evaluation of signalized crosswalks and in the signal timing design procedures.

Acknowledgements

I would like to express my gratitude to all the people who have supported me in my research activity throughout my study.

Foremost, I would like to express my deepest gratitude to my advisor, Prof. Hideki Nakamura, for his patience, motivation, enthusiasm and immense knowledge. During the past three and a half years, I learned a lot from his invaluable suggestions and insightful guidance in regard to research. All of these are treasures for my future life.

I owe my deep gratitude to Associate Professor Miho Iryo Asano, her patient comments and detailed suggestions help me a lot. Her truly scientific manner has been inspiring and enriching my research, and I am indebted to her deeply. In addition, I would like to express my appreciation to Prof. Hirokazo Kato, he gave me meaningful comments in mid-term and interim presentations.

I would like to appreciate the helps from the Assistant Prof. Yuji Kakimoto and Assistant Prof. Xin Zhang for their care and support for my research when I asked them any request.

I owe my deepest gratitude to, Prof. Manfred Boltze (Darmstadt University of Technology), Prof. Keping Li (Tongji University), Prof. Bhargab Maitra (Indian Institute of Technology, Kharagpur), Associate Prof. Wael K. M. Alhajyaseen (Qatar University) and Prof. Ouguchi, who gave me many advices in different stages of my research progress.

Many thanks for all members in Nakamura Laboratory. Thanks to Dr. Zhu and Dr. Edwin, who always gave me inspiring advices. It is a pleasure to express my thanks to all of my fellow lab members, for their kindness and supports to me. It has been my nice experience with them full of happiness and great memories.

I would like to thank MEXT who gave me the scholarship to study my doctoral degree in Nagoya University. Finally, I would like to thank my family who care for and support me for my studying in Japan.

List of variables

c : Capacity of exclusive left turn lane (*veh/hr*)
C : Cycle length (*s*)
EPR : Exclusive pedestrian red time (*s*)
EPT : Expected waiting pedestrian-presence time (s)
f : Adjustment factor
G : Left turning vehicle green time (*s*)
L : Crosswalk length (*m*)
l_c : Crosswalk set back distance (*m*)
l_s : Stop line set back distance (*m*)
PG: Pedestrian green time (*s*)
Q : Discharge flow rate (*veh/hr*)
q_f : Far-side pedestrian arrival rate (*ped/hr*)
q_{far} : Number of far-side waiting pedestrian (*ped/cycle*)
q_n : Near-side pedestrian arrival rate (*ped/hr*)
q_{near} : Number of near-side waiting pedestrian (*ped/cycle*)
q_o : Number of waiting pedestrians from opposing direction (*ped/m/cycle*)
q_s : Number of waiting pedestrians from subject direction (*ped/m/cycle*)

Table of contents

Chapter 1

Introduction

1.1 Research background

In the urban transport planning process road networks are built by connecting links at nodes. These intersection points are the critical locations in the urban road network layouts. Pedestrians and vehicles are the two main users in traffic flow analysis and pedestrian-vehicle interaction is significant at nodes of the road network compared with the links. One of the methods to facilitate the smoother movement of pedestrians and vehicles at intersections is application of signalized intersection by adjusting the signal timing based on the pedestrians and vehicles movement conditions.

Operational performance of the signalized intersection is assessed by different procedures, like capacity analysis, delay, degree of saturation, safety and so on. Capacity is one of the methods applied to measure the performance of signalized intersection. The Highway capacity manual (HCM 2016) defines capacity as; "The maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions". Thus, in the capacity estimation procedures it is necessary to consider the possible influencing factors to have more realistic estimation. If the capacity estimation is not realistic then it will be difficult to have a proper signal timing design, consequently a problem of delay and congestion may happen at the signalized intersections. For effective operational performance assessment of the signalized intersections the presence of pedestrians around the analysis zone is a critical issue. Accordingly, this issue must be considered in all way of procedures by representative estimation of the pedestrian influence on the vehicle movements.

Turning lanes

The Highway Capacity Manual (HCM, 2016) identifies seven factors that influence which type of right-turn (right hand driving system) treatment should be considered at signalized intersections.

- Approach lane (shared or exclusive);
- Demand (generally based on hourly volumes);
- Sight distance at the intersection approach;
- Degree of saturation (elevated conflicting movements);
- Arrival patterns;
- Left-turn signal phasing on conflicting street; and
- Conflicts with pedestrians.

The capacity of a shared lane is not as high as the lane capacity for exclusive lanes. A shared lane can cause severe decline of intersection capacity, thus leading to an increase of traffic delay (Wu, 2011). As protected right-turn signal control is mostly applied at intersections with high traffic volume, a right-turn vehicle can block following straight-through vehicle during straight-through green phase and straight-through vehicles can also block following turning vehicles during protected right-turn green arrow phase. Furthermore, when blockage at shared lane occurs, some queuing vehicles will change to exclusive lanes. Therefore, the capacity estimation of shared turning lanes is complicated and it is difficult to estimate the realistic lane capacity, consequently in this study an exclusive turning lane is investigated so that the maximum realistic capacity of the turning lane can be computed. After that it can be applied to estimate the shared lane capacities by multiplying with the proportion of shared turning vehicles.

Shared space

At signalized intersections, crossing pedestrians and vehicles may have shared space and time on the crosswalk based on the assigned signal timing design. If there is a shared space or time with in the assigned green time of pedestrians and vehicles, then the interaction of crossing pedestrians and vehicles is an unavoidable condition. Most of the time, traffic engineers are assigning concurrent signal timing design for crossing pedestrians and turning vehicles and facilitate the smooth traffic movement at signalized crosswalks by minimizing delay. Therefore, understanding vehicle-pedestrian interaction is significant in concurrent signal timings. One of mostly happening example for pedestrian-vehicle interaction is when left turning vehicles (left-hand traffic system) and crossing pedestrians have shared signal timing interval. **Fig.1.1**. shows typical example of signalized crosswalk in Japan with concurrent signal phasing. Even if the signal timing is green for left turning vehicles, because of the presence of crossing pedestrians at the shared space the left turning vehicles forced to stop and give priority for pedestrians. Therefore, due to the interaction situation the operational performance of left turning vehicles movement is highly influenced by crossing pedestrians.

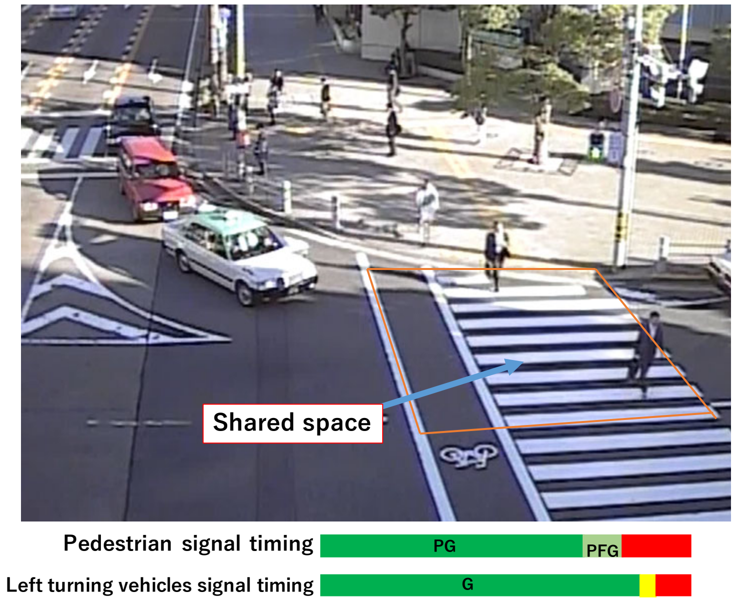


Fig.1.1 Shared space and time between left turning vehicles and crossing pedestrians

In order to understand the realistic impact of pedestrians on left turning vehicles, detailed investigation of the crossing pedestrian's behavior is necessary. Crossing behavior of pedestrians can be influenced by different factors when they cross along the crosswalk of signalized intersections and it may be expressed by the variation in walking speed. Pedestrian walking speeds at signalized crosswalk varies by various factors, such as traffic signal indication, age, gender, time of day, crosswalk length and interactions of pedestrians in the platoon.

In addition to the presence of crossing pedestrians the geometric layout of the intersection/crosswalk may have impact on the left turning vehicles and pedestrian's movement. The geometric layout of the intersection refers to the intersection angle, crosswalk setback distance, stop-line setback distance, turning radius and crosswalk length etc.

1.2 Research questions and problem statement

Research questions

In order to outline the objectives of the research and develop the methodology, the following research questions are summarized. Consequently, the research tries to answer these questions by setting proper procedures.

- How the pedestrian volume and crossing behavior influence left turn lane capacity?
- Which geometric layout parameter of the signalized crosswalks highly affects the left turning vehicles/crossing pedestrian behavior and then the left turn lane capacity?
- How to correlate the crossing pedestrian characteristics and left turn lane capacity under different crosswalk geometric layout condition within the available signal time period?

Problem statement

The interaction of pedestrians and vehicles on urban roadways is complex. At signalized crosswalks with assigned concurrent signal timing design for left turning vehicles and crossing pedestrians, investigation of the pedestrian-vehicle interaction at the shared space and time is critical. Although concerns for safety usually prevail, capacity is also important consideration in the design and evaluation of transportation facilities. The influence of crossing pedestrians on left turning vehicles is highly dependent on the individual or group of pedestrian's movement behaviour along the signalized crosswalk. Because, pedestrian crossing time can be highly influenced by variation of pedestrian platoon movement, traffic signal settings, crosswalk length and bi-directional interactions of pedestrian. The longest pedestrian-vehicle interaction time interval between crossing pedestrians and left turning vehicles is happening when the platoon of waiting pedestrians crosses the crosswalk, at this time interval left turning vehicles are blocked by crossing pedestrians significantly. The duration of time that pedestrians occupy the shared space is highly dependent on the position of each pedestrian with increasing elapsed time.

Moreover, the geometric layout of the intersection as overall and the layout of the subject crosswalk specifically may highly influence the left turning vehicles movement and their response for crossing pedestrian's presence i.e. in terms of variation in turning speed, deviation in arriving time to the shared space and visibility of crossing pedestrians etc. However, existing left turn lane capacity estimation methods did not consider those factors in their estimation procedures.

Therefore, it is still unclear that how crossing pedestrian platoon movement and bi-directional pedestrian flow influence the left turn lane capacity estimation. Besides, the combined impact of bi-directional crossing pedestrian flow and geometric layout of the signalized crosswalk on the left turning vehicles movement and influence on capacity estimation needs further investigations.

1.3 Objectives

The objective of this study is, to develop a method for the left turn lane capacity estimation under the influence of crossing pedestrians and different crosswalk geometric layout at signalized intersections. In order to achieve this objective, the following tasks are applied.

- Evaluate pedestrian-vehicle and bi-directional pedestrian interaction
- Investigate the influence of crossing pedestrians on left-turn capacity under different geometric layout condition.
- Observe the left turning vehicles movement with/without crossing pedestrian presence.
- Develop a left-turn lane capacity estimation method
- Propose an adjustment factors to consider the combined impact of bi-directional pedestrian flow and crosswalk layout on the left turn lane capacity estimation.

Then, the proposed adjustment factors can characterize the influence of pedestrians and geometric layout of the signalized crosswalks, which can be applied by practitioners in the operational performance evaluation of signalized intersections and in signal phasing design procedures.

1.4 Research flow and organization of the dissertation

In **Chapter 1,** the overall research background is introduced and then the problem statement of the research is explained. Furthermore, by considering the problem statement the research questions and the objective of the research are clearly specified. In general, in the first chapter the overall framework of the research is explained.

In **Chapter 2**, the literatures of existing researches related with capacity estimation methodology are assessed. The main research gaps in the existing methods are identified and discussed. The two main users around the signalized crosswalks are pedestrians and vehicles, therefore the characteristics of pedestrians and vehicles at the signalized crosswalks are summarized, and this is useful for understanding of possible influencing factors in the capacity estimation procedures.

In **Chapter 3**, the conceptual settings are developed when crossing pedestrians and left turning vehicles use the signalized crosswalks under concurrent signal phasing situation. Macroscopic flow of bi-directional crossing pedestrians and left turning vehicles movement are hypothesized. And then, under different bi-directional pedestrian demand conditions the left turning vehicles discharge flow situations are conjectured. Finally, by combining the conceptual bi-directional pedestrian flow condition and left turning vehicles discharge flow within the assigned signal timing the hypothetical capacity estimation equation is proposed.

Pedestrian characteristics at the signalized crosswalks are observed and investigated in **Chapter 4,** crossing pedestrian position distribution along the crosswalk at higher and lower demand conditions are observed at different signalized crosswalks. Bi-directional pedestrian flow circumstances are investigated. Furthermore, pedestrian presence probability along the crosswalk is modeled by considering crosswalk length, bi-directional pedestrian flow and signal timing setting. After that, by applying the model the estimation procedure of waiting pedestrian's presence-time at the conflict-area of the crosswalk is outlined. In addition, the proposed estimation procedure is compared with other existing presence-time estimation methods. Therefore, from the analysis of this chapter the influence of bi-directional waiting pedestrian platoon on the left turning vehicles movement is generalized.

The left turning vehicles discharge flow at the signalized crosswalk is investigated in **Chapter 5**, the left turning vehicles discharge flow under saturated and nearly saturated cycles of different signalized intersections are recorded. For each observed cycles, the condition of crossing pedestrians is simultaneously perceived. Subsequently, the discharge flow rate is modeled by considering the significant influencing factors i.e. influence of bi-directional pedestrian presence and geometric layout conditions (crosswalk length, crosswalk and stop-line setback distances).

In **Chapter 6**, the left turn lane capacity is estimated by selecting representative case study values. Moreover, sensitivity and validation of the proposed method is checked. The left turn lane capacity is estimated by changing the basic settings of the proposed method. The proposed method has three main components i.e. the influence of bi-directional pedestrians, crosswalk layout and the left turning vehicles discharge flow rate. Therefore, the capacity estimations under different bi-directional pedestrian demand combinations, crosswalk length, crosswalk and stop-line setback distances are evaluated. In addition, the proposed method left turn lane capacity estimation is compared with observed capacity at saturated cycles of signalized crosswalks and with selected existing left turn lane capacity estimation methods.

Adjustment factors to consider the influence of bi-directional crossing pedestrians and layout of the crosswalk at signalized crosswalks are also proposed in this chapter. The adjustment factors are useful for practitioners in the operational performance evaluation of signalized crosswalks and in the signal timing design procedures. Finally, conclusions and some outlooks for future research are provided in **Chapter 7**.

The general research framework is shown in **Fig. 1.2**

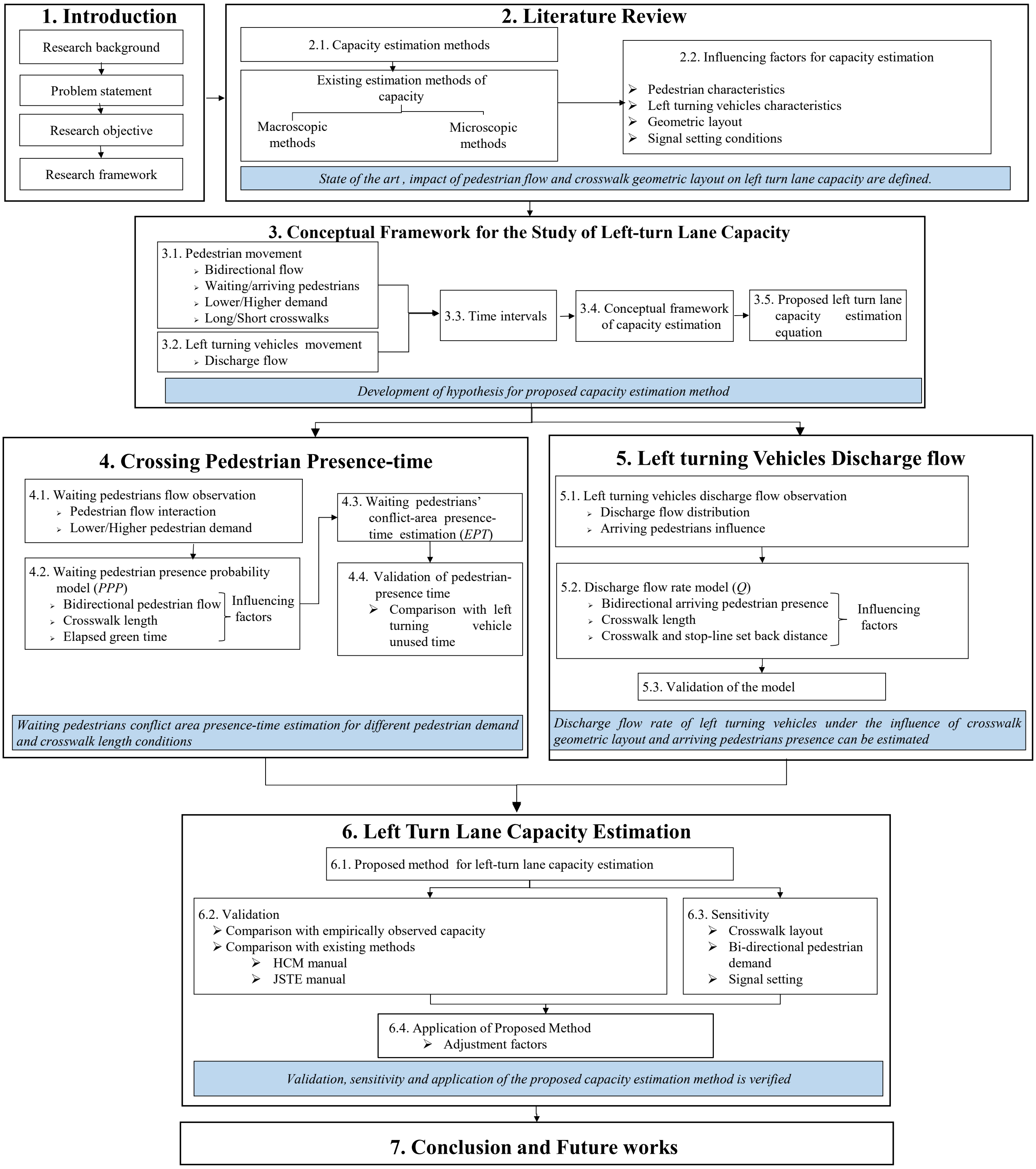


Fig.1.2 Research framework

Chapter 2

Literature Review

2.1 Overview

To study the left turn lane capacity estimation method by considering crossing pedestrians and the crosswalk layout, exploration of existing researches about the capacity estimation procedure and the existing manuals estimation procedures must be summarized. Furthermore, understanding of past researches findings about the crossing pedestrians and left turning vehicles characteristics in the signalized intersections needs detailed investigation. Therefore, in this chapter the state of the art, impact of pedestrian flow and existing estimation methods of capacity are explained.

2.2 Capacity estimation at signalized intersections

2.2.1 Background

Intersections may be signalized to address a road safety, efficiency or operational performance issue or to improve crossing opportunities for pedestrians. In urban arterial roads signalized intersections are generally installed at intersections of major roads. Signalized intersections have the following objectives (David, 1974):

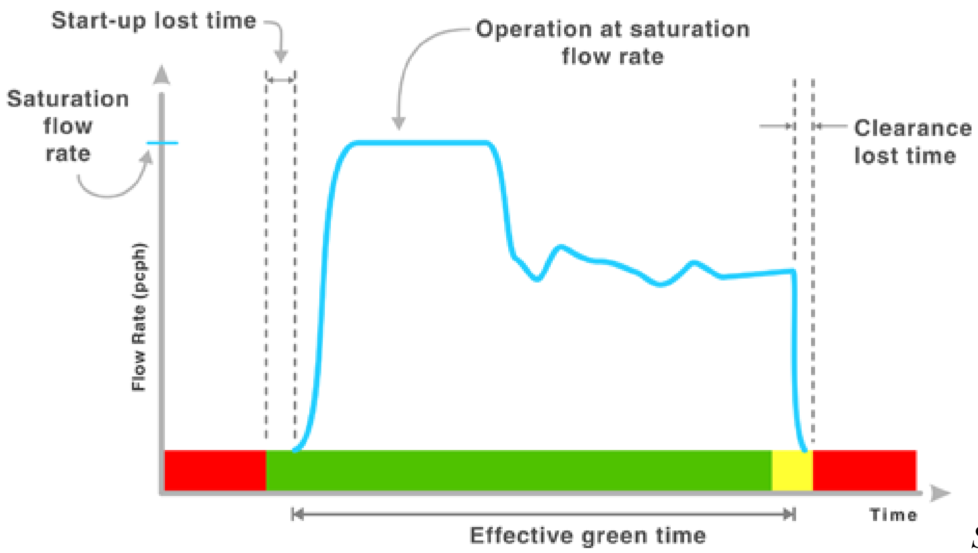
- The provision for the orderly movement of traffic

- When used in construction with proper physical layouts and control measures, to increase the capacity of an intersection
- Reduction in the frequency of certain types of accidents, especially the turning vehicles
- Under favorable conditions signals can be coordinated to provide for continuous or nearly continuous ·movement of traffic at a definite speed along a given route
- They may be used to interrupt heavy traffic at intervals to permit other traffic, vehicular or pedestrian, to cross.

Capacity of through lane and left turn lane

Capacity is one of the methods applied to measure the performance of signalized intersection. The Highway capacity manual (HCM 2016) defines capacity as; "The maximum sustainable flow rate at which vehicles or persons reasonably can be expected to traverse a point or uniform segment of a lane or roadway during a specified time period under given roadway, geometric, traffic, environmental, and control conditions". One of the critical capacities needed to be estimated at signalized intersection is the left turn lane capacity. Most of the time, the consideration of crossing pedestrian is necessary in the estimation procedure of left turn lane capacity if they have comment signal timing interval.

Fig 2.1 shows the basic operation of vehicular movement through a signalized intersection and the relationships among saturation flow rate, effective green time and cycle length as it is explained in HCM. The signal display is presented on the horizontal axis, the instantaneous flow of vehicles on the vertical axis. During the time while the movement is receiving a red indication, vehicles arrive and form a queue, and there is no flow. Upon receiving a green indication, it takes a few seconds for the driver of the first vehicle to recognize that the signal has turned green and to get the vehicle in motion. The next few vehicles also take some time to accelerate. This is defined as the start-up lost time or start-up delay and is commonly assumed to be approximately 2 seconds. After approximately the fourth vehicle in the queue, the flow rate tends to stabilize at the maximum flow rate that the conditions will allow, known as the saturation flow rate. This is generally sustained until the last vehicle in the queue departs the intersection. Upon termination of the green indication, some vehicles continue to pass through the intersection during the yellow change interval; this is known as yellow extension. The usable amount of green time, that is, the duration of time between the end of the start-up delay and the end of the yellow extension, is referred to as the effective green time for the movement. The unused portion of the yellow change interval and red clearance interval is called clearance lost time.



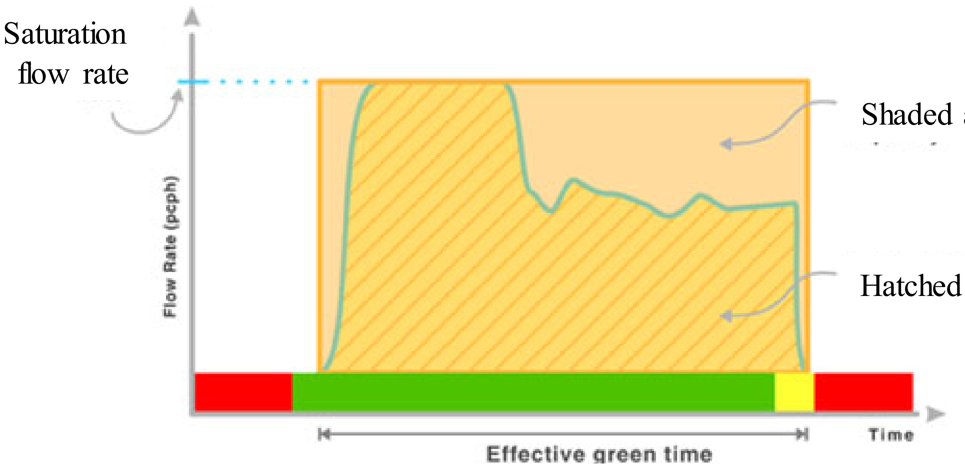
Source: FHWA

Fig. 2.1 Typical flow rates at a signalized intersection

Fig. 2.2 shows how the volume and capacity of signalized intersections computed based on the typical saturation flow rate illustration. Capacity is computed based on the ratio of effective green time and cycle length multiplied by saturation flow rate.

$$c = s\left(\frac{g}{C}\right) \tag{2.1}$$

Where c is the capacity (veh/hr), s is the saturation flow rate (pcu/hr), g is the effective green time (s) and C is the cycle length (s).



Source: FHWA

Fig. 2.2 Illustration of volume and capacity of a signalized movement

Saturation flow rate (s) is a significant parameter used to derive a signalized intersection's capacity. The HCM defines saturation flow rate as the equivalent hourly rate at which previously queued vehicles can cross an intersection approach under prevailing conditions, assuming that a green signal is available at all times and no lost time is experienced (HCM,2016). Saturation flow rate can be estimated by first measuring average saturation headway in the field, and then computed using Equation 2.2:

$$s = \frac{3600}{h} \qquad (2.2)$$

Where: s is Saturation flow rate (vehicle/hour), h is Saturation headway (second). The saturation headway is the average time gap between vehicles occurring after the fourth or fifth vehicle in the queue and continuing until the last vehicle in the initial queue clears the intersection (HCM, 2016). However, for left turning flow under concurrent signal timing due to the interaction of bi-directional crossing pedestrians every discharge vehicle has the potential to get influenced. Therefore, discharge flow or discharge headway of left turning vehicles show large fluctuation within the green phase. The left turning lane saturation flow rate and capacity also can be computed based on the above procedure of through movement of signalized intersections. However, for left turning vehicles movement analysis the effect of crossing pedestrians (if they have shared signal timing interval) and the influence of geometric layout of the signalized intersection have significant impact on the estimation procedure of saturation flow rate and capacity. Therefore, the realistic impact of crossing pedestrians influence must be investigated in the capacity estimation procedure.

Generally, the existing estimation methods for left turn lane capacity under concurrent signal timing design of left turning vehicles and crossing pedestrians can be classified into two approaches; Macroscopic and Microscopic approaches. Macroscopic approach focuses on modeling the overall relationship of left turn lane capacity and average number of crossing pedestrians, while microscopic approach focuses on left turning vehicles and pedestrian's individual behaviors.

2.2.2 Macroscopic and Microscopic capacity estimation methods

Capacity estimation methods can be broadly classified as macroscopic and microscopic methods. The basic analysis procedures in these two methods are discussed below.

Macroscopic methods

Macroscopic methods try to correlate the capacity with the influencing pedestrian flow by generalizing the group movement of pedestrian and vehicles. One example of commonly used macroscopic method is regression model. Regression models are developed in macroscopic estimation methods to reflect the relationship of left turn lane capacity and crossing pedestrian flow. Roshani M. and Bargegol, 2017, estimate the saturation flow rate and then capacities, considering the effect of pedestrian on right turn (right hand driving system) movements of the signalized intersections in Rasht city. Using different regression models, the best relationship between pedestrian's volume and right turning vehicles flow parameters was evaluated. Their results indicated that there is a primarily linear relationship between pedestrian volume and right turning vehicles flow with R^2=0.6261. Milazzo et al., 1998, investigated the relationship between crossing pedestrians and right turning vehicles (right hand driving system). They proposed a stepwise linear regression model to correlate the pedestrian volume and respective conflict-zone occupancy. After that, their model is used to develop an adjustment factor of pedestrian influence on right turning vehicles movement to calculate saturation flow rate and then capacity. Zhao J. et al., 2012, proposed an adjustment factor by using regression analysis, the model for estimating the pedestrian-bicycle adjustment factor for right-turn capacity (right hand driving system) was established by observing different signalized intersections in China. Pengpeng J. et al., 2014, studied the estimation of right turn lane capacity (right hand driving system), the key of their study is the right-turn capacity calculation formula based on statistical analysis, i.e., the vehicle capacity is presented as a curve with the capacity as the vertical axis and the pedestrian or bicycle flow as the horizontal axis, which is fitted using traffic survey data of six typical intersections in Beijing.

In general, in most of the macroscopic studies the pedestrian influence on left turning vehicles considered using pedestrian volume as the main factor without investigating the characteristics of waiting pedestrians and arriving pedestrians separately, in addition the situation of bi-directional pedestrian flow interaction and pedestrian platoon movement effects are not studied well.

Microscopic methods

Gap acceptance theory is one example of microscopic methods of capacity estimation. At priority-controlled intersections, there are traffic streams which have different ranks in the priority hierarchy. For example, the case of left turning vehicles and crossing pedestrians in concurrent signal phasing design. The capacity of minor flow is determined by the length of the acceptable gap which decides how many vehicles can cross in one accepted gap in the major flow and how the accepted gaps appear in major flow which is dependent on arrival patter of subjects in major flow. $E(t)$ is defined as the maximum number of vehicles can enter in one available gap at gap size of t sec and $h(t)$ is defined as the density function of headway distribution. Therefore, entry capacity c is calculated as the cumulative result of product of $h(t)$ and $E(t)$ at all levels of pedestrian flow q_p during certain period as shown in Equation 2.3.

$$c = q_p \int_0^{\infty} h(t)E(t)dt \tag{2.3}$$

Chen et al., 2008, investigated the pedestrian behaviors and pedestrian-vehicle conflict mechanism. In their analysis pedestrian arrivals in groups are also taken into account. Capacity of the conflict zone is modeled based on the interacting process analysis. In their analysis, right turn lane capacity yielded by the proposed model depends significantly on the average conflict times, and the downtrend of the results becomes less and less sharp with the increase of conflicting pedestrian volumes. Gao et al., 2016, studied the effect of pedestrian and bicycle infrastructure on the capacity for right-turn lane at signalized intersection. The number of right-turn cars going through signalized intersection was calculated by using probability theory and mathematical statistics.

In general, to apply the microscopic approach for the capacity estimation, it needs detailed investigation of individual crossing pedestrian's interaction with conflicting vehicles. Therefore, for safety analysis of signalized intersection microscopic approach is suitable; however, for capacity estimation it is may be to detailed procedure.

2.2.3 Existing manuals procedure to estimate left-turn lane capacity

The widely accepted method to estimate the left turn lane capacity is adjustment factor method. The adjustment factors are proposed by empirical or simulation analysis of left turning vehicles under the influence of crossing pedestrians. After that, the capacity is estimated by multiplying the base saturation flow rate with the adjustment factors.

HCM procedure

The HCM pedestrian and bicycle adjustment factors are proposed based on four phase empirical analysis. The first phase examines the relationship between pedestrian volume and the resulting occupancy of the conflict zone. The second phase, which applies only with opposing vehicular traffic, determines the amount of that occupancy that actually affects the saturation flow of turning vehicles.

The third phase considers the actual relationship between conflict zone occupancy and turning vehicle saturation flow rate. Quantification of this phase-three relationship required intersections at which permitted turns departing from a queue interact only with pedestrians or other non-motorized traffic, such as bicycles. The fourth and final phase applies this adjustment to a lane group, taking into account both the proportion of turning vehicles in the group and the proportion of turning vehicles using the protected phase. This phase merely involved an algebraic manipulation of formulas, so no data collection was performed for this last phase

Details of Recommended Procedure for Determining pedestrian-bicycle adjustment factor for right turns, fRpb , or left turns, fLpb as it is explained in HCM.

1) Calculate pedestrian conflict zone occupancy, *OCCpedg*.

First, get the pedestrian flow rate, V*pedg* from the conflicting pedestrian hourly volume, V*ped*:

$$Vpedg = Vped * (\frac{C}{g_p}) \qquad (Vpedg < 5000) \qquad (2.3)$$

Then, compute the average pedestrian occupancy during the effective pedestrian green time.

- *For pedestrian flow rates up to 1000 pedestrians/h green:*

$$OCCpedg = \frac{V_{pedg}}{2000} \qquad (Vpedg < 1000; OCCpedg < 0.5) \qquad (2.4)$$

- *For pedestrian flow rates between 1000 and 5000 pedestrians/h green:*

OCCpedg = 0.4 + *Vpedg* / 10,000 (1000 < *Vpedg* < 5000; 0.5 < *OCCpedg* < 0.9) (2.5)

2) Determine the relevant conflict zone occupancy from the driver's perspective, *OCCr*.

- *For a right turn with no bicycle interference or a left turn from a one-way street:*

  The relevant occupancy is exactly the pedestrian occupancy computed above, and: *OCCr* = *OCCpedg*

- *For a right turn with bicycle interference:*

  First convert bicycle hourly volume, Vbike, to bicycles/h green, Vbikeg:

  Vbikeg = V*bike* * (C/g) (Vbikeg < 1900) (2.6)

  Next, determine the relevant, combined occupancy of the adjacent pedestrian and bicycle conflict zones. Alternatively, determine the occupancy of the bicycle conflict zone by itself, *OCCbikeg*:

  OCC*bikeg* = 0.02 + V*bikeg* / 2700 (V*bikeg* < 1900; OCC*bikeg* < 0.72) (2.7)

  And then compute the relevant, combined occupancy, OCC*r*, by:

  OCC*r* = OCC*pedg* + OCC*bikeg* – (OCC*pedg* * OCC*bikeg*) (2.8)

- *For a left turn from a two-way street:*

  First check if opposing traffic screens the conflict zone for the entire effective green time: If g*q* > g*p* Then *fLpb* = 1.0 ; end procedure.

If the opposing queue does not consume the entire pedestrian green, determine the pedestrian occupancy after the opposing queue clears, OCC*pedu*.

$$OCC_{pedu} = OCC_{pedg} * (1 - 0.5 (\frac{g_q}{g_p})) \quad (2.9)$$

The relevant conflict zone occupancy after the queue clears is the occupancy that is not screened by additional opposing vehicles. To determine this relevant occupancy, OCC*r*, multiply the total occupancy after the queue clears, OCC*pedu*, by the probability that opposing vehicles do not screen the conflict zone.

$$OCCr = OCC_{pedu} * e^{-(5/3600)Vo} \quad (2.10)$$

3) Calculate the permitted phase pedestrian-bicycle adjustment for turning vehicles, A*pbT*.

- *If the number of receiving lanes equals the number of turning lanes (i.e., Nrec < = Nturn):*

  Vehicles cannot maneuver around pedestrians or bicycles, and the adjustment is logically the proportion of time the conflict zone is unoccupied from the turning driver's perspective.

  $$A_{pbT} = 1 - OCC_r \quad (2.11)$$

- *If the number of receiving lanes exceeds the number of turning lanes (i.e., Nrec > Nturn):*

  Vehicles may have opportunities to maneuver around pedestrians or bicycles, and the effect of pedestrian’s and bicycles on turning traffic is reduced.

  $$A_{pbT} = 1 - 0.6 * OCC_r \quad (2.12)$$

4) Compute the pedestrian-bicycle adjustment factor for right turns, f*Rpb*, or left turns, f*Lpb*.

- *For right turns, the pedestrian-bicycle adjustment factor, fRpb, is:*

  $$f_{Rpb} = 1.0 - PRT (1 - A_{pb}T)(1- PRTA). \quad (2.13)$$

- *For left turns, the pedestrian adjustment factor, fLpb, is:*

  $$f_{Lpb} = 1.0 - PLT(1 - A_{pb}T) (1 - PLTA) \quad (2.14)$$

List of variables used in HCM procedure

C	Cycle Length (s)
V_{ped}	Pedestrian Volume (pedestrians/h)
V_{pedg}	Pedestrian Flow Rate (pedestrians/h of green)
g_p	Pedestrian Green Time (Walk + Flashing Don't Walk), s
OCC_{pedg}	Average Pedestrian Occupancy During the Effective Pedestrian Green Time
V_{bike}	Bicycle Volume (bicycles per h)
g	Effective Green (for vehicles or bicycles/s)
V_{bikeg}	Bicycle Flow Rate (bicycles/h of green)
OCC_{bikeg}	Average Bicycle Occupancy During the Effective Green Time
g_q	Extent of Opposing Vehicle Queue (s)
v_o	Opposing Flow Rate After Queue Clears (vehicles/h)
OCC_{pedu}	Average Pedestrian Occupancy After the Opposing Queue Clears
OCC_r	Relevant Conflict Zone Occupancy from the Driver's Perspective
N_{turn}	Effective Number of Turning Lanes
N_{rec}	Effective Number of Receiving Lanes
A_{pbT}	Permitted Phase Pedestrian-Bicycle Adjustment for Turning Vehicles
P_{LT} ; P_{RT}	Proportion of Left or Right turns in Lane Group
P_{LTA} ; P_{RTA}	Proportion of Left or Right Turns Using Protected Phase
f_{Rpb}	Pedestrian-Bicycle Adjustment Factor for Right Turns
f_{Lpb}	Pedestrian Adjustment Factor for Left Turns

Canadian capacity manual

To consider the parallel movement of pedestrians and right turning vehicles (right hand driving system), in Canadian capacity guide of signalized intersections (2008) three empirical functions are developed in which they represent three Canadian cities. They can be used to determine the right turn saturation flow penetrating the adjacent pedestrian flow. Calculation of saturation flow adjustment factor for right turns crossing pedestrian flow can follow the following procedures.

1. Calculate the average pedestrian flow rate during the phase

$$q'_{ped} = q_{ped}\,(c / g) \tag{2.15}$$

2. If $q'_{ped} \leq 200$ and the intersection is in an area with few pedestrians on sidewalks $FR_{ped} = 1.0$
3. Calculate the adjustment factor using one of these functions

Toronto: $FR_{ped} = 0.60 - \frac{q_{ped}}{8516}$ (2.16)

Edmonton: $FR_{ped} = 0.44 - \frac{q_{ped}}{9320}$ (2.17)

Vancouver: $FR_{ped} = 0.44 - \frac{q_{ped}}{14100}$ (2.18)

Where q_{ped} is approximate average pedestrian flow rate during the walk interval and pedestrian clearance period (ped/h), q_{ped} is two-way pedestrian flow on the crosswalk (ped/h), c is cycle time (s), g is green interval (s) and FR_{ped} is saturation flow adjustment factor for right turns with pedestrians.

Finnish method

Capacity and Level of Service manual of Finnish Signalized Intersections (2002) proposed the saturation flow rate for a turning lane (S_e) with pedestrian conflicts as the following equation.

$S_e = 1692 - 1.13q_{ped}, \quad q_{ped} < 900 \quad or \quad S_e = 660 - 0.083(q_{ped} - 900), \quad q_{ped} > 900,$ (2.19)

Where, q_{ped} is the pedestrian flow rate in the pedestrian crossing.

The proposed estimation equation is computed based on measurements in the southern Finland supplemented with simulations (Niittymäki, 1998) of some signalized intersections.

Japan society of Traffic Engineers (JSTE) manual

According to Manual on Traffic Signal Control of Japan (2018), traffic capacity C_{LT} of the shared left turning lane is described in equation (2.20) and (2.21) considering the adjustment factor of the percentage of left turning vehicles α_{LT}.

$$\alpha_{LT} = \frac{100}{(100 - P_L) + P_L E_{LT}} \tag{2.20}$$

$$C_L = S_L \frac{G_P}{C} f_L + S_L \frac{(G - G_P)}{C} \tag{2.21}$$

Where, G: effective green time (sec), C: cycle length (sec), E_{LT}: adjustment factor of left-turning vehicle, P_L: percentage of LV (%), and C_L: traffic capacity of left-turning vehicles in (veh/h), S_L: saturation flow rate of left-turning traffic lane (pcu/hr), G_P: pedestrian green time (PG) + pedestrian flashing green time (PFG) (sec), and f_L: adjustment factor for left-turning vehicles. f_L is defined as the ratio of “the vehicle number which passed without stopping” among the total vehicle number which passed and stopping or stopped head leading vehicle” in subject time period (Ikenoue and Saito, 1972).

All the factors above are derived based on simulation results. Then different conditions are suited based on an ideal through-lane SFR of 2000 vphgpl, higher than that suggested by HCM. One obvious drawback in this JSTE guideline is that no analytical model embedded in the simulation is presented. Only reference values simulated at ideal situations (i.e. green split equals to 0.5; pedestrian volume per cycle is set to 5, 20, 40 and 60, respectively.) are given, which makes it difficult for practical SFR adjustment under non-ideal situations. (Kawai et al., 2005) also found the simulated factors might not satisfy all the boundary conditions in reality and sometimes significantly deviate from field observation.

All the adjustment factors are derived based on simulation results. After that, different conditions are suited based on an ideal through-lane SFR of 2000 vphgpl, higher than that suggested by HCM. One weakness in this JSTE guideline is that no analytical model inserted in the simulation analysis. Only reference values simulated at ideal situations (i.e. green split equals to 0.5; pedestrian volume per cycle is set to 5, 20, 40 and 60, respectively.) are given, which makes it difficult for practical SFR adjustment under non-ideal situations. Kawai et al. ,2005, also found the simulated factors might not satisfy all the boundary conditions in reality and sometimes significantly deviate from field observation.

2.3 Influencing factors for capacity estimation

To understand the realistic situation at the signalized intersections when concurrent signal phasing is applied, it is necessary to investigate the crossing pedestrian’s behaviour, the left turning vehicles movement characteristics and crosswalk geometric layout conditions.

2.3.1 Pedestrian characteristics at signalized crosswalks

Pedestrians are one of the users at signalized intersections. The operational performance of the intersection and the safety of the crossing pedestrians are highly related with the characteristics of individual and group pedestrians around the intersection.

Crossing pedestrian behavior

Crossing behavior of pedestrians can be influenced by different factors when they cross along the crosswalk of signalized intersections. The influence of pedestrians on left turning vehicles movement is highly related with the crossing behavior of pedestrians. Crossing behavior of pedestrians can be influenced by different factors when they cross along the crosswalk of signalized intersections like by their age, walking speed, gender and time of day etc.

A research by Akash et.al., 2014 and Abinav et.al., 2019, on the analysis of pedestrian crossing behavior by considering pedestrian characteristics like age, gender and that of carrying baggage influence on crossing speed and waiting time showed that there is a significant variation in waiting time and crossing speed due to the difference in the composition of crossing pedestrians. Muley et al., 2018, analyzed three types of pedestrian speeds (entry speed, crossing speed, and exit speed) at signalized crosswalks and their results showed that the pedestrian entry speeds were significantly affected by the pedestrian signal indications. In addition, the crossing speeds were positively correlated with crosswalk length for pedestrians crossing on green and red indications while pedestrian exit speeds were independent of crosswalk length but significantly affected by crossing direction. Zhu and Yang, 2004, analyzed pedestrian crossing speed characteristics at signalized intersections, their result showed that pedestrian had slower crossing speed when they were in larger groups which are one implication of pedestrian interaction effect on crossing speed. Furthermore, their analysis regarding crosswalk length effect on crossing speed showed that the average crossing speed increases with the increase of crossing distance. Goh and Lam, 2004, studied the relationship of pedestrian flow volume and walking speed, the study revealed that the pedestrian flow volume had significant effect on walking speed. Zhang et al., 2014, analyzed and modeled pedestrian walking speed at signalized crosswalks considering crosswalk length and signal timing. Their empirical analysis showed that, pedestrian walking speed increases as pedestrian green time proceeds. Moreover, their findings implied that longer crosswalks correspond to higher walking speed. Because of that, the duration of time that crossing pedestrians stay at the crosswalk is highly dependent on these factors. Park et al., 2014, investigated pedestrian crossing speed at signalized intersections with heavy pedestrian volumes. They attempted to model the relationships between pedestrian speeds and densities, and between speeds and directional proportions of pedestrian flows based on signalized intersections with heavy pedestrian volumes. Their findings showed that pedestrian crossing speeds within high density crosswalks are significantly affected by opposing pedestrian flows.

Existing considerations of pedestrian behavior

The existing guidelines or researches do not fully consider the effects of these factors on pedestrian flows for estimating turning vehicle capacity. For instance, (Highway Capacity Manual (HCM), 2016)) and a Planning and Design of at-grade Intersections - Basic Edition -; Guide for Planning, Design and Traffic Signal Control of Japan (JSTE manual 2018, here after) considered the influence of pedestrians for estimating capacity of turning lanes. The basic idea to consider the crossing pedestrian impact is based on the time occupancy of crossing pedestrians within green time. Zhang et al., 2012, quantitatively analyzed pedestrian impact on the capacity of right-turn vehicles at signalized intersections in China. They applied pedestrian-grouping model based on gap acceptance theory to estimate potential capacity. Chen et al. 2015, studied impact of pedestrian traffic on saturation flow of left-turn at urban intersections in Japan; they found that 93% of left-turn vehicles slowed down due to the presence of pedestrians. However, in the manuals and also other studies use the number of pedestrians as a factor without considering the bi-directional pedestrian flow, the effect of pedestrian platoon characteristic and change in geometric layout of the crosswalk. Roshani and Bargegol, 2017, investigated the effect of pedestrians on the saturation flow rate of right turn (right-hand traffic system) movements at signalized intersection in Iran. They did an empirical observation and model the relation between pedestrian flow and turning vehicles flow. Their results indicated that there is a primarily linear relationship between pedestrian volume and right turning vehicles flow.

Pedestrian flow

When the green time starts pedestrians waiting at sidewalks simultaneously starts to cross from both sides of the crosswalks. Generally speaking, these pedestrians forming platoons at the onset of green significantly contribute to block turning vehicle flows. Therefore, the maneuver of pedestrian platoons needs to be investigated. Alhajyaseen and Nakamura, 2009, proposed a methodology for modeling pedestrian discharge and crossing times by considering the effects of bi-directional pedestrian flow and crosswalk geometry on crossing time. However, they made an assumption that all the pedestrians in a platoon walk in a constant speed and constant distance between pedestrians. In reality pedestrians have different speeds and the distance between pedestrians will also vary within the green time. It is also important to understand the dispersion of the pedestrian platoon. Teknomo, 2006, Lee and Lam 2008 and Lu et al. 2015, studied the pedestrian flow by some simulation tools. These microscopic simulation models are able to represent platoon dispersion characteristics by representing avoidance/following behaviors between pedestrians, taking into account the relative locations and speeds of them at each moment. Despite of that, these models are not sufficient for the capacity evaluation of turning vehicles since they do not consider the impact of traffic signals and crosswalk geometries. On this study we applied a direct method to represent the probability distribution of the pedestrian existence without requiring the detailed maneuver of pedestrians. We applied an empirical analysis with macroscopic approach to model the pedestrian crossing behavior, then applied that to estimate the pedestrian presence-time at the conflict-area of signalized crosswalks, finally left turn lane capacity is estimated based on the pedestrian presence-time model under the influence of crossing pedestrians. Zhang and Nakamura, 2017, analyzed and modeled pedestrian presence probability along several signalized crosswalks by considering signal timing, pedestrian arrival rate and crosswalk length as influencing factors. They found that pedestrian position distribution along the crosswalk dispersed when crosswalk length and elapsed time of the pedestrian green time PG increase. Moreover, it was found that the distributions will shift to the moving direction slowly and their variations become larger when pedestrian arrival rate increases. However, they analyzed the sites with relatively lower pedestrian traffic demand only, and interaction with opposing pedestrian flow has not been investigated. Therefore, a methodology which considers pedestrian presence at the crosswalk at higher and lower pedestrian demand conditions and easy to use in macroscopic level is necessary for operational performance evaluation of signalized crosswalks.

In general, in the existing manuals and other previous studies use the number of pedestrians as a factor to consider the influence of crossing pedestrians on turning vehicles without considering the bi-directional pedestrian flow, the effect of pedestrian platoon characteristic and change in geometric layout of the crosswalk. However, the walking speed of crossing pedestrians is highly influenced by their interaction with nearby pedestrian’s movement and the perception of crossing pedestrians differ based on the layout of the crosswalk, i.e. at long crosswalks pedestrians may tends to cross quickly. Therefore, if those factors are not considered properly, the influence of pedestrians in the operational analysis of signalized intersections cannot be captured well. Consequently, this study will propose an estimation method by considering the above mentioned factors.

2.3.2 Left turning vehicles characteristics at signalized intersection

When left turning vehicles arrive at the signalized intersections, they have to interact with the signal timing, crossing pedestrians and also consider the geometric layout of the intersection. Therefore, the performance of the vehicles movement is highly related with those factors.

Impact of intersection geometric layout on turning vehicle

Left turning vehicles movement around the signalized intersections can be affected by different influencing factors. One of the important factors is the visibility of crossing pedestrians during the concurrent signal phasing conditions. When the left-turn vehicles share the same phase with pedestrians on divided lanes at intersection, the stopped buses or medium size vehicles that closed to the right-turn vehicles would block the view of drivers and pedestrians from the far-side of crosswalk. (Jin et al., 2014) studied the effects of restricted sight on right-turn driver (right hand driving system) behaviour with pedestrians at signalized intersection. They analysed the changes of lag/gap acceptance behaviour with/without considering sight obstruction and the headway of following right-turn vehicles by video observation. Their result showed that right-turn drivers tend to accept a smaller lag/gap with restricted sight, and it would lead to more potential conflicts between right-turn vehicles and pedestrians. However, larger headway of following right-turn vehicles with sight obstruction may result in delay and reduced capacity in right-turn lanes.

A technical report by Texas transportation institute (Fitzpatrick and Schneider, 2005) about the turn speeds of right-turn vehicles (right hand driving system) investigated the difference in turn speed of the turning vehicles at the beginning of right turn and at the middle of the right turn. Their result showed that the turning radius positively impacted turning speed at the beginning of the right turn. However, at the middle of the right turn the turning speed was positively affected by right-turn lane width in addition to turning radius. Therefore, according to this study the turning vehicles speed variation may influence the arrival time of turning vehicles, consequently, the capacity of the left-turn lane may decrease.

Asano et al. 2010, studied the the variation in the trajectory of left turning vehicles considering intersection geometry. The study intention was to analyse and model the trajectory variations of left-turning vehicles considering intersection geometric characteristics and the interaction with other users. Such variations in vehicle trajectories reflect the difference in driver behaviour which may lead to misunderstanding of other users' decisions and the analysis revealed a significant variation in trajectories depending on intersection angle, number of exit lanes and vehicle type.

Iasmin et al., 2015, studied the yielding behaviour of left turning vehicles for pedestrians under different intersection angle conditions. Their analysis showed that turning vehicles behaviour at obtuse angle intersections is more non-yielding than other intersections. Consequently, there will be variation in saturation flow rate of the turning lane.

Tarawneh and Mccoy, 1996, studied the effects of the geometries of right-turn (right hand driving system) lanes on the turning performance of drivers with respect to driver age and gender. Their results indicated that right-turn channelization affects the speed at which drivers make right turns and the likelihood that they will stop before making a right turn on red (RTOR). Drivers, especially middle-aged (25–45 years old) drivers, turn right at speeds 5 to 8 km/h higher on intersection approaches with channelized right-turn lanes than they do on approaches with unchannelized right-turn lanes. In addition, they argued that drivers are less likely to attempt to make an RTOR at a skewed intersection where the viewing angle to traffic from the left on the cross street is greater than 90 degrees. Likewise, drivers turning right at these locations are more likely to use their side mirrors than they are when making an RTOR at non-skewed intersections.

Interaction with pedestrians

Pedestrians have priorities at signalized intersections in the case of shared space and time with turning vehicles. Akin and Sisiopiku, 2007, studied the interactions between pedestrians and turning vehicles at signalized crosswalks operating under combined pedestrian-vehicle interval. Their result showed that the interactions between turning-vehicles and pedestrians at signalized intersection crosswalks are a function of the traffic flow of turning-vehicles and pedestrians. Nassereddine et.al, 2021, investigated the pedestrian-vehicle interaction and tried to establish a simple non-probabilistic regression model that explains the attitude of right-turning drivers (right-hand traffic system) at a specific intersection towards the presence of conflicting pedestrians. Their result confirmed that, when drivers perceive the possibility of pedestrian reaching a critical conflict point at the same time as them, they will modify their behavior even if not coming to a stop. Therefore, they argued that these interaction modeling can be used to evaluate the effectiveness of traffic signal control.

Hagiwara et al., 2008, investigated conflicts between the let-turning vehicle and the pedestrian coming from the right in the crosswalk in Japan. A field experiment was conducted to measure a driver's avoidance behavior according to the time lag, defined as the interval between the time that the right-turning vehicle reaches the conflict point and the time that the pedestrian coming from the left side reaches the conflict point. They revealed that the driver slows and enters the crosswalk behind the pedestrian, if the time lag at the conflict point is less than 2 s. Similarly, the braking location of left turning drivers who braked to avoid conflict with the pedestrian was stated as 10.3 m before the conflict point.

2.3.3 Geometric layout condition

One of the important factors that control the performance of signalized intersections is related with the geometric design arrangements. The geometric design around the signalized intersections may refer as turning curb radius, crosswalk setback distance, stop line setback distance, intersection angle, availability of channelization, approach road alignment, approach road gradient, road width etc.

Iasmin et al., 2015, studied the yielding behavior of left turning vehicles for pedestrians under different intersection angle conditions. Their analysis showed that turning vehicles behavior at obtuse angle intersections is more non-yielding than other intersections. Consequently, there will be variation in saturation flow rate of the turning lane. A study by Georges, 2012, about determining the ideal location for pedestrian crosswalks at signalized intersections suggested that increased pedestrian safety and congestion benefits result from recessed crosswalks. The benefits are most significant in dense urban areas that have large numbers of pedestrians and turning vehicles, while at intersections with very low volumes of pedestrians or where there are few turning vehicles, the benefits of recessed crosswalks are less substantial. Teply, 1990, investigated the combined effect of radius and pedestrians on right-turn saturation, he evaluated Canadian, Australian, and U.S, right-turn saturation flow adjustment procedures and he found that the consideration of radius in the saturation flow estimation is necessary. Therefore, a recommendation is proposed to express the combined effect of a small radius and the presence of pedestrians on right-turn lane capacity flow at signalized intersections. A study by Helmy et. al, 2018, about the impact of geometry and traffic characteristics on saturation flow rates at signalized intersections in Egypt showed that with the increase of the turning radius, the saturation flow rate increases.

2.3.4 Signal setting

Traffic signal timing is one of the most important responsibilities of traffic engineers and Saturation flow is a key input for optimal signal timing procedure. A small variation in saturation flow values could affect changes in cycle length and thereby affect the efficiency and operation performance of signalized urban arterial roads. In Japan, pedestrian signal phase consists of pedestrian's green time (PG), pedestrian flashing green time (PFG), pedestrian red time (PR). About the progress of the pedestrian crossing, when a pedestrian arrived at the pedestrian crossing before the PG starts, it will wait near the pedestrian crossing. After the PG shows, for the first few seconds' pedestrians will enter the pedestrian crossing as a platoon. The· Pedestrians arriving after the first few seconds can enter the pedestrian crossing without waiting. Then before the start of PR a PFG will be shown to facilitate the clearance of pedestrians from the crosswalk. After PFG, sometimes an early pedestrian red time interval (EPR) which provides an earlier finish of pedestrian green phase than the concurrent vehicle green phase is utilized for increasing the saturation flow rate of turning vehicular traffic. However, longer EPR will increase pedestrian delay and results in pedestrian signal violations. Therefore, the time interval assigned for the above mentioned signal timings will have significant impact on the crossing pedestrian's characteristics.

Yokozeki et al. 2016, confirmed that to reduce the number of pedestrians who are still on the pedestrian crossing after the red phase of the pedestrian is shown, the length of PFG rather than the length of PG must be extended. The longer the PFG, the more pedestrians will choose to enter the pedestrian crossing at the beginning of PFG, in the second half of PFG fewer pedestrians choose to go. Finally, after the red phase of the pedestrian is shown, the number of pedestrians on the crosswalk who are crossings will reduce. Clearance time in Japan is quite short, so that quite many pedestrians rush into crosswalks even after the onset of PFG (Suzuki et al., 2004). As a result, many pedestrians remain on crosswalks at the end of the phase and even some of them may try to enter at EPR time, that will significantly influence the discharge flow of left turning vehicles and obviously it will generate higher safety concern for the crossing pedestrians.

A study about saturation flow at signalized intersections during longer green time in the Dallas–Fort Worth (DFW) metropolitan area of Texas (Khosla, 2006) examined the fluctuation of headways under different green time settings. Their analysis of the data did not indicate a significant effect of longer green time when compared with their expectations. They mentioned that only one signalized intersection approach showed an increase in headway after about 60 s of green time. Even if they observe small variations, their result was not significant in concluding that longer green times had an effect on vehicle headways. In the same way a study by Boumediene et. al, 2009, investigated the saturation flow versus green time at two-stage signal controlled intersections, they found that saturation flow decreased slightly between 2 and 5 pcu/gh/sec of green time with increasing green time. However, the analyses of the discharge rate during the successive time intervals of 6-seconds showed a substantial reduction of 10% to 13% in saturation flow levels after 36 seconds of green time compared to those relating to 6–36 seconds range. They also mentioned that, no reduction in saturation flow levels was detected at the sites where only green periods of 44 seconds or less were implemented.

2.4 Summary

In this chapter the related researches and existing capacity estimation procedures are summarized. As it is discussed above, the left turn capacity estimation methods can be classified in to two; Macroscopic and Microscopic estimation procedures. While microscopic analyses treat individual pedestrian behavior and left turning vehicles interaction, macroscopic ones do pedestrians as a flow and aggregate their behavior to consider the overall influence on left turning vehicles. This study adopts the latter approach, a macroscopic analysis.

Existing left turn lane capacity estimation procedures proposed by different countries manuals (USA, Canada, Finland and Japan) are also investigated. In general, the existing manuals used the pedestrian volume as a factor to study the influence of crossing pedestrians on left turning vehicles. They proposed an adjustment factors based on fitted equations.

Furthermore, past studies about the crossing pedestrian's characteristics around the signalized crosswalks are summarized to understand the representative influence of crossing pedestrians on left turning vehicles. The investigated researches showed that the crossing pedestrian's behavior may vary by additional factors other than the pedestrian demand. Moreover, existing findings about the left turning vehicles characteristics in the signalized intersections by considering the crossing pedestrians are shortened. In addition, existing findings for the influence of geometric layout conditions and signal timing settings on the discharge flow of left turning vehicles or crossing pedestrians is also summarized.

In general, from the detailed literature reviews it is understandable that for effective left turn lane capacity estimation, realistic crossing pedestrians and left turning vehicles behaviors analysis is essential. In other words, there is a research gap about considering the effect of bi-directional pedestrian flow and pedestrian group platoon dispersion behavior on left turning vehicles movement, i.e. the walking speed of crossing pedestrians is highly influenced by their interaction with nearby pedestrian's movement and the perception of crossing pedestrians and left turning vehicles may differ based on the layout of the crosswalk, consequently the left turn lane capacity estimation may have significant variations. Moreover, from the literature review we can understand that the combined influence of signalized crosswalk layout and crossing pedestrians is not studied sufficiently.

Chapter 3

Conceptual Framework for the Study of Left-turn Lane Capacity

3.1 Overview

In the previous chapter the state of the art, impact of pedestrian flow and crosswalk layout on left turn lane capacity are defined, in addition the conditions in the existing manuals are also explained. The conceptual framework of the proposed capacity estimation procedure is discussed in this chapter based on the macroscopic empirical pedestrian/vehicle flows.

For the empirical study of crossing pedestrian and left turning flows the possible flow conditions and the interaction situations can be hypothesized. The hypothesis can be based on the basic understanding of pedestrian/vehicle movements within assigned signal time. Basically, with assigned exclusive left turn lane we have two directional pedestrian movements (from the two opposite sides of the crosswalk) and the left turning vehicles discharge from the left corner of the signalized intersections (left hand driving system). Shared left turn lanes are not covered by this study.

For concurrent signal timing, crossing pedestrians and left turning vehicles start to cross at the same time and the interaction of the crossing pedestrians and left turning vehicles will continue till the end of shared time interval. Therefore, the left turn lane capacity estimation procedure must consider these situations within the available shared time and space. Thus, this chapter tries to summarize the macroscopic approach for the pedestrian and left turning vehicles flows under the condition of concurrent signal phasing plan.

3.2 Bi-directional crossing pedestrian movement conditions

Pedestrians enter the crosswalk from opposite sides of the crosswalk as shown in **Fig.3.1**, i.e. there is a bi-directional pedestrian flow along the crosswalk. However, as it is discussed in chapter 2 existing researches and methods consider crossing pedestrians by merging the bi-directional movements. Therefore, the realistic consideration of pedestrian flows along the signalized crosswalks is not sufficiently represented. In some signalized crosswalks the crossing pedestrian demand from the two directions of the crosswalk may significantly varies, so that the interaction of crossing pedestrians would be highly influential on the crossing time of pedestrians. Consequently, the left turning vehicles movement also significantly influenced. In this study, the characteristics of the bi-directional pedestrian flow are investigated separately.

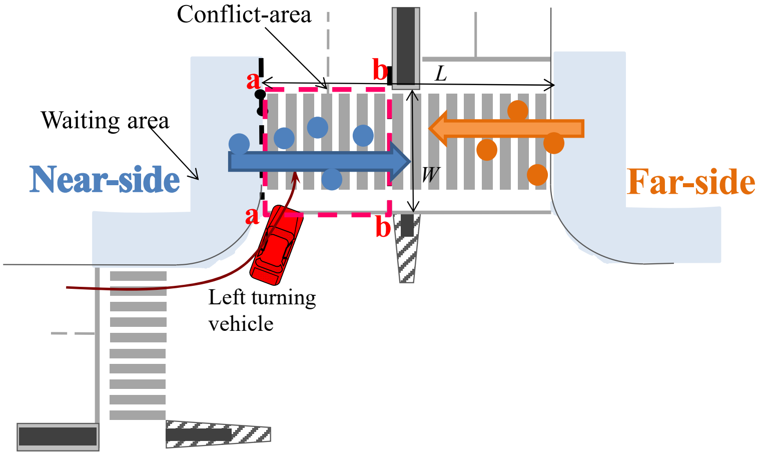


Fig.3.1 Crosswalk layout

Conflict-area is a shared space by crossing pedestrians and left turning vehicles on the crosswalk as shown in **Fig.3.1**. Left turning vehicles must give priority for crossing pedestrians if they arrive at the space at the same time. Near-side pedestrians are coming from the nearest side of the crosswalk to left turning vehicles. While, far-side pedestrians are coming from the opposite side from the near-side pedestrian flow and this is the farthest side of crosswalk for left turning vehicles. Crossing pedestrians are classified as waiting pedestrians and arriving pedestrians based on arrival timing. Waiting pedestrians are pedestrians who arrived at the crosswalk before the onset of pedestrian green time and arriving pedestrians are pedestrians who arrive at the crosswalk after the onset of pedestrian green time.

3.3 Green time intervals

Under concurrent signal timing design, the available pedestrian and left turning vehicles green time may divide in two general portions;

a) *Necessary time interval for near-side and far-side waiting pedestrian platoon to cross the conflict-area*. In this portion, the possibility for left turning vehicles to pass the conflict-area is low. Because, the near-side and far-side waiting pedestrians platoon cross the conflict-area in a group with small gap between them. Moreover, the pedestrian-pedestrian interaction is significant in this time interval portion.

b) *Available passing time interval for left turning vehicles*. In this portion, left turning vehicles pass the conflict-area by searching acceptable gap between randomly arriving pedestrians from both directions of the crosswalk.

The two portions can be subdivided in to different time intervals as shown in the **Fig.3.2**.

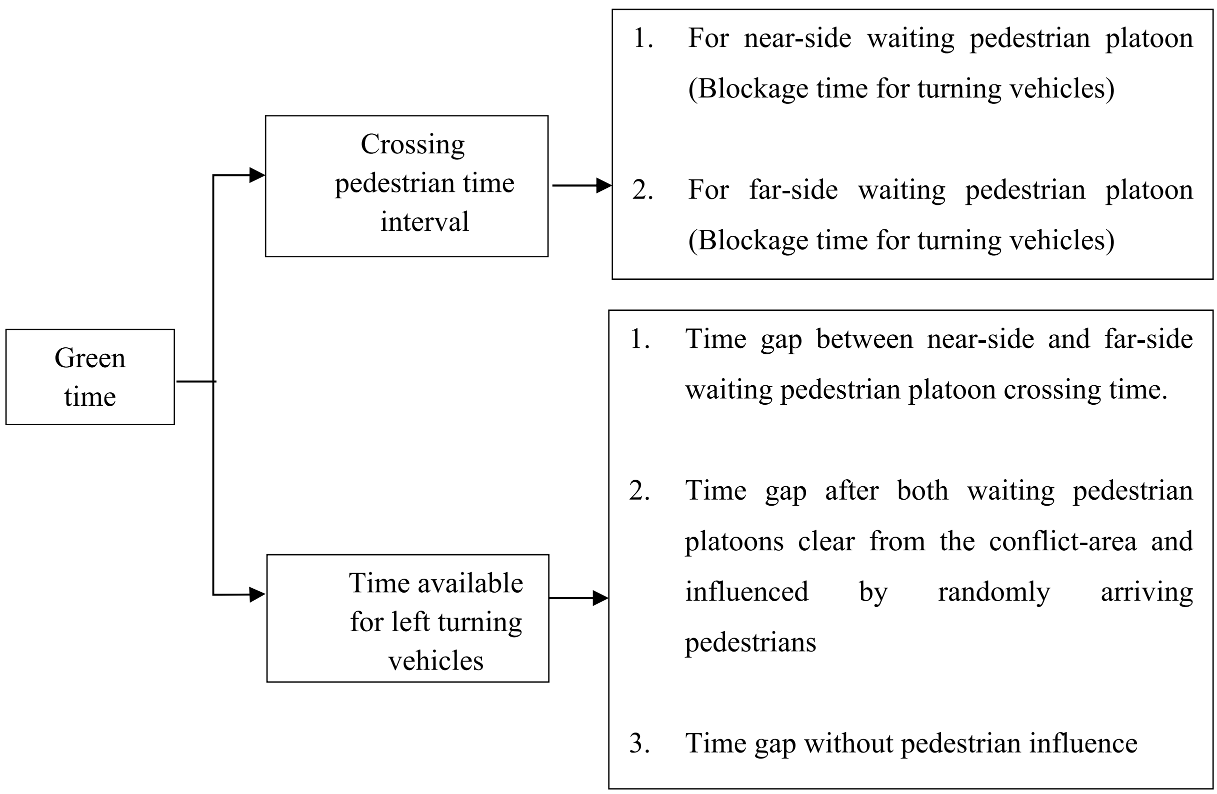


Fig.3.2 Green time intervals

In concurrent signal timing design the pedestrian-vehicle interaction analysis will start from the onset of green and when the first waiting pedestrian from the near-side direction enter inside the conflict area. The longest interaction time interval between crossing pedestrians and left turning vehicles is happening when the platoon of waiting pedestrians crosses the crosswalk, at this time interval left turning vehicles are blocked by crossing pedestrians significantly. Therefore, in this study waiting pedestrians and arriving pedestrians are analyzed separately.

Time thresholds

The time intervals are defined by six time thresholds. These time thresholds can be defined by assuming a group movement of bi-directional waiting pedestrians. In addition, left turning vehicles will pass the conflict area by searching acceptable gaps between arriving pedestrians.

1) t_{n_en}: this is the time threshold when the near-side waiting pedestrians start to enter into the conflict-area at the onset of green. Section *a-a* is the measurement section as shown in **Fig.3.1**. It is influenced by startup lost time and the waiting area of the crosswalk.
2) t_{n_ex}: this is the time threshold when the near-side waiting pedestrian platoon clears from the conflict-area. Section *b-b* is the measurement section as shown in **Fig.3.1.** It is influenced by the near-side waiting pedestrian volume, crosswalk length, crosswalk width, waiting pedestrian composition and opposing pedestrian flow.
3) t_{f_en}: this is the time threshold when the far-side waiting pedestrian platoon enters into the conflict-area. Section *b-b* is the measurement section as shown in **Fig.3.1.** It is influenced by the far-side waiting pedestrian volume, crosswalk length, crosswalk width, waiting pedestrian composition and opposing pedestrian flow.
4) t_{f_ex}: this is the time threshold when the far-side waiting pedestrian platoon clears from the conflict-area. Section *a-a* is the measurement section as shown in **Fig.3.1.** It is influenced by the far-side waiting pedestrian volume, crosswalk length, crosswalk width, signal timing, waiting pedestrian composition and opposing pedestrian flow.
5) ***End of PFG*** (t_{e_PFG}): this is the end time for pedestrian's presence along the crosswalk. Pedestrians have priority to pass the conflict-area within pedestrian green time and they must quickly clear from the crosswalk at the pedestrian flash green (PFG) time if they are still on the crosswalk. After the end of PFG left turning vehicles will have a chance to pass the conflict-area without pedestrian influence.
6) ***End of Yellow time*** (t_{e_Y}): left turning vehicles that already passed the stop line must clear out from the conflict-area before the end of yellow time.

The six time thresholds can divide the green time into five time intervals.

Time interval 1 (t_1): this is the necessary time for near-side waiting pedestrians to clear from the conflict-area. In this time interval it is difficult for left turning vehicles to pass the conflict-area since the gap between waiting pedestrians will be short.

Therefore, this time interval is the unused green time for left turning vehicles due to the presence of near-side waiting pedestrian's platoon. Time interval 1 can be estimated using equation 3.1.

$$t_1 = t_{n\_ex} - t_{n\_en} \tag{3.1}$$

Time interval 2 **(t_2)**: this is the time interval between the near-side waiting pedestrian platoon clear the conflict-area and when the far-side waiting pedestrian platoon enter into the conflict-area, which is estimated by equation 3.2.

$$t_2 = \begin{pmatrix} t_{f\_en} - t_{n\_ex} & if & t_{f\_en} > t_{n\_ex} \\ 0 & if & t_{f\_en} \le t_{n\_ex} \end{pmatrix} \tag{3.2}$$

Time interval 3 **(t_3)**: this is the necessary time for far-side waiting pedestrians to clear from the conflict-area. In this time interval it is difficult for left turning vehicles to pass the conflict-area since the gap between waiting pedestrians will be short. Therefore, this time interval is the unused green time for left turning vehicles due to the presence of far-side waiting pedestrian's platoon. Time interval 3 can be estimated using equation 3.3.

$$t_3 = t_{f\_ex} - t_{f\_en} \tag{3.3}$$

Time interval 4 **(t_4):** this time interval starts after the clearance of far-side waiting pedestrian platoon and ends at the end of PFG. In this time interval left turning vehicles discharge flow by searching acceptable gaps between arriving pedestrians coming from both directions of the crosswalk.

$$t_4 = PG + PFG - t_1 - t_2 - t_3 \tag{3.4}$$

Time interval 5 **(t_5)**: this is the left turning vehicles discharge flow interval without the influence of crossing pedestrians.

$$t_5 = G + Y - t_1 - t_2 - t_3 - t_4 \tag{3.5}$$

The arrangements of the time thresholds are highly depending on the pedestrian demand and crosswalk length. **Fig.3.3** shows basic bi-directional pedestrian flow condition by using one observed cycle, **Fig.3.3** illustrate how the waiting pedestrian presence influencing time interval and remaining discharge flow time of left turning vehicles is defined based on the observation of bi-directional pedestrian flows. The two basic scenarios based on the condition of pedestrian demand and crosswalk length are presented in **Fig. 3.4** and **Fig 3.5**. **Fig 3.4** shows the basic scenario (scenario-1) for longer or lower pedestrian demand crosswalk. While **Fig.3.5** shows the basic scenario (scenario-2) for shorter or higher pedestrian demand crosswalk.

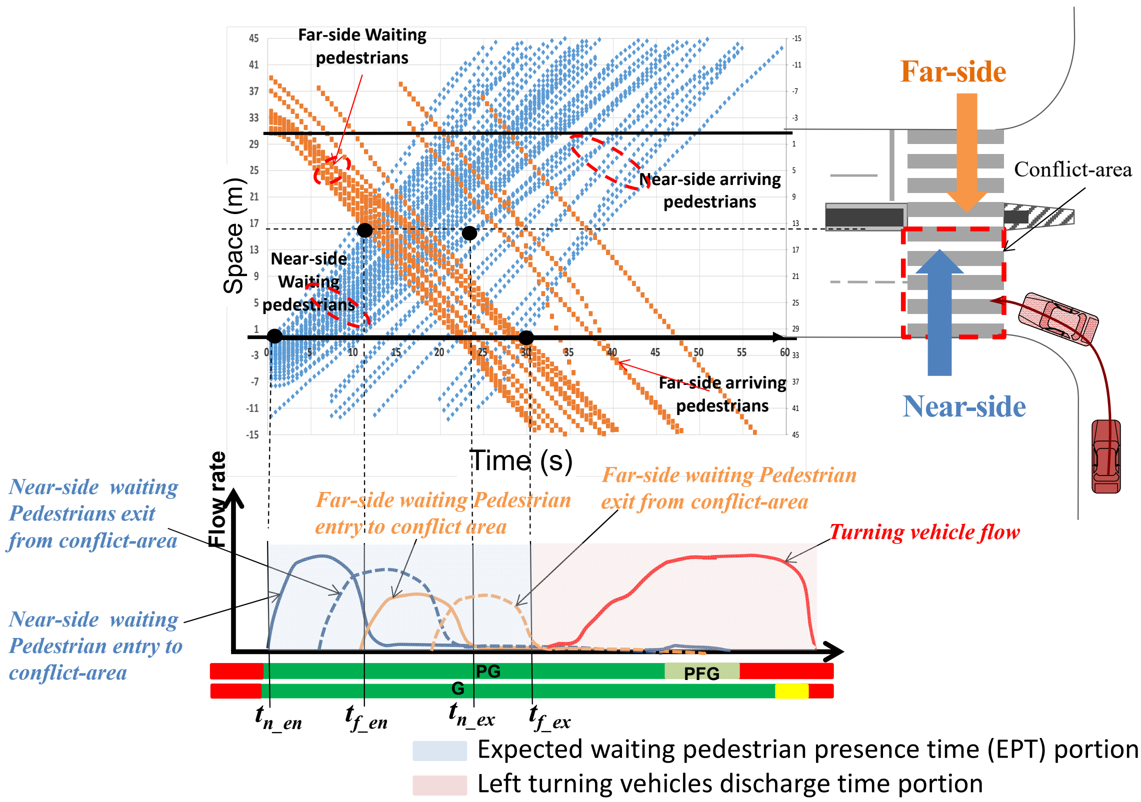


Fig. 3.3 Bi-directional pedestrian flow conceptual illustration

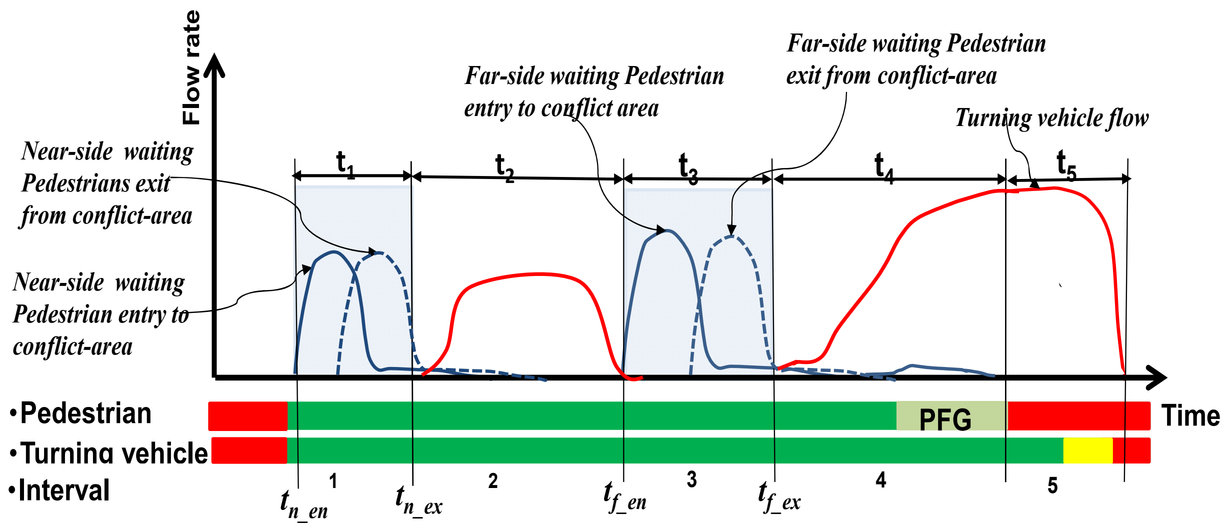


Fig.3.4 Time interval thresholds of Scenario-1

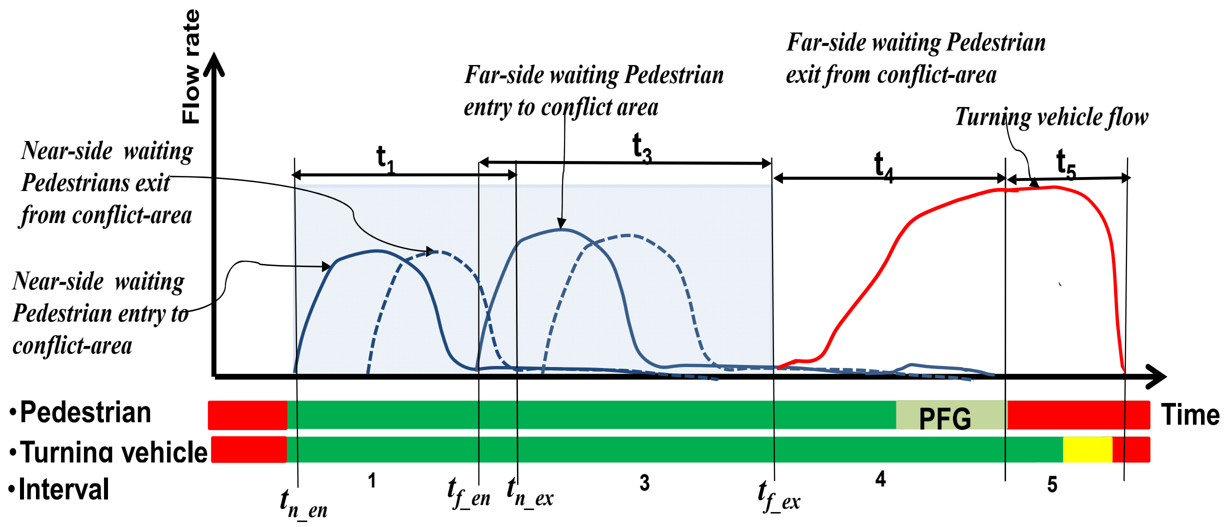


Fig.3.5 Time interval thresholds of Scenario-2

Based on the bi-directional pedestrian flow, left turning vehicles flow and crosswalk layout condition different cases may happen for scenario 1 and 2 for each time intervals within the assigned green time.

Fig.3.6 shows one hypothetical pedestrian flow example case when the crosswalk is long, the pedestrian flow from near-side direction has lower pedestrian demand and the opposite far-side has higher pedestrian demand.

For this case, the possibility to have a t_2 interval is high since the near-side waiting pedestrians may clear from the conflict-area before the arrival of the far-side waiting pedestrians. It is expected that the far-side pedestrian flow may influence the near-side waiting pedestrian flow, so that near side pedestrians may have significantly variable crossing speed along the crosswalk.

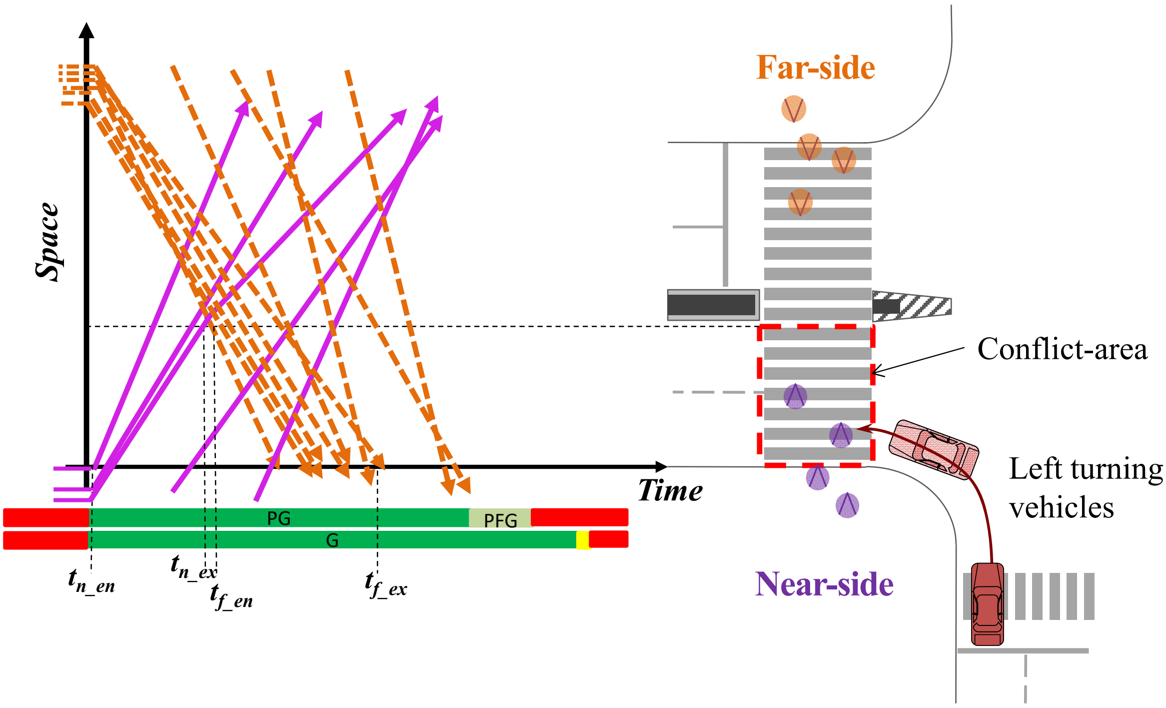


Fig.3.6 Conceptual bi-directional pedestrian flow example case for scenario 1

Fig.3.7 shows one hypothetical pedestrian flow example case for scenario 2 when the crosswalk is short, the pedestrian flow from near-side direction has higher pedestrian demand and the opposite far-side flow has also higher pedestrian demand.

For this case the bi-directional pedestrian flow interaction is higher, so that that pedestrians crossing speed may significantly have affected. The possibility to observe t_2 interval is low therefore left turning vehicles have small available green time.t_1 and t_3 may have long time interval due to the bi-directional higher pedestrian demand. The influence of crossing pedestrians on left turning vehicles is higher for this case compared with other cases.

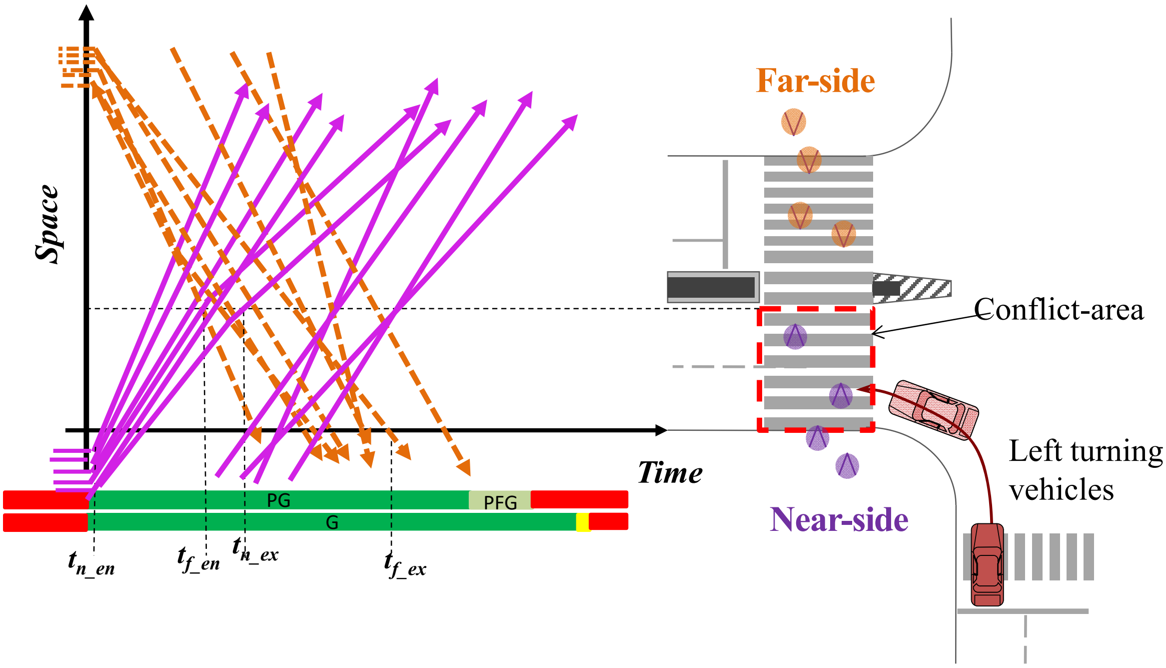


Fig.3.7 Conceptual bi-directional pedestrian flow example case for scenario 2

In general, the time intervals are highly related with the bi-directional pedestrian's flows and crosswalk layout. Therefore, the crossing pedestrian's characteristics must be empirically observed and their position distribution along the crosswalk within the assigned green time must be generalized. The influencing factors of crossing pedestrian's behavior must be also investigated to understand the significant crossing speed variations, consequently variation in crossing time. Therefore, in chapter 4 the pedestrian characteristics will be discussed in detail.

3.4 Left-turning vehicles movement

The movement of left turning vehicles under concurrent signal phasing is highly influenced by the presence of crossing pedestrians on the conflict-area. As it is discussed in the above section, the pedestrian flow can be categorized in to two types based on the arrival time to the crosswalk; as waiting pedestrians and arriving pedestrians. The interaction situation of left turning vehicles with waiting pedestrians and arriving pedestrians is different. When left turning vehicles arrive on the conflict-area while waiting pedestrians are crossing, it is difficult to find a gap and pass the conflict-area. Therefore, the assumption is the bi-directional waiting pedestrians flow will block the movement of left turning vehicles, because most of the times waiting pedestrians are crossing in a platoon starting from the waiting area. However, after the bi-directional waiting pedestrian platoon clears from the conflict-area randomly arriving pedestrians may enter in to the conflict-area till the end of PG, at this time interval the possibility for left turning vehicles to pass the conflict area is high by searching acceptable gaps between arriving pedestrians.

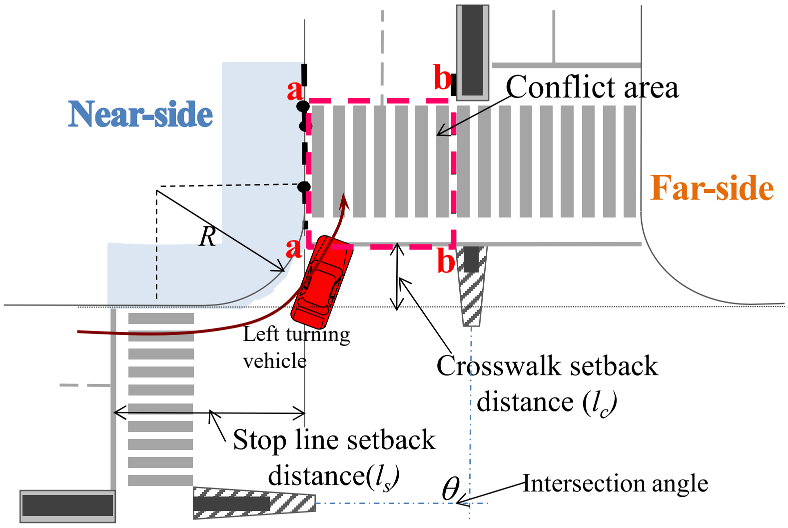


Fig.3.8 intersection layout

In addition to the presence of crossing pedestrians the geometric layout of the signalized crosswalk may influence the movement of the left turning vehicles. As it is shown in **Fig.3.8,** the geometric layout of the intersection can be explained by turning radius, intersection angle, stop line setback distance, crosswalk length and crosswalk setback distance. These elements are variable for different intersections according to the geometric design of the intersection and the proposed manuals. The movement of left turning vehicles at the signalized intersection may significantly influenced by the assigned layouts of those elements. When the setback distance of the stop line is far from the conflict-area then the arrival time of left turning vehicles will be long, due to that some portion of the green time will be unused. In addition, the clearance time of left turning vehicles after the end of green time will be long due to the availability of long waiting space for left turning vehicles.

The left turning radius or the intersection angle may also highly influence the turning speed of left tuning vehicles. If the intersection angle is obtuse and have gentle turning radius it will give chance for left turning vehicles to drive with higher speed. Consequently, they may attempt to accept short available gaps between crossing pedestrians.

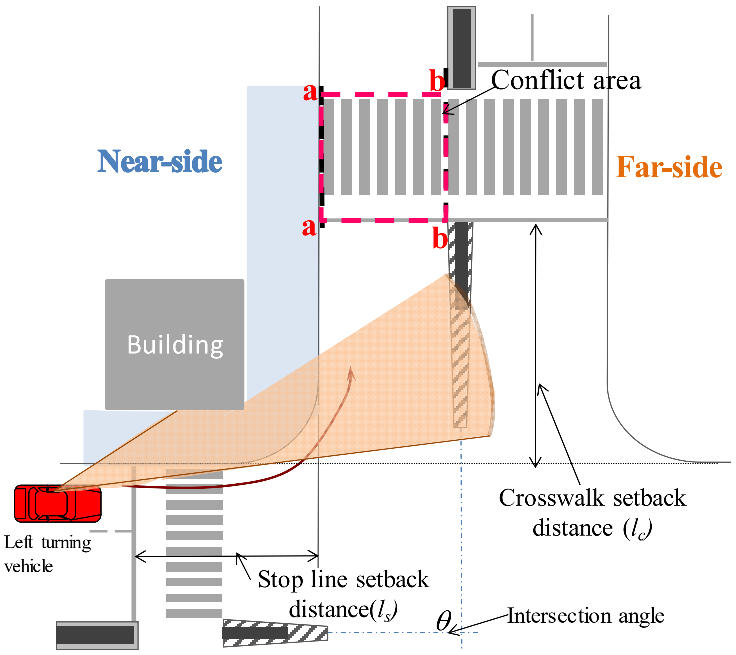


Fig 3.9 Left turning vehicle sight distance with long crosswalk setback distance

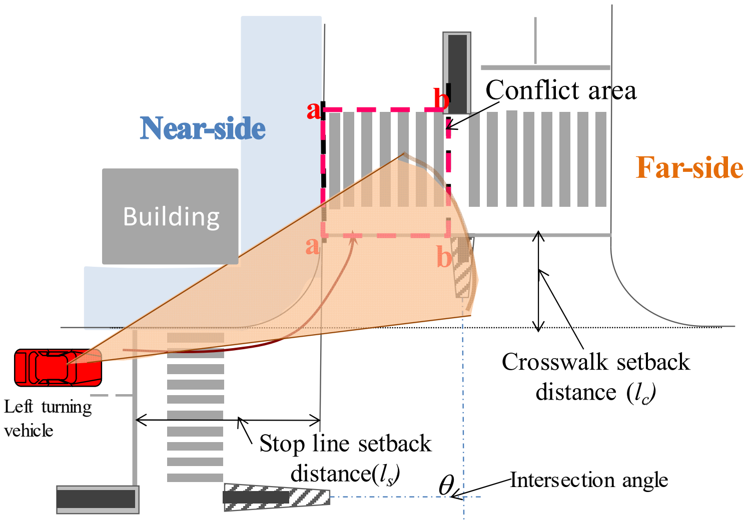


Fig 3.10 Left turning vehicle sight distance with short crosswalk setback distance

On the other hand, the setback distance of the subject crosswalk has significant effect on the left turning vehicles movement. The influence may be related with the visibility of crossing pedestrians and acceleration/deceleration distance of left turning vehicles. **Fig 3.9** and **Fig 3.10** show the left turning vehicle sight distance with long and short crosswalk setback distances, respectively. When the subject crosswalk is near to the intersection left turning vehicles may clearly notice crossing pedestrians even before they arrive at the conflict area and they can adjust their turning speed accordingly. However, the situation for the case of safety analysis may be different from the capacity analysis concept as it is discussed by (Georges J., 2012). The balanced position between the stop line and crosswalk setback distance is necessary to minimize the influence of geometric layout of the crosswalk on the left turning vehicles movement.

Hypothetical left turning vehicles discharge flow condition within green interval

For the hypothesized time intervals discussed in the above section, the left turning vehicles have different discharge flow characteristics. **Fig.3.11** shows the basic scenario (scenario-1) for longer or lower pedestrian demand crosswalk. While **Fig.3.12** shows the basic scenario (scenario-2) for shorter or higher pedestrian demand crosswalk.

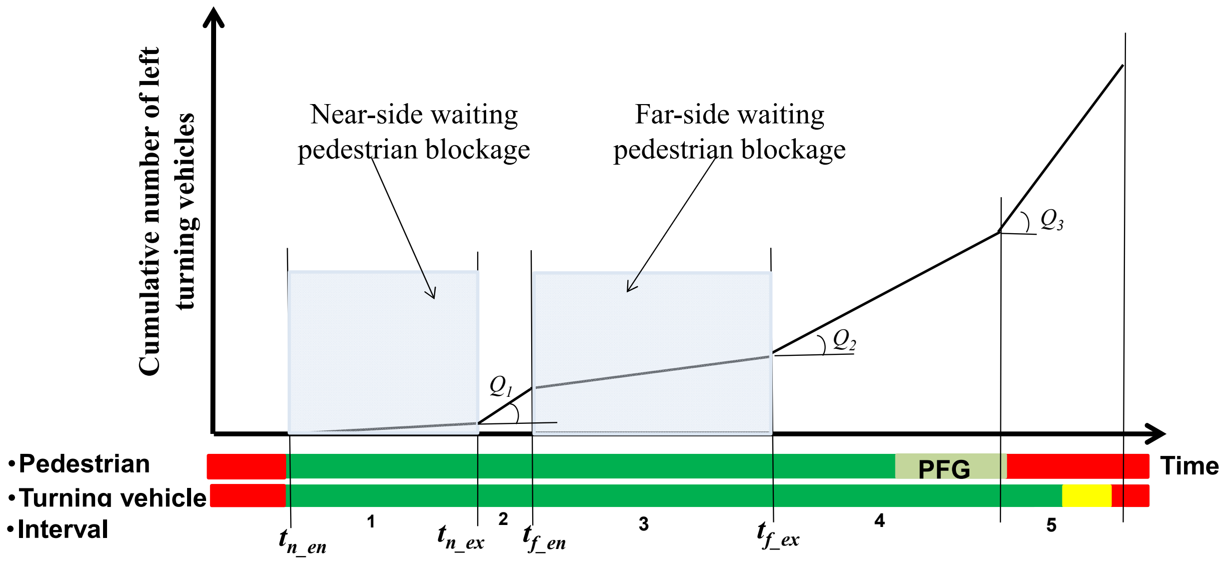


Fig.3.11 Hypothetical left turning vehicles discharge flow for scenario-1

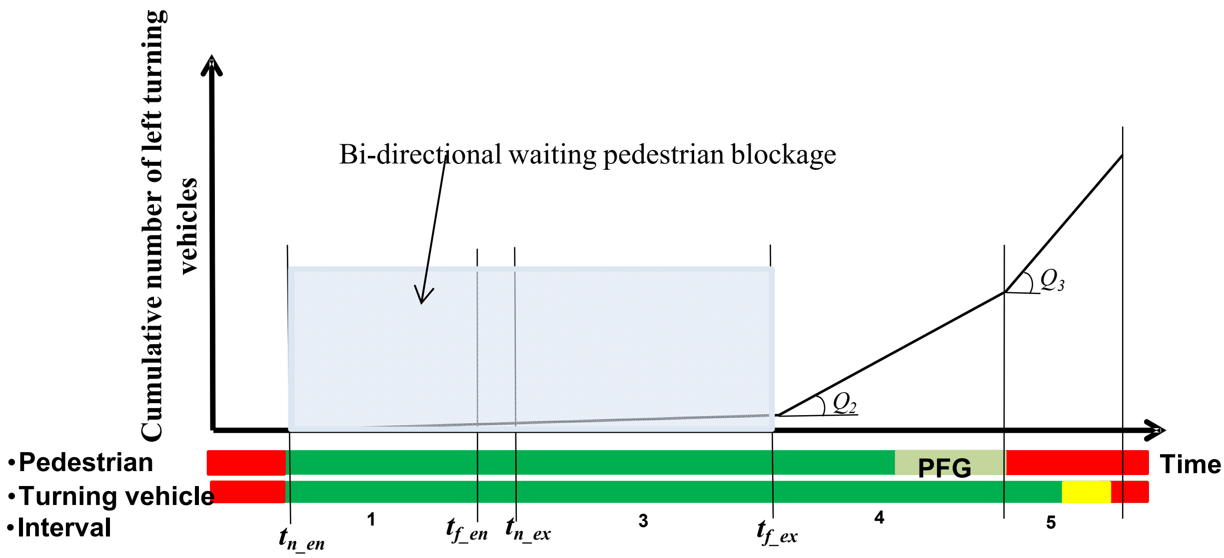


Fig.3.12 Hypothetical left turning vehicles discharge flow for scenario-2

For scenario-1 case we may have three different discharge flows at three time intervals, the first discharge flow (Q_1) is located between the bi-directional pedestrian flows (near-side and far-side flows). For long or lower pedestrian demand crosswalk since the arrival time of the far-side first waiting pedestrian to the conflict-area is long or the near-side waiting pedestrians clear from the conflict-area quickly, left turning vehicles may have a chance to pass the conflict-area immediately after the clearance of the near-side waiting pedestrian's platoon if there are no arriving pedestrians. The second discharge flow (Q_2) is located after the clearance of the far-side waiting pedestrians and the left turning vehicles attempt to pass the conflict-area by searching acceptable gaps between arriving pedestrians and finally the left turning vehicles will discharge without the influence of crossing pedestrians with Q_3 flow rate. For scenario-2 case also the same discharge flow characteristics can be observed except the possibility of observing Q_1 will be zero, because of the higher pedestrian's demand or short crosswalk length the bidirectional pedestrian flow will merge together inside the conflict-area.

For both scenarios the left turning vehicles movement will be blocked by waiting pedestrian platoon and the magnitude of blockage effect is highly dependent on the pedestrian demand. Discharge flow of left turning vehicles is highly influenced by arriving pedestrians and the geometric layout of the crosswalk.

3.5 Conceptual framework of capacity estimation

Fig 3.13 shows the conceptual framework for the left turn lane capacity estimation procedure. Mainly there are two paths in the estimation procedure; the first one is observation and generalization of crossing pedestrians by separating the waiting and arriving pedestrians independently. The second path is observation and modeling of the left turning vehicles discharge flow rate by considering all possible influencing factors. Finally, the results of the two paths are combined to generalize the left turn lane capacity under the influence of crossing pedestrians and different crosswalk layout conditions. i.e. the proposed exclusive left turn lane capacity equation has two input models, pedestrian presence-time model (EPT) and left turning vehicles discharge flow rate model (Q).

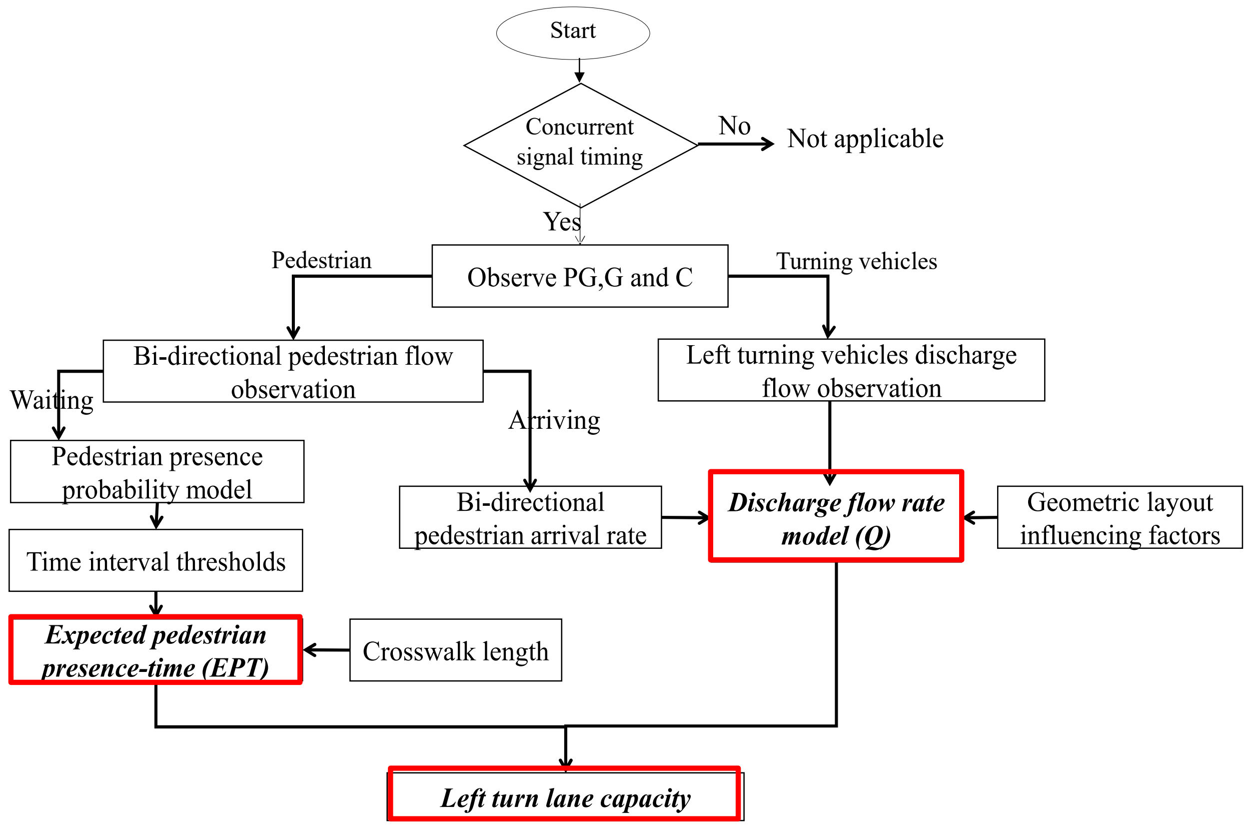


Fig 3.13 Conceptual framework of capacity estimation

3.6 Hypothetical left-turn lane capacity estimation equation

Left turning vehicles pass the conflict-area based on the condition of crossing pedestrians on the crosswalk. Immediately at the onset of green the presence probability of near-side waiting pedestrians on the conflict-area is high, since near-side waiting pedestrians' waiting area is near to the conflict-area compared with left turning vehicles and far-side waiting pedestrians. After that, far-side waiting pedestrians join the near-side pedestrians or arrive to the conflict area after near-side waiting pedestrians clear from the conflict-area. Meanwhile, arriving pedestrians may presence on the conflict area any time after the onset of green. When crossing pedestrians present on the conflict-area left turning vehicles must give priority, so that they may have delay time due to the presence of crossing pedestrians. Therefore, the maximum number of left turning vehicles that can pass the conflict-area within the assigned green time is highly influenced by the interaction with crossing pedestrians. As it is discussed in the above sections, the characteristics of waiting pedestrian flow and the interaction of arriving pedestrians on left turning vehicles can be empirically observed and modeled to capture the influence of crossing pedestrians on left turning vehicle movements. The necessary time for the bi-directional waiting pedestrians to cross the conflict-area (EPT) can be estimated based on the crossing pedestrian's characteristics and it will be discussed in chapter 4. To estimate left turn lane capacity first, the available green time for left turning vehicles must be computed by deducting the unused time from the assigned vehicle green time, then the available time will be multiplied by the average discharge flow rate of left turning vehicles under the condition of arriving pedestrians and geometric layout of the crosswalk. In the estimation of the discharge flow rate of left turning vehicles the influence of basic geometric layout elements of the intersection must be investigated. After we understand the crossing pedestrian's characteristics and the discharge flow rate as we hypothesized in the above sections, by considering the pedestrian's behavior and geometric layout of the crosswalk, the left turn lane capacity can be estimated using Equation (3.6).

$$c = \frac{((G - EPT) * Q(X_p, X_c) + (t_5 * Q(0, X_c))}{C} \tag{3.6}$$

Where, c is capacity of left turn lane (veh/hr), G is left turning vehicle green time (s), EPT is expected waiting pedestrian-presence time (s), Q is discharge flow rate (veh/hr), Xp is the influence of pedestrians, Xc is the influence of crosswalk layout and C is cycle length (s), t_5 is time interval without the influence of crossing pedestrians (s).

Therefore, to compute the left turn lane capacity based on the above proposed method, the basic parameters of the equation will be analyzed in the following chapters. By empirical observation of signalized crosswalks, the bi-directional pedestrian flow and left turning vehicles discharge flow rate will be generalized.

3.7 Summary

In this chapter the bi-directional pedestrian flow and the left turning vehicles discharge flow rate are hypothesized within the assigned green time. In general, the green time can be divided into five time intervals based on the basic bi-directional pedestrian flow characteristics. The five intervals have six critical time thresholds which determine the boundary of the time intervals. Based on the bi-directional pedestrian demand condition and the geometric layout of the crosswalk two scenarios can be hypothesized. In scenario-1 for long or lower demand crosswalks, the possibility to have a t_2 interval is high since the near-side waiting pedestrians may clear from the conflict-area before the arrival of the far-side waiting pedestrians. Furthermore, in this scenario we may have three different discharge flows at three time intervals, the first discharge flow (Q_1) is located between the bi-directional pedestrian flows (near-side and far-side flows). For long or lower pedestrian demand crosswalk since the arrival time of the far-side first waiting pedestrian to the conflict-area is long or the near-side waiting pedestrians clear from the conflict-area quickly, left turning vehicles may have a chance to pass the conflict-area immediately after the clearance of the near-side waiting pedestrian's platoon if there are no arriving pedestrians. The second discharge flow (Q_2) is located after the clearance of the far-side waiting pedestrians and the left turning vehicles attempt to pass the conflict-area by searching acceptable gaps between arriving pedestrians and finally the left turning vehicles will discharge without the influence of crossing pedestrians with Q_3 flow rate. For scenario-2, the bi-directional pedestrian flow interaction is higher, therefore the possibility to observe t_2 interval is low and left turning vehicles have small available green time, consequently the discharge flow rate in scenario-2 is lower compared with sceanro-1 cases.

In general, the conceptual framework of the capacity estimation procedure and the left turn lane capacity estimation equation is proposed by considering the influence of the bi-directional pedestrian flow along the crosswalk and the geometric layout of the crosswalk.

Chapter 4

Crossing Pedestrian Presence-time

4.1 Overview

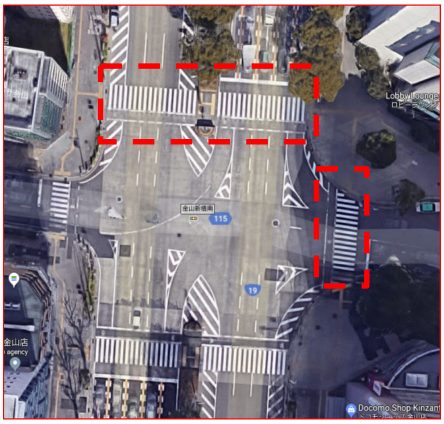
Understanding crossing pedestrian's characteristics is a necessary step to study their influence on left turning movement. The movement of crossing pedestrians along the crosswalk significantly varies depending on different factors, like crosswalk geometry, signal timing design and pedestrian interactions. Because of the various influencing factors, the crossing time of pedestrians may significantly affect, consequently the position of pedestrians at elapsed green time t will be different. Therefore, in this chapter the crossing pedestrian's characteristics when they pass the crosswalk within the assigned green time is observed, and then the pedestrian position distribution along the crosswalk is generalized to further apply for the understanding of left truing vehicles influence.

4.2 Observed intersections and data extraction

For the pedestrian position distribution analysis and modelling at the crosswalk, signalized intersections located in Nagoya, Japan are selected. The intersections have different pedestrian demand situation and crosswalk geometric characteristics. The geometric condition, traffic condition and signal settings of the selected signalized crosswalks are summarized in **Table 4.1**. and the layout of the observed crosswalks are shown in **Fig.4.1**.

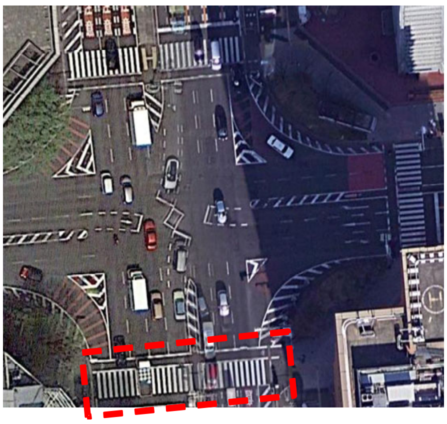
Table 4.1 Geometric characteristics and traffic conditions of observed sites.

Intersection name in Fig.4.1	Subject crosswalk	Crosswalk geometry		Signal phase		Waiting pedestrian volume (ped/cycle)				Number of sample cycles
		Length (m)	Width (m)	PG (s)	Cycle length (s)	Near-side	Near-side average	Far-side	Far-side average	
(a) Kanayama	East	16	6	48-61	148-174	0-12	5	0-6	3	40
	North	36	6	36-48	148-174	0-13	4	0-10	5	60
(b) Ueda	South	21	5	38-47	144-176	0-4	2	0-5	2	63
(c) Fushimi	South	35	6	39-42	159-161	0-10	4	0-8	5	45
(d) Sasashima	West	31	10	40	160	16-62	37	10-30	13	18
(e) Otsu-dori	West	34	6	37	160	0-12	4	1-10	3	30
(f) Nishiosu	North	32	6	38	160	0-4	3	0-3	2	22

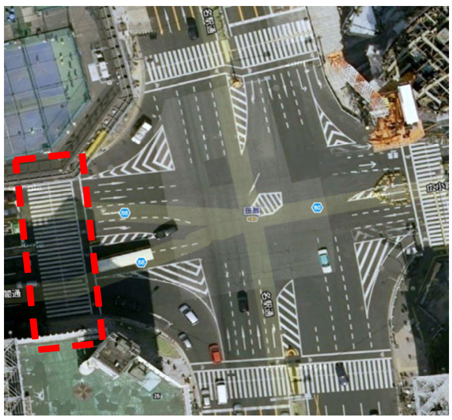


(a) Kanayama

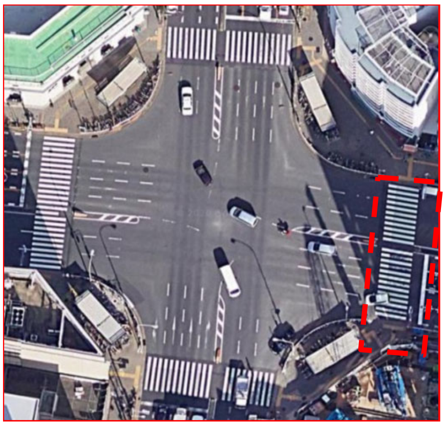
(b) Ueda



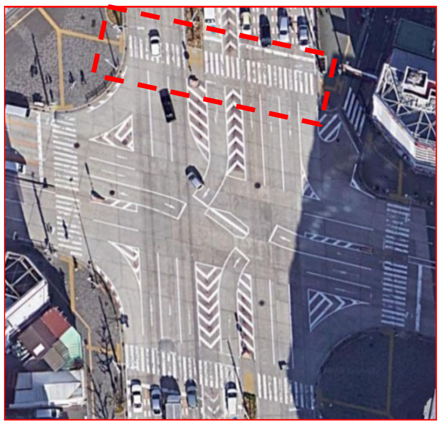
(c) Fushimi



d) Sasashima



(e) Otsu-dori



(f) Nishiosu

Fig.4.1 Layout of observed crosswalks

Data extraction

The positions of pedestrians at every 5s are manually extracted from observation videos by using the image processing system Traffic Analyser (Suzuki and Nakamura1, 2006) then the coordinates in these images are converted to global coordinate through the projective transformation. Kalman smoothing method is applied to estimate trajectories of pedestrians at each time interval. Then pedestrian presence probability along the crosswalk is extracted at different elapsed time thresholds.

A coordinate transformation of pedestrian position is done referring to the edge of the crosswalk. The horizontal axis x is defined in parallel to the edge of bicycle crossing path and the vertical axis y is perpendicular to this line as shown in **Fig.4.2**. This pedestrian position is used to calculate the pedestrian presence probability within a PG time. Here in the analysis, pedestrian presence probability at time t and position x is defined as the number of pedestrians present at time t and position x divided by the total waiting pedestrian number of the cycle.

4.3 Empirical observation of pedestrian position distribution

Examining pedestrian timing and position is very important to understand pedestrian-vehicle interaction and analyse their influence on turning vehicles movement. (Zhang and Nakamura ,2017) analysed and modelled pedestrian presence probability along several signalized crosswalks by considering signal timing, pedestrian arrival rate and crosswalk length as influencing factors. They found that pedestrian position distribution along the crosswalk dispersed when crosswalk length and elapsed time of PG increase. Moreover, it was found that the distributions will shift to the moving direction slowly and their variations become larger when pedestrian arrival rate increases. However, they analysed the sites with relatively lower pedestrian traffic demand. Moreover, interaction with opposing pedestrian flow has not been investigated by them. This study followed their procedures about the pedestrian position observation and extraction procedures by including observation of additional higher pedestrian demand crosswalks.

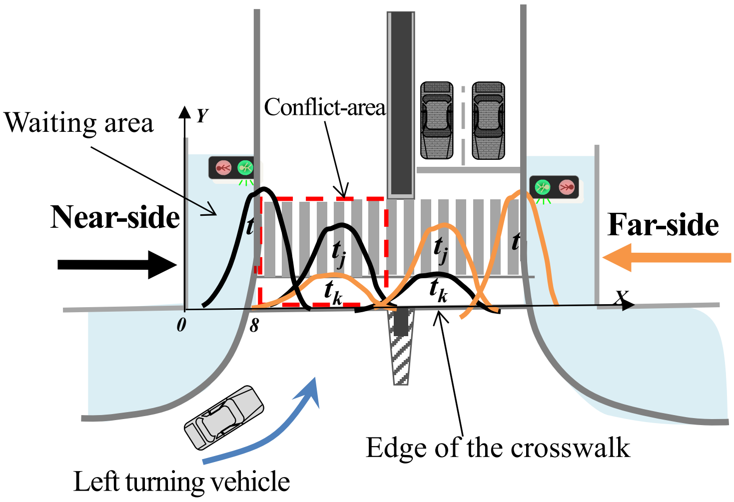


Fig.4.2 Bi-directional crossing pedestrians position distribution by time.

A coordinate transformation of pedestrian position is done referring to the edge of crosswalk. The horizontal axis (x) is parallel to the edge of bicycle crossing path, and then the vertical axis (y) is perpendicular to that as shown in **Fig.4.2**. Since the distance on horizontal axis (x) is more important for analysing the pedestrian-vehicle interaction, only the distance on x axis will be considered and all the pedestrians are assumed to walk on this axis. In order to consider the position where pedestrians wait on the sidewalk for pedestrian green phase, a space between the beginning of crosswalk and 8m upstream from that is defined as waiting area as indicated in **Fig.4.2**. Consequently, the origin of the horizontal axis is defined at the location 8m upstream from the beginning of near-side of the crosswalk. The bi-directional pedestrian flow from the two sides of the crosswalk is observed by time as the conceptual distribution shown in **Fig.4.2**.

At the onset of pedestrian green (PG), waiting pedestrians from both edges of a crosswalk start walking. After PG elapsed, pedestrian's position along the crosswalk gradually changes because of a variation in walking speeds of individual pedestrians. In addition, there is an interaction between pedestrians crossing from two sides of the crosswalk. The significance of interaction may depend on the number of pedestrians from both sides of the crosswalk. This spreading and interaction situation is more significant for the waiting pedestrian platoon compared with arriving pedestrians, since most of the time arriving pedestrians enter the crosswalk individually while waiting pedestrians enter the crosswalk in a group. Therefore, in this study the influence of waiting pedestrians and arriving pedestrians on left turning vehicles movement is investigated separately.

4.3.1 Observed pedestrian flow conditions

The observed cycles can be categorized into three groups based on Equation (4.1) for the comparison analysis.

$$p = \frac{q_s - q_o}{q_s} \tag{4.1}$$

Where, p is proportion of demand difference, q_s is subject direction pedestrian demand and q_o is opposing direction pedestrian demand.

From empirical data observation, the influential threshold for categorization of the groups is 20 percent demand difference between the two directional pedestrian flows.

- Group H-L: When higher pedestrian demands in the subject direction interacting with lower pedestrian demand of the opposing direction. $p > 0.2$
- Group L-H: When lower pedestrian demand in the subject direction interacting with higher pedestrian demand of the opposing direction. $p < -0.2$
- Group B-B: Balanced pedestrian demand in both directions. $-0.2 < p < 0.2$

Fig.4.3, **Fig. 4.4** and **Fig.4.5** show examples of observed cumulative pedestrian presence probability distributions for waiting pedestrians along the crosswalk, for H-L, L-H and B-B pedestrian flow categories of Sasashima and Kanayama east crosswalks. The observed pedestrian presence probability distributions are plotted by aggregating observed samples of each group. When t increases, pedestrian presence probability distributions shift to the moving direction. The slopes of the distributions become wider due to the variation in pedestrians' crossing speeds. For both all cases, the distributions of Kanayama east crosswalk are positioned on the right side of Sasashima distributions. This is due to the lower number of pedestrian demand in Kanayama east crosswalk; pedestrian can quickly cross without influence of surrounding pedestrians. However, for the case of Sasashima crosswalk, due to the high pedestrian demand it is difficult for pedestrians to cross with their desire speed.

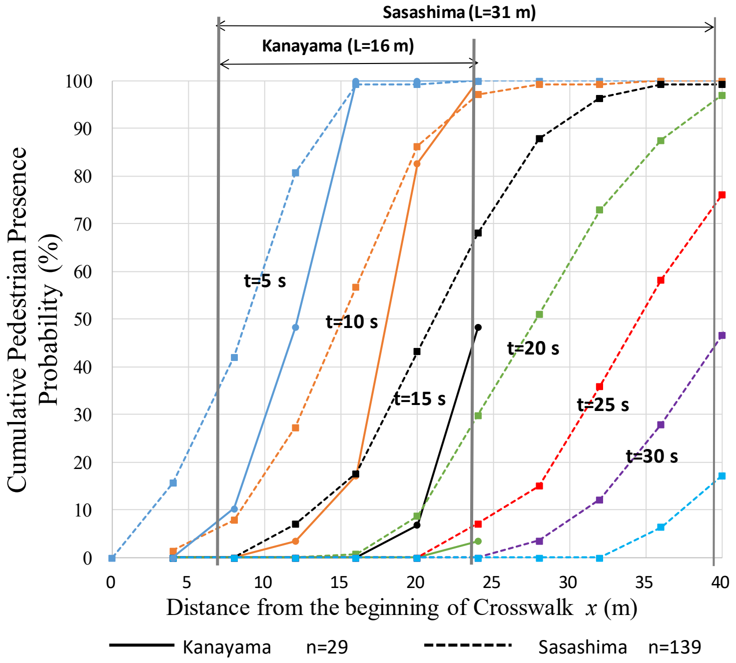


Fig.4.3 Observed H-L pedestrian presence probability distribution for Sasashima and Kanayama east crosswalks

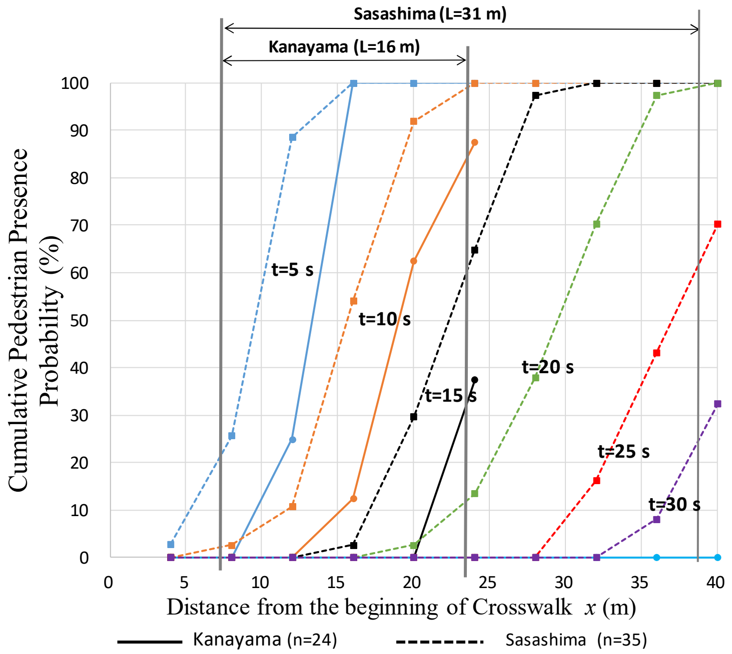


Fig.4.4 Observed L-H pedestrian presence probability distribution for Sasashima and Kanayama east crosswalks

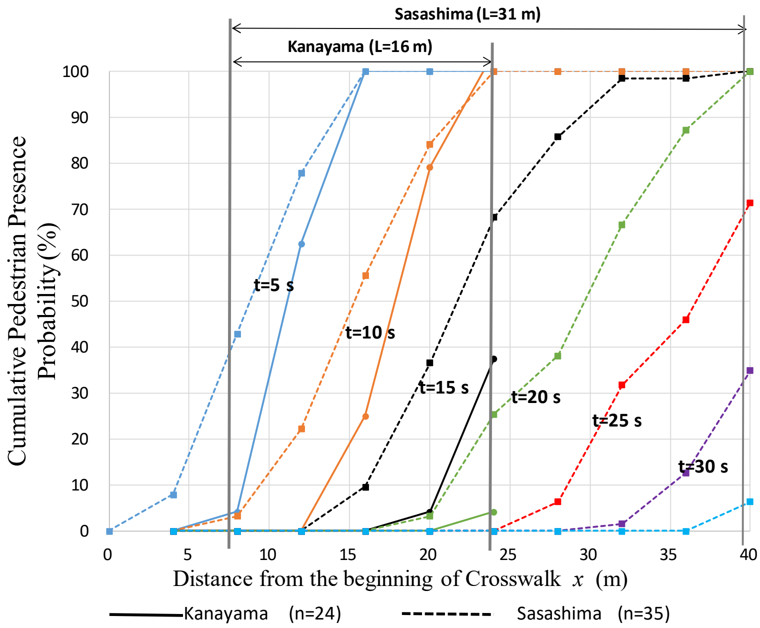


Fig.4.5 Observed B-B pedestrian presence probability distribution for Sasashima and Kanayama east crosswalks

Pedestrian flow at the crosswalk is a bi-directional flow. Therefore, pedestrian presence probability along the crosswalk is affected by the interaction with not only the subject pedestrian flow but also the opposing pedestrian flow. Comparing three groups of pedestrian flow, it can be found that the distributions of H-L group move slower than L-H group. It is because when the number of pedestrian in subject flow is high, pedestrians cannot cross in the desired speed. The variations become larger when subject pedestrian flow is high, and when opposing pedestrian flow become higher, the variations become lower.

Three possible influencing factors can be noted from the above observations: reduction in crossing speed due to the high pedestrian demand condition, the effect of crosswalk length and the effect of elapsed green time

4.3.2 Bi-directional pedestrian flow interaction

Ccrossing pedestrian position distribution along the crosswalk is affected by the interaction with the opposing pedestrian flow. The effect of bi-directional pedestrian flow interaction on pedestrian presence probability distribution curve may be significant for high demand pedestrian flow crosswalks. The difference in the pedestrian presence probability distribution before and after the interaction occurrence between the subject and opposing flows can be quantitatively investigated using standard deviation of the distribution curve. When there is some influencing interaction between the two flows (subject direction and opposing direction flows), the standard deviation of pedestrian presence distribution curve at the interaction time will be shorten. This is due to the situation that, the leading pedestrians in the subject direction may reduce their walking speed when they face counter flow with high demand, while the following pedestrians keep their walking speed.

One sample cycle of this condition for Sasashima crosswalk bi-directional flow is shown in **Fig. 4.6**, here in the figure the bi-directional flow interacts between the elapsed time of 15s and 25s. A higher demand near-side pedestrian flow interacts with the opposing lower demand far-side pedestrian flow. The standard deviation of the pedestrian presence distribution curve for the near-side pedestrian flow at an elapsed time of 15s, 20s and 25s are 5.2m, 4.5m, and 4.4m, respectively. For the opposite far-side flow standard deviation at an elapsed time of 15s, 20s and 25s are 1.25m, 1.1m, and 3m, respectively. Therefore, as it is observed from this computation, the changing rate of the standard deviation for far-side pedestrian flow pedestrian presence distribution curve is less than that of near-side pedestrian flow pedestrian presence distribution curve. This is one implication of opposing direction higher-pedestrian demand effect on the pedestrian presence probability distribution of lower-pedestrian demand direction. Therefore, it is necessary to consider bi-directional pedestrian flow in the estimation of pedestrian presence probability distribution.

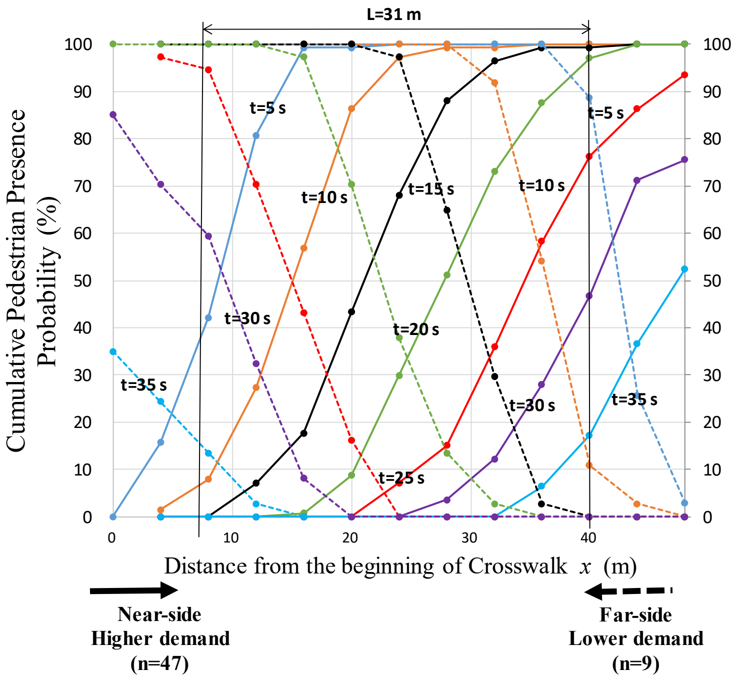


Fig. 4.6 Bi-directional pedestrian presence probability distributions

Additionally, the difference in the pedestrian presence probability distribution before and after the interaction occurrence between the subject and opposing flows is quantitatively investigated by using Equation (4.2).

$$X(t) = X_{85}(t) - X_{15}(t) \tag{4.2}$$

Where, $X_{85}(t)$ and $X_{15}(t)$ [m] are the positions of the 85^{th} and the 15^{th} percentile values of the cumulative pedestrian presence probability distribution curve at an elapsed time t, respectively. $X(t)$ is the difference of these values at time t.

When there are some influencing interactions between the two flows, the value of $X(t)$ at the interaction time will be shorten. This is due to the situation that, the leading pedestrians in the subject direction may reduce their walking speed when they face counter flow with high demand, while the following pedestrians keep their walking speed.

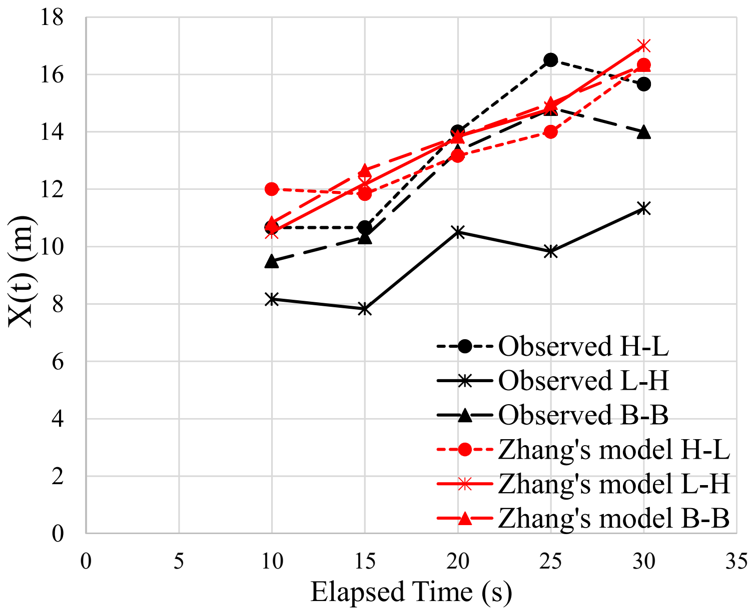


Fig. 4.7 Horizontal position difference of pedestrian presence probability distribution

Fig.4.7 shows a trend of $X(t)$ with an increasing elapsed time for the three groups (H-L, L-H and B-B) by aggregating samples observed in all(three) cycles of each group. For all groups, $X(t)$ is increasing with increase in elapsed time, which shows there is pedestrian presence probability dispersion when elapsed time increases. However, the increasing rate of L-H case pedestrian flow is lower than other groups. This implies that when the bi-directional flow is lower pedestrian demand interacting with higher pedestrian demand, then the lower pedestrian demand direction presence probability distribution would be compacted due to interaction influence.

For Zhang's model, the value of $X(t)$ is increasing with increase in elapsed time. However, the values of $X(t)$ for the model estimate of L-H cases pedestrian presence probability distribution are greater than that of the observed pedestrian presence probability distribution. Since L-H cases are the interaction of lower pedestrian demand with higher pedestrian demand, the result implies, the impact of opposing higher pedestrian demand on the pedestrian presence probability distribution of lower pedestrian demand direction is not considered significantly in Zhang`s model.

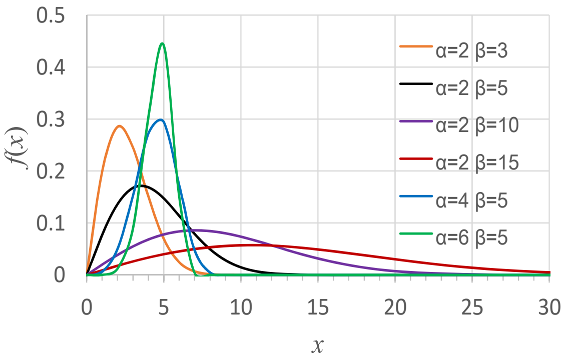
4.4 Pedestrian presence probability model

Pedestrian presence probability at time *t* and position *x* is defined as the number of pedestrians present at time *t* and position *x* divided by the total waiting pedestrian number of the cycle. (Zhang and Nakamura, 2017) observed and modelled time-dependent pedestrian presence probability along the crosswalk as a function of time. From empirical data, the frequency distribution of pedestrians' presence along the crosswalk is analysed by observing pedestrian positions at time *t* then pedestrian presence probabilities are modelled by using the probability density distribution function of Weibull distribution as shown in Equation (4.3).

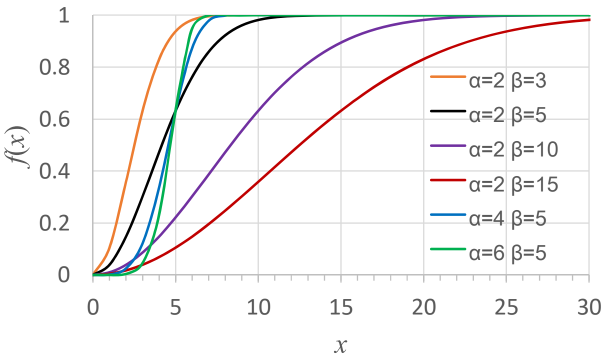
$$PPP\,(x;\alpha,\beta) = \frac{\alpha}{\beta}\left(\frac{x}{\beta}\right)^{\alpha-1} e^{-\left(\frac{x}{\beta}\right)^{\alpha}} \tag{4.3}$$

Where, *PPP* is pedestrian presence probability density function (PDF) of Weibull distribution, α is shape parameter ($\alpha > 0$), and β is scale parameter ($\beta > 0$). Parameters α and β are simultaneously estimated as linear functions of elapsed time, pedestrian red time, crosswalk length and pedestrian arrival rate. Their model can effectively estimate the pedestrian presence probability and is useful to predict pedestrian presence probability distribution. However, the impact of interaction with opposing pedestrian flow is not considered and they analysed relatively lower pedestrian traffic demand. Therefore, in this study the pedestrian presence probability distribution is re-estimated by using the additional dataset which contains a dataset of a crosswalk with relatively higher pedestrian demand.

Weibull distributions are originally used for survival analysis, while it is used in this study due to its mathematical characteristics. The Weibull distribution is versatile distribution that can take on the characteristics of other types of distributions, based on the value of the shape and scale parameters. Therefore, by modelling alpha and beta parameters as a function of influencing factors different characteristics of pedestrian presence probability can be captured. The shapes of the distributions are changed under different combinations of α and β parameter values as shown in **Fig 4.8**.



a) Probability density distribution



b) Cumulative probability distribution

Fig. 4.8 Probability distributions of Weibull distribution

4.4.1 Modeling procedure

Pedestrian presence probability is modelled by using Weibull distribution function and it is estimated by following two steps. The first step is to fit each pedestrian presence probability distribution of each t at each crosswalk by following Weibull distribution and estimate the shape parameter α and scale parameter β of Weibull distribution. Since the pedestrian demands are low for the crosswalks except Sasashima crosswalk, the pedestrian position data is not enough to draw one distribution for each cycle separately. In order to get sufficient number of samples for one distribution, the data of all the cycles are aggregated. However, for Sasashima crosswalk, the distribution is modelled for each cycle independently. The pedestrian density is also important for pedestrian crossing behaviour. Since the y axis of crosswalk will not be considered in this paper, the pedestrian number of subject and opposing flow are normalized by crosswalk width.

The relationship between estimated α and β and number of subject waiting pedestrian q_s and number of opposing waiting pedestrian q_o are shown in **Fig.4.9** to **Fig.12**, respectively. The correlation coefficients r are also shown in the figures. In **Fig.4.9** and **Fig.4.10**, it is found that when q_s have negative relationship with α but weak correlation with β. In **Fig.11** and **Fig.12**, q_o show positive results for α but weak correlation with β. It is indicated that the number of pedestrians for both subject and opposing directions will strongly affect the shape of the distributions. However, for the speed of the platoons, there is no significant effect from number of pedestrians.

In general, the above relations revealed that the number of pedestrians for both subject and opposing directions will strongly affect the shape of the distributions and the speed of the platoon.

Moreover, from all the figures, by comparing the results among t = 5s, 15s and 25s, it is indicated that with increasing t both α and β will increase, which shows the dispersion of pedestrian platoon as elapsed time increase.

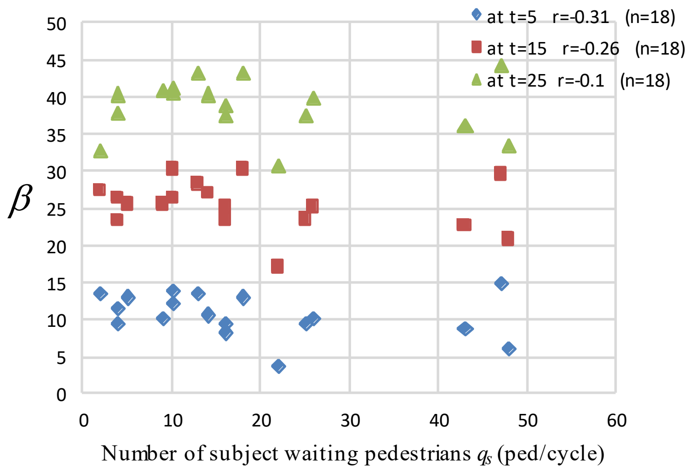


Fig. 4.9 The relationship between number of subject waiting pedestrians and β

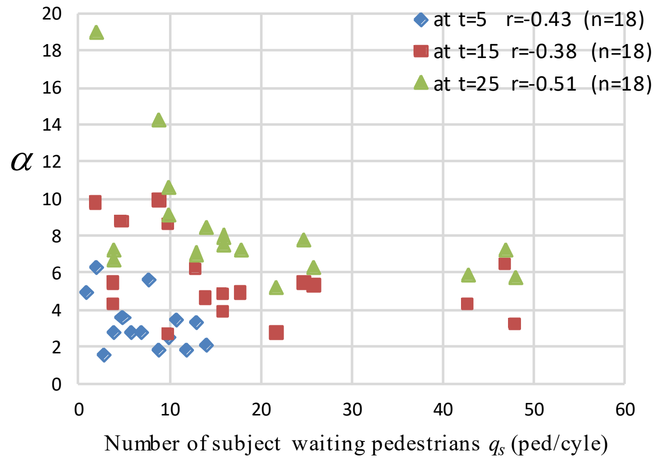


Fig. 4.10 The relationship between number of subject waiting pedestrians and α

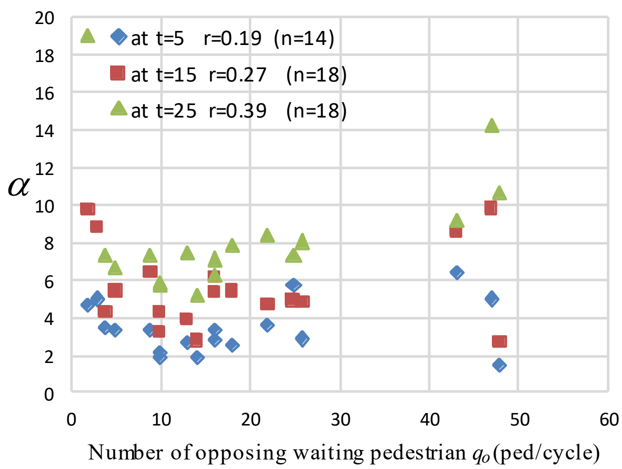


Fig. 4.11 The relationship between number of opposing waiting pedestrians and α

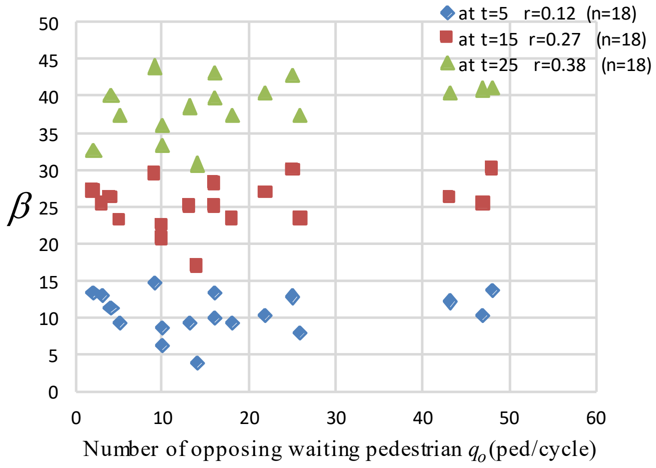


Fig. 4.12 The relationship between number of opposing waiting pedestrians and *β*

After the estimation of *α* and *β* for each distribution the trend with increasing elapsed time and for each crosswalk is checked. **Fig 4.13** and **Fig. 4.14** show the trend of *α* and *β* parameters, from the trend we can notice that the parameters can be estimated as a linear function of influencing factors. Therefore, the next step is to model them using linear function with parameters of elapsed time of PG *t*, crosswalk length *L* and number of subject waiting pedestrians q_s and an additional parameter which is number of opposing waiting pedestrian q_o

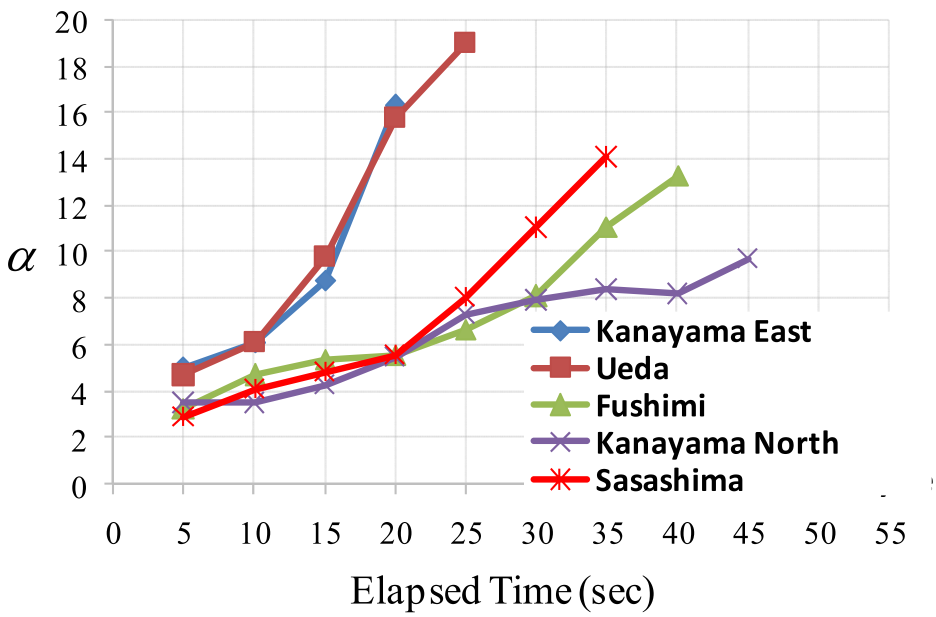


Fig. 4.13 Tendency of parameter α with increasing elapsed time and different crosswalk length

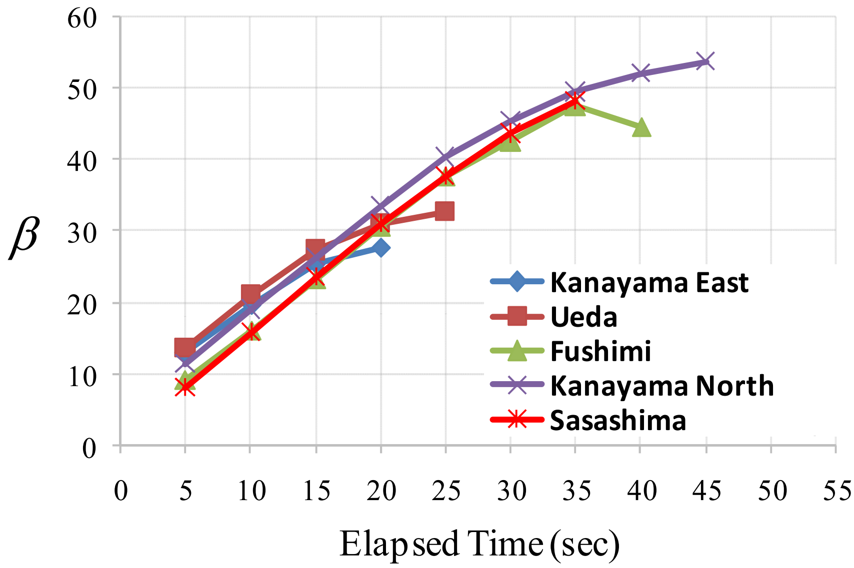


Fig. 4.14 Tendency of parameter β with increasing elapsed time and different crosswalk length

4.4.2 Modified pedestrian presence probability model estimation result

Table 4.2 shows the coefficients for α and β of the pedestrian presence probability model. t has a positive impact to both α and β, which shows pedestrian platoon dispersed in the direction of flow when t increases. L has a negative impact to α, suggesting that there are the wider distributions along the longer crosswalk. On the longer crosswalks, pedestrian platoon tends to disperse. The number of subject direction waiting pedestrian q_s is another significant variable for α and β with negative impact. It shows that higher q_s results in the wider distribution. For the number of opposite direction waiting pedestrian q_o, after addition of the new data samples of Sasashima crosswalk, it become significant and the impact is positive for α. With increase in q_o the distributions become compacted. This is because the high demand of opposing pedestrians will impede the pedestrian platoon to disperse.

Table 4.2 Pedestrian presence probability model coefficients

	Variables	Coefficients	t-stat	R^2
Shape parameter α	Elapsed time of PG t (s)	0.322	16.3	0.70 (n=148)
	Crosswalk length L (m)	-0.206	-3.59	
	Number of waiting pedestrians from subject direction q_s (ped/m/cycle)	-0.477	-3.45	
	Number of waiting pedestrians from opposing direction q_o (ped/m/cycle)	0.679	4.84	
	Constant	7.54	4.82	
Scale parameter β	Elapsed time of PG t (s)	1.30	69.9	0.97 (n=148)
	Number of waiting pedestrian from subject direction q_s (ped/m/cycle)	-1.17	-8.17	
	Constant	6.73	14.6	

4.4.3 Model validation

To validate the proposed model, the pedestrian presence probability distributions by the proposed model compared with observed pedestrian presence distributions at Sasashima and Kanayama east crosswalks. The two crosswalks are selected to represent the performance of the model at lower/higher pedestrian demand and shorter/longer crosswalk length.

Fig.4.15 and **Fig.4.16** show the comparison of observed distribution curve with two estimated pedestrian presence probability distributions using Zhang's model and modified model at different elapsed times. The observed pedestrian presence distribution is plotted by aggregating observed samples of cycles. It is indicated that the estimated pedestrian presence probability distribution curves fitted well by modified model with observed distributions for higher demand crosswalk of Sasashima. While Zhang's model estimated distribution curve for Sasashima has some gaps, since high pedestrian demand crosswalks were not included in their model estimation. However, both estimation models perform similarly for lower demand crosswalk of Kanayama east.

In addition, in order to test the goodness of fit of the modified model distribution curve, K-S test is conducted. D_{max} between the observed distribution and estimated ones(estimated by modified model and Zhang's model) is calculated at each elapsed time, and then compared with the critical value calculated as $D_c = 1.36/\sqrt{n}$ .The summery of the results shown in **Table 4.3**, thus according to the comparison results of all elapsed times, the D_{max} value is less than the critical value(D_c) for most of the elapsed time comparisons of modified model with observed distribution of Sasashima and Kanayama east crosswalks. While in the case of Zhang's model estimation compared with observed ones of Sasashima crosswalks, the D_{max} value is greater than the critical value (D_c) for most of the elapsed times, this implies that the modified model estimated well compared with Zhang's model especially at high pedestrian demand crosswalk of Sasashima.

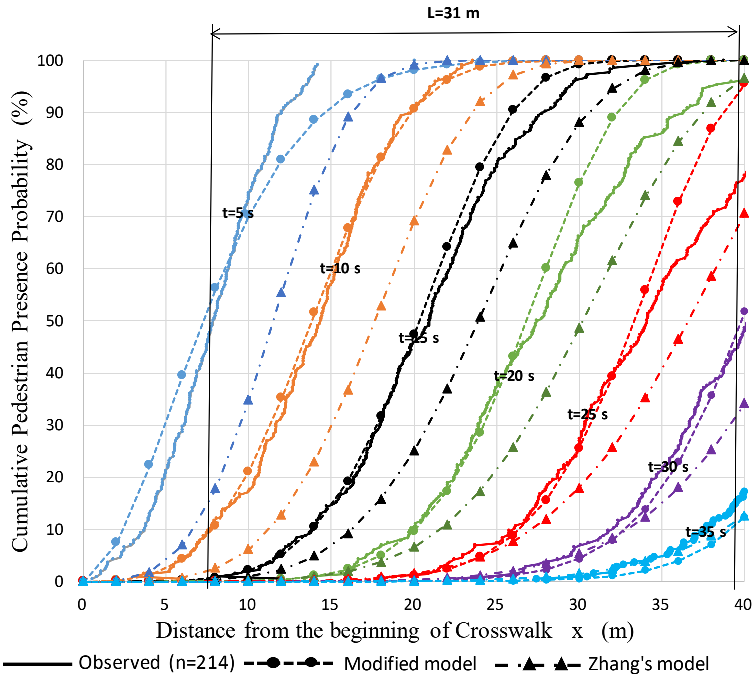


Fig. 4.15 Comparison among modified model, Zhang's model and observed results for Sasashima crosswalk

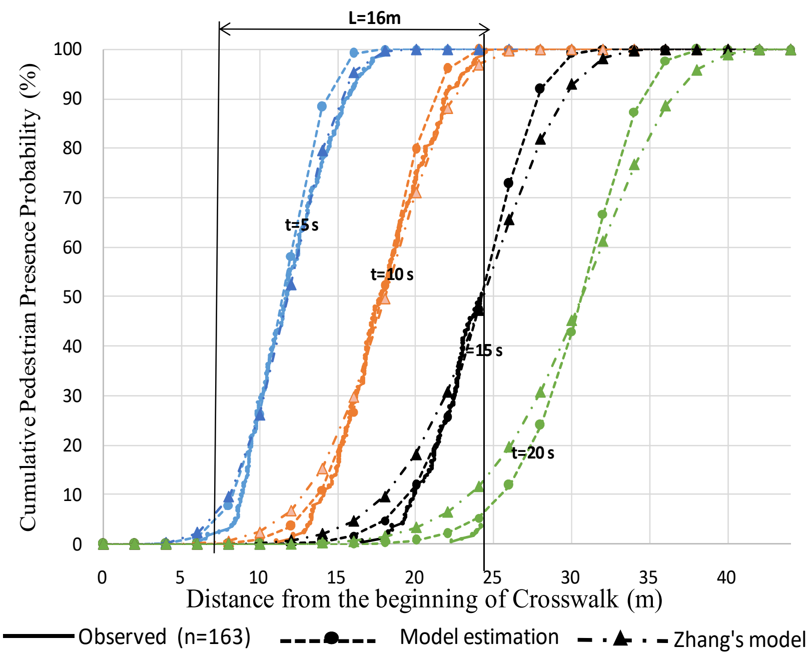


Fig. 4.16 Comparison among modified model, Zhang's model and observed results for Kanayama east crosswalk

Table 4.3 Validation by K-S test

Elapsed time of PG t	Kanayama east					Sasashima				
	Number of samples	Modified		Zhang's		Number of samples	Modified		Zhang's	
		D_{max}	D_c	D_{max}	D_c		D_{max}	D_c	D_{max}	D_c
5	162	0.12	0.11	0.09	0.11	214	0.07	0.09	0.40	0.09
10	163	0.09	0.11	0.08	0.11	215	0.04	0.09	0.32	0.09
15	82	0.05	0.15	0.07	0.15	215	0.08	0.09	0.24	0.09
20	6	0.03	0.56	0.07	0.56	212	0.15	0.09	0.18	0.09
25						205	0.19	0.09	0.17	0.09
30						171	0.24	0.1	0.16	0.10
35						118	0.44	0.13	0.21	0.13

4.5 Estimation of waiting pedestrians' conflict-area presence-time

To estimate the expected pedestrian presence-time, it is necessary to consider the clearance time of pedestrian flow from both sides of the crosswalk at the edges of the conflict-area. Mostly, turning vehicles are conscious to crossing pedestrians when they are present inside the conflict-area. As it is discussed in the previous sections and in chapter 2, the bi-directional crossing pedestrian's behaviour can be influenced by different factors and the position distribution of the crossing pedestrians by elapsed green time can be expressed using the pedestrian presence probability model. After pedestrian crossing probability of the edge of the conflict-area is estimated (at the entry and exit section of the conflict-area), then estimation of the expected elapsed time necessary to cross the conflict-area by assuming some percentage of crossing pedestrian clearance is possible.

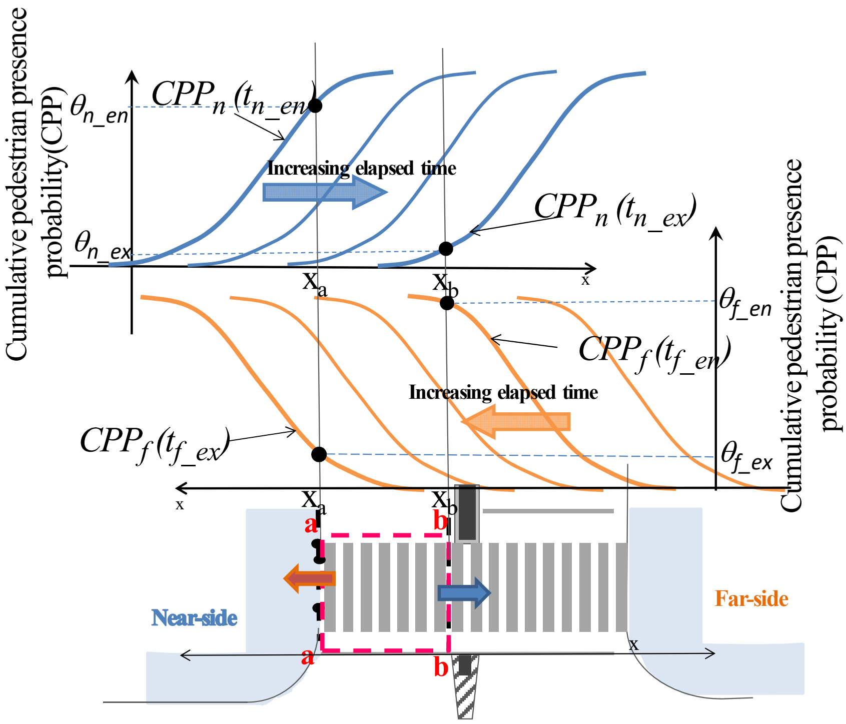


Fig.4.17 Pedestrian crossing probability of conflict-area

4.5.1 Estimation methodology

Fig. 4.17 depicts the concept of how to calculate expected durations that waiting pedestrian platoon blocks the conflict area using the pedestrian probability distribution. The upper part of figure in **Fig. 4.17** shows the time dependent trend of cumulative pedestrian presence probability distribution of near-side pedestrians. Let us define q_n as the number of near-side waiting pedestrians per cycle. As the time goes, the distribution moves toward right-hand side. In this figure, at time $t = t_{n\_en}$, the cumulative probability at cross section a ($x = x_a$) will be θ_{n_en}. This means that the expected number of $(1-\theta_{n\_en})q_n$ has been already passed the cross section a by that time. Suppose this expected number is equal to one as in Equation (4.4). Then, the time t_{n_en} will be considered as the expected time that first pedestrian in the platoon arrives at the cross section a.

$$(1-\theta_{n\_en})q_n = 1 \tag{4.4}$$

Actually, θ_{n_en}, x_a are given from the definition but t_{n_en} is the unknown variable. So the problem to be solved is to find out t_{n_en} so that Equations (4.4) and (4.5) are satisfied.

$$\theta_{n\_en} = \int_0^{t_{n\_en}} PPP_n(x_a, t)dt \tag{4.5}$$

Pedestrian presence probability (PPP) is modelled as a function of elapsed time. However, it is difficult to analytically solve the right-hand side of Equation (4.5). Therefore, numerical calculation is applied to obtain t_{n_en}.

Similarly, time t_{n_ex}, i.e. the expected exit time of the last nearside pedestrian from conflict area, is given by satisfying equations (4.6) and (4.7).

$$\theta_{n\_ex} \times q_n = 1 \tag{4.6}$$

$$\theta_{n\_ex} = \int_0^{t_{n\_ex}} PPP_n(x_b, t)dt \tag{4.7}$$

Where, θ_{n_ex} is cumulative probability at cross section $b(x = x_b)$

Entering (t_{f_en}) and exit time (t_{f_ex}) of far-side pedestrian platoons are also derived by using **Fig. 4.13** following the same approach, so that equations (4.8) to (4.11) are satisfied.

$$(1 - \theta_{f\_en})q_f = 1 \tag{4.8}$$

$$\theta_{f\_en} = \int_0^{t_{f\_en}} PPP_f(x_b, t)dt \tag{4.9}$$

$$\theta_{f\_ex} \times q_f = 1 \tag{4.10}$$

$$\theta_{f\_ex} = \int_0^{t_{f\_ex}} PPP_f(x_a, t)dt \tag{4.11}$$

Where, θ_{f_en} and θ_{f_ex} are cumulative probabilities at cross section $b(x = x_b)$ and $a(x=x_a)$, respectively. q_f is number of far-side waiting pedestrians.

So far, the expected entry/exit times of both nearside and far-side platoons are obtained. Using this information, total expected pedestrian presence-time by waiting pedestrian platoon, EPT, will be calculated by Equation (4.12).

$$EPT = \min\begin{pmatrix} (t_{f\_ex} - t_{n\_en}), \\ (t_{n\_ex} - t_{n\_en}) + (t_{f\_ex} - t_{f\_en}) \end{pmatrix} \tag{4.12}$$

Pedestrian presence-time may have two values as shown in Equation (4.12) and the selection of the minimum value depends on crosswalk length, pedestrian demand and assigned signal timing. These factors influence the overlap time of the last pedestrian from near-side direction flow and first pedestrian of far-side direction flow.

4.5.2 Expected pedestrian presence-time (EPT) model

To generalize the estimation of EPT a model is proposed by considering the influencing parameters of the pedestrian presence time distribution. The two main influencing factors are crosswalk length and bi-directional pedestrian demand. The bi-directional waiting pedestrian demand in the PPP model is expressed as a number of ped/cycle. Therefore, to generalize the EPT model the number of waiting pedestrians per cycle is calculated from the pedestrian arrival rate (ped/hr) as shown in equation 4.13, because the pedestrian arrival rate will change at different cycle length of the signalized crosswalks.

$$q_{near} = \frac{q_n * (C - PG)}{3600} \quad and \quad q_{far} = \frac{q_f * (C - PG)}{3600} \tag{4.13}$$

Where, q_{near} is number of near-side waiting pedestrian per cycle(ped/cycle) , q_{far} is number of far-side waiting pedestrian per cycle (ped/cycle), q_n is near-side pedestrian arrival rate (ped/hr), q_f is far-side pedestrian arrival rate (ped/hr), C is cycle length(s) and PG is pedestrian green time (s) .

Fig 4.18 shows the estimated EPT times based on the procedure discussed in section 4.5.1 for different crosswalk length and bi-directional demand cases for the case of G=65s and PG=60s. The model line is also plotted for each crosswalk under 50/50 pedestrian ratio. When the crosswalk increases EPT has an increasing trend and with increase in pedestrian demand the variation between EPT estimations will be wider due to the effect of bi-directional pedestrian demand

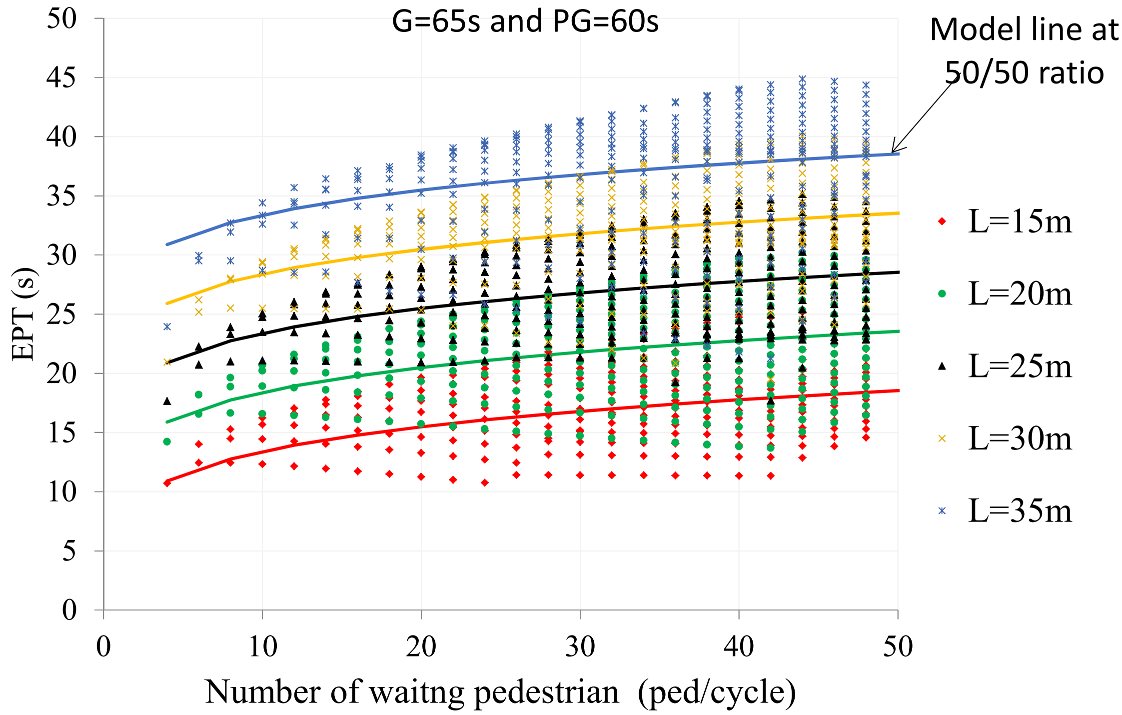


Fig 4.18 Estimated EPT with increase in crosswalk length and bi-directional pedestrian demand

By comparing different trend lines to fit the plots in **Fig 4.18**, it is noticed that the power function line can be adopted, thus equation (4.14) is proposed to generalize the relationship between EPT, crosswalk length and bi-directional pedestrian demand.

$$EPT = aX_1^{(b^*X_2+c)} + d \tag{4.14}$$

Where X_1 is the total number of waiting pedestrians (q_{near} + q_{far}) (ped/cycle), X_2 is far-side waiting pedestrian ratio, *a, b* and *c* are coefficients of the variables, *d* is coefficient for vertical shift of the curve.

The coefficients *a, b* and *c* can be estimated based on the calculated values plotted in **Fig 4.18**. The coefficient *c* is used to move the curve vertically, as it is noticed from **Fig 4.18** the estimated EPT is moving vertically due to the variation of crosswalk length.

Therefore, the coefficients *a, b* and *c* are fitted by regression analysis using power function equation and the coefficient *c* is added on the power function equation based on the crosswalk length conditions to consider the vertical movement of the curve**. Table 4.4** shows the estimated values for the coefficients *a, b* and c.

Table 4.4 coefficient estimation

Parameter	Coefficient	t-stat
a	17.6	147.5
b	0.201	14.07
c	0.023	4.107
R^2	0.96	
n	1316	

The coefficient *c* can be written as a function of crosswalk length since it represents the vertical movement of the power function. Therefore, equation 4.15 shows the generalized EPT estimation equation by considering the influence of crosswalk length and bi-directional pedestrian demand.

$$EPT = 17.6*\left(q_{near} + q_{far}\right)^{0.201*\left(q_{far}/q_{near}+q_{far}\right)+0.023} + L - 25 \qquad (4.15)$$

Where, q_{far} is number of far-side waiting pedestrians (ped/cycle), q_{near} is number of near-side waiting pedestrians(ped/cycle) and L is crosswalk length (m).

Fig 4.19 shows the validation of the proposed EPT estimation equation by comparing with the proposed method calculated values. As it is noticed from **Fig 4.19** the proposed EPT estimation equation can represent the proposed EPT estimation procedure in a good way. The MAPE and RMSE values also revealed the estimation of EPT based on the proposed equation can reasonably represent the estimations of the proposed method estimations.

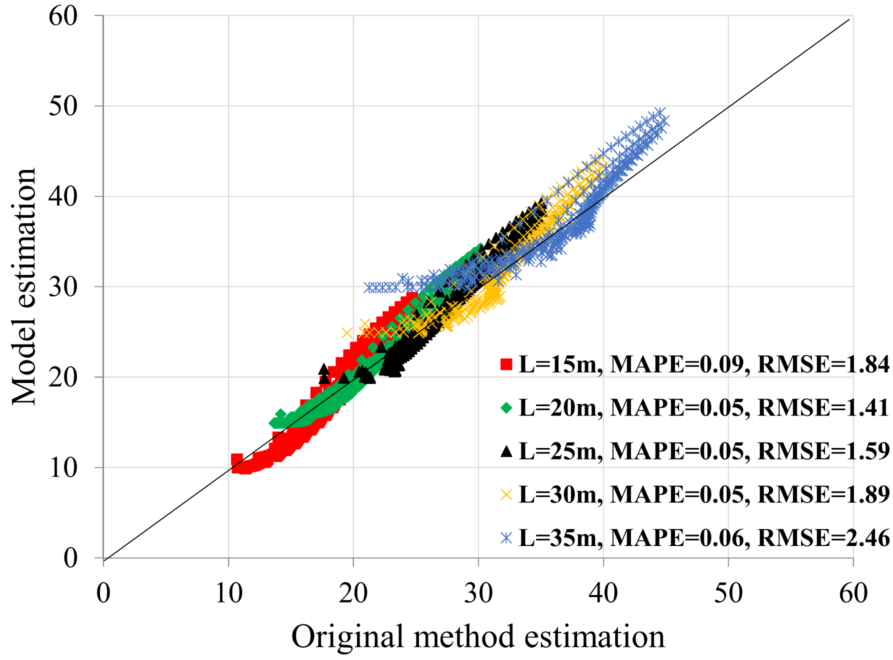


Fig 4.19 Validation of EPT estimation

4.5.3 Observed waiting pedestrian-presence time at the conflict area

The clearance time of the first and the last pedestrians in the waiting pedestrian group are empirically observed when they cross the conflict-area of the observed crosswalks. After the observation of the first and last pedestrian clearance time the waiting pedestrian presence-time is computed following the procedures explained in the above section. **Fig.4.20** shows the observed cumulative distribution of the first pedestrian's entry time and the last pedestrian's exit time from the conflict area at each cycle at Sasashima crosswalk. Similarly, the cumulative distribution at Kanayama East crosswalk is shown in **Fig. 4.21**. The variation of first and last pedestrian passing time distribution is grater for Sasashima crosswalk compared with Kanayama East crosswalk, this is due to the higher pedestrian demand and long crosswalk situation at Sasashima crosswalk pedestrians crossing speed may significantly vary. Using these pedestrian's entry/exit times, the average observed pedestrian presence-time is calculated. The results are 29s for Sasashima and 14.5s for Kanayama East crosswalk, respectively. The reason for higher pedestrian presence-time for Sasashima crosswalk is related with the higher pedestrian demand condition and longer crosswalk length.

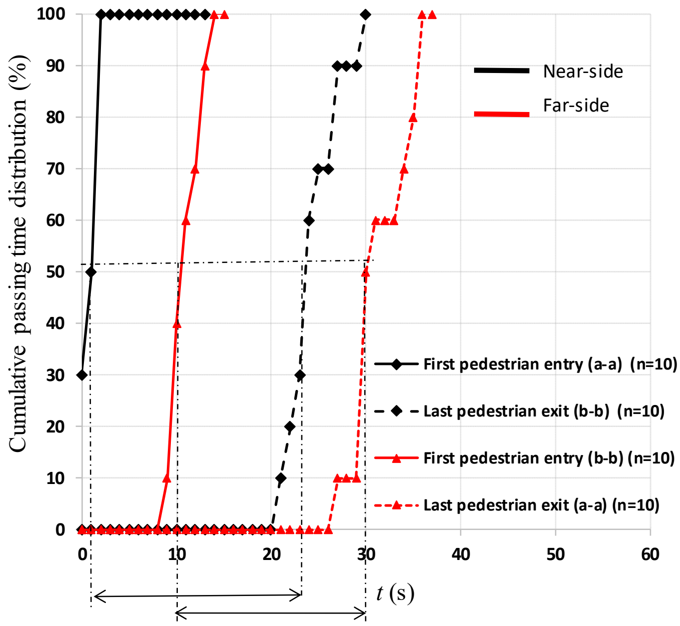


Fig.4.20 First pedestrian's entry time and the last pedestrian's exit time at Sasashima crosswalk.

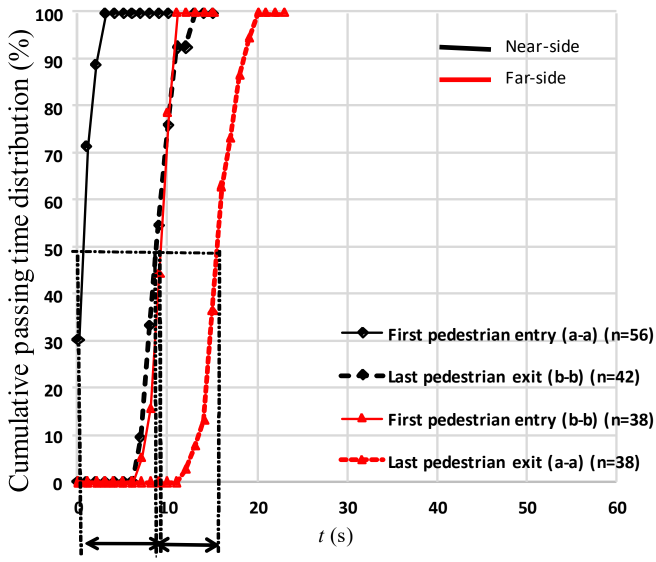


Fig.4.21 First pedestrian's entry time and the last pedestrian's exit time at Kanayama East crosswalk.

4.5.4 Estimated waiting pedestrian presence-time at the conflict-area

Fig. 4.22 and **Fig. 4.23** show pedestrian crossing probability distribution curves at Sasashima and Kanayama East crosswalks, respectively. The pedestrian crossing probability curves are plotted by estimating the cumulative probabilities of crossing pedestrians using the proposed model, when they pass the two edges of the conflict-area with increase in elapsed time. Based on the procedure explained in section 4.4.1 the waiting pedestrian presence-time is estimated for all observed crosswalks.

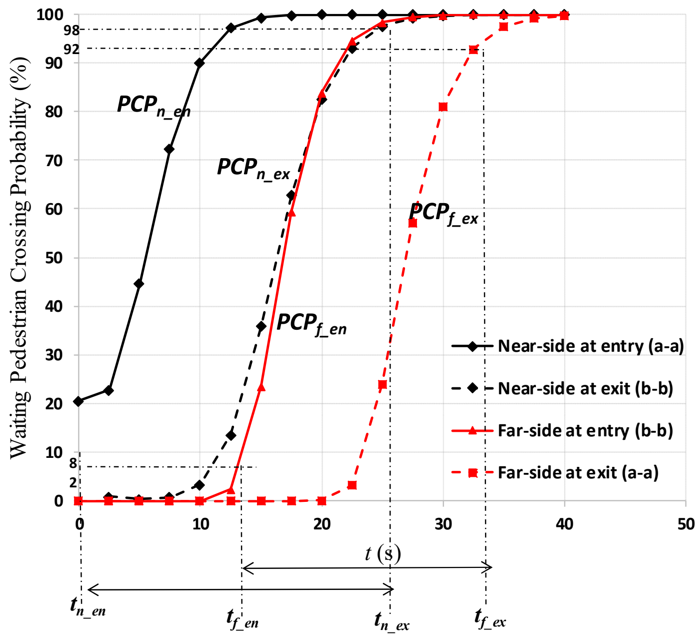


Fig.4.22 Estimated waiting pedestrians crossing time probability at the edges of conflict-area for Sasashima west crosswalk.

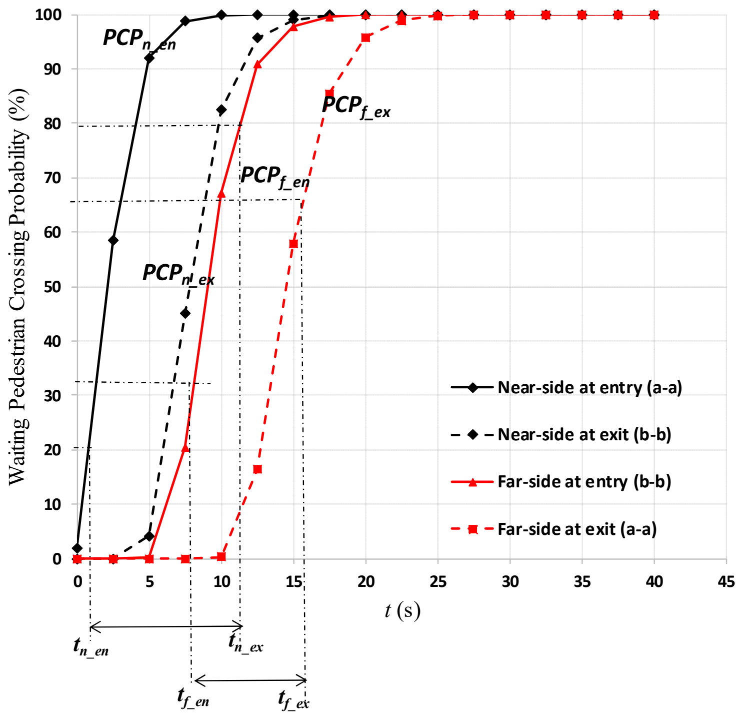


Fig.4.23 Estimated waiting pedestrians passing time probability at the edges of conflict-area for Kanayama east crosswalk.

4.5.5 Comparison of pedestrian presence-time estimation procedures

Fig.4.24 shows the estimated expected waiting pedestrian presence-time for the observed intersections compared with Zhang's model estimation, HCM estimation, JSTE manual estimation and the observed pedestrian presence-time. For JSTE procedure, the JSTE manual estimation procedure for minimum required pedestrian green time as shown by Equation (4.16) is applied. This equation gives the necessary time for the last waiting pedestrian to complete crossing.

$$t = \frac{L}{v_p} + \frac{p}{s * W} \tag{4.16}$$

Where, t is minimum pedestrian green time (s), L is crosswalk length (m), v_p is pedestrian crossing speed (m/s), p is the number of waiting pedestrians at the start of green signal, s is saturation flow rate of pedestrians crossing at signalized intersections (ped/m/s), W is crosswalk width (m). For the comparison in this research, it can be assumed that the waiting pedestrian platoon occupies the conflict area from the onset of green to the time when last pedestrian of far-side waiting pedestrian complete crossing. Then, the pedestrian presence-time will be equal to the minimum pedestrian green time. v_p is set as 1m/s and p is set as 0.92 ped/m (commuting flow) following the settings of the manual.

The HCM procedure proposes adjustment factors to consider the influence of pedestrians on turning vehicles. The first step of the procedure is to estimate the conflict-area occupancy proportion of crossing pedestrians. The average occupancy proportion of crossing pedestrians is proposed by examining the relationship between pedestrian volume and the resulting average occupancy of the conflict area by pedestrians from empirical analysis of different sites. They defined the conflict-area as the crosswalk area in which pedestrians and turning vehicles compete for space. The crosswalk width and a lane width that is used by turning vehicles path roughly bound this area. After they defined the conflict-area they observe the total crossing pedestrian volume per phase and the resulting occupancy of the conflict-area during that phase. Finally, after doing the above procedure for several intersections and plotting the relationship of pedestrian volume with conflict-area occupancy of pedestrians, piecewise linear Equations 4.14 and 4.20 are proposed to estimate average occupancy proportion of crossing pedestrians. Therefore, by utilizing the average pedestrian occupancy proportion (OCC_{pedg}) concept of HCM to apply for waiting pedestrians flow rate, the waiting pedestrian platoon presence-time can be estimated by Equation 4.17.

$$V_{pedg} = V_{ped} * \frac{c}{g_p} \qquad V_{pedg} < 5000 \tag{4.17}$$

$$OCC_{pedg} = \frac{V_{pedg}}{2000} \qquad \text{V}_{\text{pedg}} < 1000 \tag{4.18}$$

$$OCC_{pedg} = 0.4 + \frac{V_{pedg}}{10000} \qquad 1000 < \text{V}_{\text{pedg}} < 5000 \tag{4.19}$$

$$EPT_{HCM} = OCC_{pedg} * g_p \tag{4.20}$$

Where, V_{ped} is pedestrian hourly volume (ped/h), in this study only waiting pedestrian hourly volume is considered, V_{pedg} is pedestrian flow rate (ped/h of green), C is cycle length (sec), g_p is green time of pedestrian (sec), g is green time for vehicles (sec), OCC_{pedg} is average pedestrian occupancy proportion during the effective pedestrian green time, EPT_{HCM} is waiting pedestrians platoon presence-time based on HCM procedure (sec).

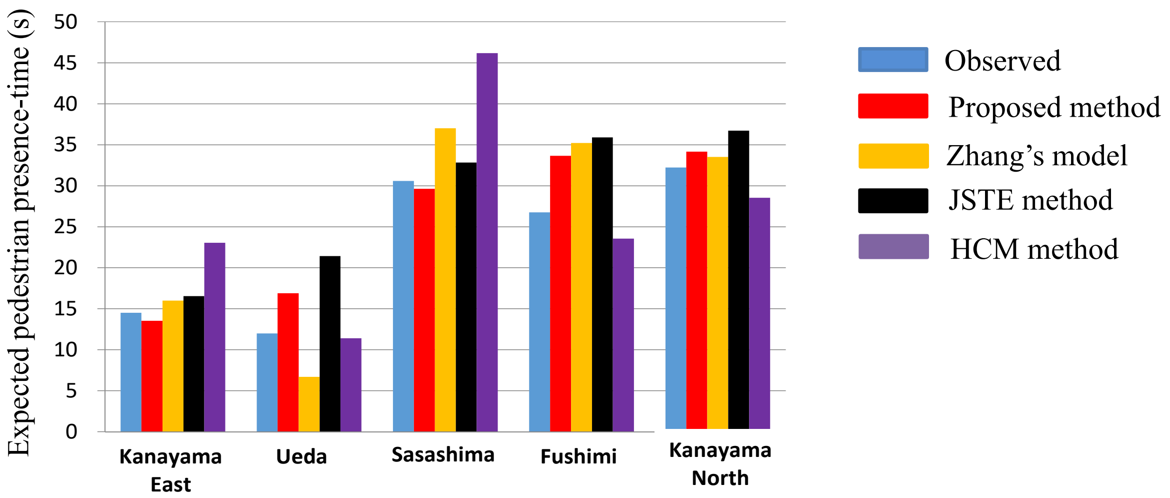


Fig 4.24 Waiting pedestrian presence-time comparison

As it is shown in **Fig.4.24** the estimated pedestrian presence-times have similar trend as the observed ones. In addition, the estimated pedestrian presence-time has similar values compared to estimation by JSTE manual method and estimated by Zhang’s method, except for Ueda intersection. In Ueda intersection, the JSTE manual estimation method is larger than the observed, Zhang’s model estimation and proposed method estimated pedestrian presence times, this shows the estimation procedure of JSTE manual estimation may overestimate minimum crossing time of pedestrians at low pedestrian demand crosswalks. The possible reason can be related with consideration on speed of slowest pedestrian. JSTE manual assumes design speed of the pedestrian speed as 1.0m/s, which is considered as 10th percentile speed of ordinary pedestrians. In Ueda intersection, the numbers of waiting pedestrians are less than 5 ped/cycle. Therefore, due to the low demand of pedestrian the probability that slow pedestrians are observed in each of the cycles will be low. Meanwhile, the proposed model considers the expected pedestrian presence-time taking into account the expected number of waiting pedestrians to determine the entry/exit time of conflict area. This improved the accuracy of pedestrian presence time estimation.

Estimated expected pedestrian presence-times of Kanayama North, Fushimi and Sasashima crosswalks are relatively greater than other crosswalks, with 34.8s, 35.5s and 34.6s pedestrian presence-time, respectively. Therefore, when crosswalk length is long the pedestrian presence distribution become wide along the crosswalks and the pedestrian presence time will be larger due to the dispersion of crossing pedestrians along the crosswalk. The estimated pedestrian presence-time of Kanayama East and Ueda crosswalks are lesser than other crosswalks, with 16.7s and 10.4s pedestrian presence-times, respectively. This is because of the short crosswalk length and low pedestrian demand. The pedestrian presence time probability is inversely related with the available crossing time for turning vehicle within the green interval, when expected pedestrian presence-time increase then the available time for turning vehicles will decrease. This must be balanced by proper signal timing design.

Furthermore, as it is shown in **Fig. 4.24**, the estimated expected pedestrian presence-time of Sasashima crosswalk based on Zhang's model is greater than the other estimations. The possible reason is that; higher pedestrian demand condition is not considered in Zhang's model. When the HCM procedure estimation is compared with the other methods as it is shown in **Fig. 4.24**, it has larger estimation for Kanayama east and Sasashima crosswalks and lower estimation for Kanayama north and Fushimi crosswalks. The possible reasons for the deviation of the HCM procedure estimations are related with proper consideration of different crosswalk layout effect on pedestrian crossing behaviour. Sasashima crosswalk is wider compared with others, therefore positions of crossing pedestrians may laterally distribute along the width of the crosswalk and due to that their occupancy time can be lower even if the number of pedestrians is high. The variation in estimation of HCM procedure for Kanayama east, Kanayama north and Fushimi crosswalks are related with consideration of crosswalk length, HCM procedure estimates occupancy time based on pedestrian flow rate, however pedestrian crossing behaviour is highly dependent on crosswalk length as it is discussed in the above sections. Pedestrian presence-time estimation of HCM procedure has an increasing trend when pedestrian flow rate increases; meanwhile pedestrian flow and pedestrian occupancy have linear relation as shown in Equation 4.15 and 4.16, regardless of short or long crosswalks. While the proposed method considers crosswalk length and bi-directional pedestrian flow interaction effect as influencing factors it can represent the observed occupancy time in a better way compared with HCM procedure. However, the HCM procedure estimation has similar values compared with proposed method estimation and observed ones for Ueda crosswalk. This implies, at low pedestrian demand condition the proposed method and HCM procedure estimation method may estimate waiting pedestrian presence-time similarly. In general, from the above discussion we can conclude that, in the pedestrian presence-time estimation procedure the consideration of geometric layout of the crosswalk is necessary in addition to pedestrian demand.

In addition, to quantitatively evaluate the methods the mean absolute percentage error (MAPE) of proposed method estimation, Zhang's model estimation, HCM estimation and JSTE estimation resulted as 13.3s, 21.1s, 27.5s and 33.1s, respectively. This implies that the proposed method can estimate pedestrian presence-time in a better way compared with the others, since the MAPE value of the proposed method estimation is less than the other estimation methods.

From the analysis it is noticed that, waiting pedestrians crossing position on the crosswalk at elapsed time t is influenced by opposing pedestrian counter flow, due to this the clearance time of pedestrians from the conflict-area highly affected by pedestrian distribution along the crosswalk. Therefore, in the operational performance evaluation of signalized intersections to consider the influence of pedestrians on turning vehicles, proper estimation of pedestrian presence time is necessary. Moreover, in signal timing design, the proper estimation of expected pedestrian presence-time of crossing pedestrians is useful. Because, improper signal phase design may affect the capacity and safety of signalized crosswalks. If the assigned green time for turning vehicle is highly influenced by crossing pedestrians, then the capacity of turn-lane may significantly decrease. In addition, when turning vehicles wait for long time to yield crossing pedestrians they may show impatient driving behaviour, which causes safety problem for crossing pedestrians.

4.5.6 Validation of pedestrian-presence time by comparing with unused time of turning vehicles

Unused time is measured from observation by recording the time when the back wheel of the first left turning vehicle pass the conflict-area. When left turning vehicles (LTV) has the same onset of green time with pedestrians, then at the beginning of green time they will not use some portion of assigned green time. This is due to the priority of crossing pedestrians. The magnitude of unused time depends on the number and crossing characteristics of waiting pedestrians. In addition, the geometric layout of the crosswalk may influence the passing time of the left turning vehicle. **Fig. 4.25** shows one example of the relation between unused times, observed pedestrian presence-time and estimated pedestrian presence time under different pedestrian demand conditions for Kanayama east crosswalk. As it is clearly shown in **Fig. 4.25,** the estimated and observed presence-times show similar trend under all pedestrian demand conditions, while the unused times are greater than that of the presence-time values.

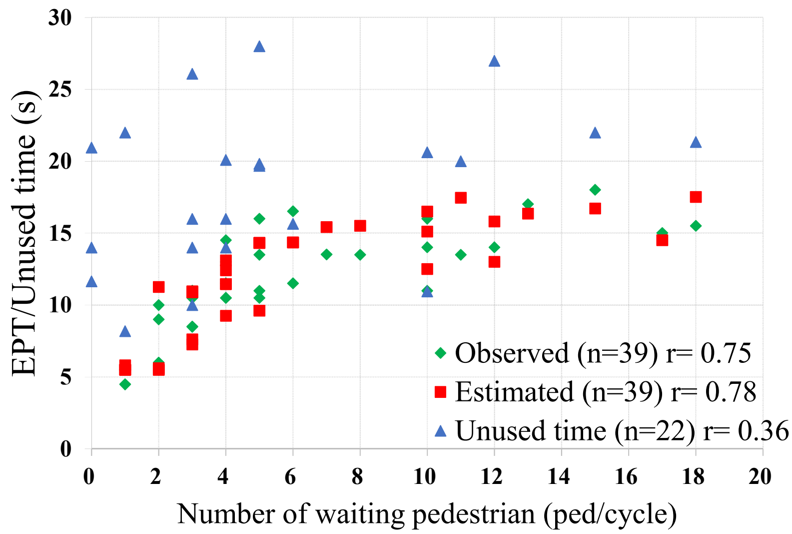


Fig. 4.25 Comparison of EPT and unused time

Fig.4.26 and **Fig.4.27** shows observed cumulative distribution of the first waiting pedestrian's entry time, the last waiting pedestrian's exit time and the unused time of first LTV from the conflict-area at each cycle of Kanayama crosswalk and Nishiosu crosswalk, respectively. The variation of first and last waiting pedestrians enter and exit time distributions are greater for Nishiosu crosswalk compared with Kanayama crosswalk. This is due to the longer crosswalk situation at Nishiosu crosswalk pedestrians crossing speed may significantly vary.

As it is shown in **Fig.4.26**, the cumulative passing time distribution of first LTV is located after the far-side last waiting pedestrian's exit time distribution for Kanayama crosswalk. On the other hand, for Nishiosu crosswalk case shown in **Fig.4.27** the passing time distribution of the first LTV is located between the near-side last waiting pedestrian's distribution and the far-side first waiting pedestrian's distribution. The reason behind this difference is mainly due to the variances in pedestrian demand and crosswalk length of the two crosswalks. When the number of waiting pedestrians is higher, the possibility is high for the far-side first pedestrian to arrive at the conflict-area before the near-side last pedestrian exit. Similarly, in shorter crosswalks the far-side first waiting pedestrians will arrive quickly before the near-side last waiting pedestrians exit from the conflict-area.

Therefore, from the above observations we can understand that the blockage effect of left turning vehicles under the influence of pedestrians depends on the entry and exit times of waiting pedestrians, waiting pedestrian demand and geometric layout of the intersection.

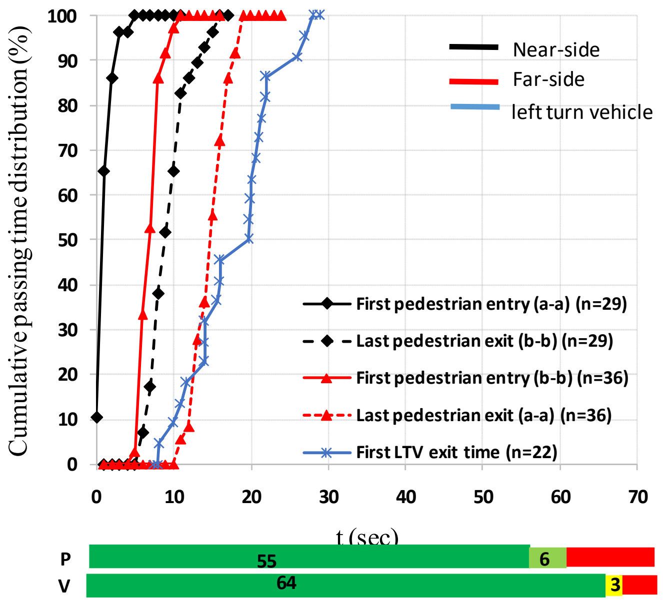


Fig.4.26 First pedestrian entry, last pedestrian exit and first LTV exit time distribution at Kanayama crosswalk

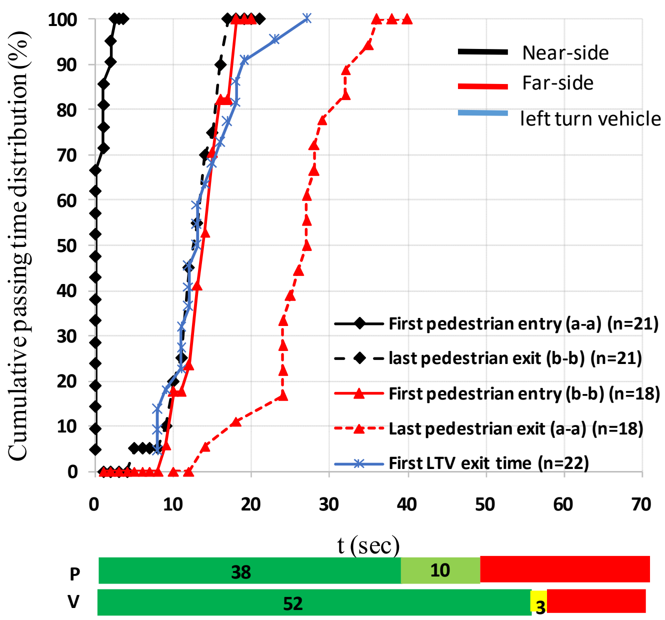


Fig.4.27 First pedestrian entry, last pedestrian exit and first LTV exit time distribution at Nishiosu crosswalk

As it is shown in **Fig.4.28** the estimated pedestrian presence-times are compared with observed pedestrian presence times and unused times of first LTV. The observed and estimated times are plotted using the average number of waiting pedestrians. The unused time is the average of the observed unused times per cycle. In general, the trend of estimated pedestrian presence-times compared with others follow similar trend.

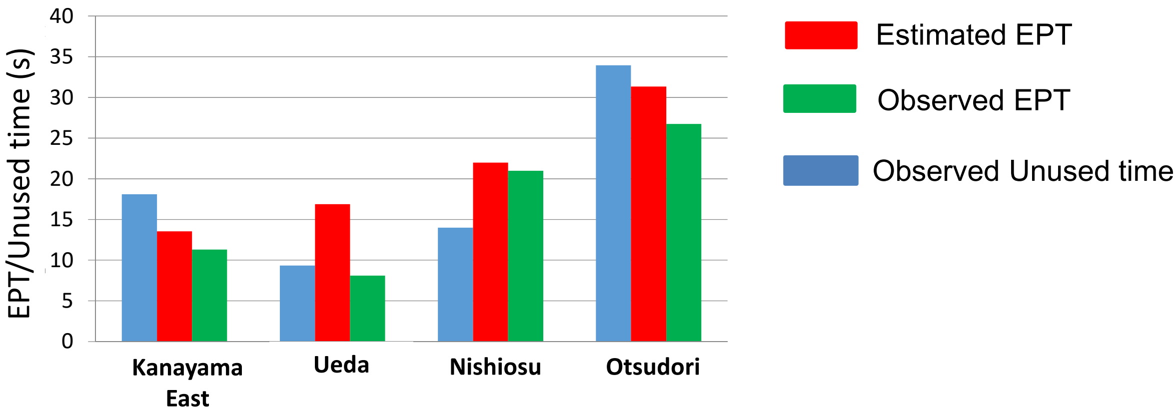


Fig. 4.28 Comparison of observed, estimated pedestrian presence-time and observed unused time of LTV

From the observations it is found that there are very low pedestrian demands on Ueda and Nishiosu crosswalks. Therefore, there is a gap between the last near-side pedestrian exit time and the entry time of first far-side pedestrian. In this case there are not overlap time between near-side pedestrian platoon blockage time and far-side pedestrian platoon blockage time. Then the first LTV vehicles have a chance to pass before the far-side pedestrian arrive the conflict-area. It will result in short unused time of LTV. Therefore, as it is shown in **Fig.4.28** the observed unused time (blue colour) is shorter than proposed method estimation (red colour) for Ueda and Nishiosu crosswalks. Furthermore, since the Nishiosu crosswalk is longer than Ueda crosswalk, the larger difference between observed unused time of LTV and proposed method estimation can be found. Kanayama and Otsu-dori crosswalks have higher number of pedestrians compared with the others. Therefore, the possibility is less for the far-side first pedestrian to arrive at the conflict-area before the near-side last pedestrian exit from the conflict-area, due to this the blockage effect is determined by the entry time of the near-side first pedestrian and the exit time of the far-side last pedestrian. The proposed method estimation is slightly larger than the observed ones for Otsu-dori and Kanayama crosswalks; the possible reason is related with the variation in number of pedestrians per cycle. The proposed method used the average number of pedestrians while the observed pedestrian presence-times are calculated per cycle and then averaged. In addition, the unused time of first LTV at Kanayama and Otsu-dori crosswalks are larger than the proposed method estimated presence-times. The possible reason is related with the arrival time of turning vehicles from stop line to the crosswalk, due to this sometimes even at low pedestrian demand condition LTV may take long time to arrive and pass the crosswalk. Therefore, in this case unused time has extra time (in addition to pedestrian presence-time). Estimated pedestrian presence-time of Otsu-dori crosswalk is relatively greater than other crosswalks. Therefore, when crosswalk length is long the pedestrian presence distribution become wide along the crosswalks and the presence-time will be larger due to the dispersion of crossing pedestrians along the crosswalk.

4.6 Summary

In this chapter, the waiting pedestrian's characteristics are empirically observed and modelled. The observed crossing pedestrian's position distribution under different crosswalks within elapsed green time is characterized by grouping the bi-directional pedestrian flows. The influencing factors for the bi-directional pedestrian movement within the assigned green time are also investigated. The observation indicated that the position distribution of crossing pedestrians along the crosswalk can be influenced by signal timing, bi-directional pedestrian flow interaction and crosswalk length. The waiting pedestrian presence probability is modelled based on the pedestrian position distribution observation results of signalized crosswalks. The Weibull distribution α and β coefficients of the pedestrian presence probability model are fitted by considering the influencing factors. Elapsed time has a positive impact to both α and β, which shows pedestrian platoon dispersed in the direction of flow when elapsed time increases. Crosswalk length has a negative impact to α, suggesting that there are the wider distributions along the longer crosswalk. On the longer crosswalks, pedestrian platoon tends to disperse. The number of subject direction waiting pedestrian q_s is another significant variable for α and β with negative impact. It shows that higher q_s results in the wider distribution. For the number of opposite direction waiting pedestrian q_o, after addition of the new data samples of Sasashima crosswalk, it become significant and the impact is positive for α. With increase in q_o the distributions become compacted. This is because the high demand of opposing pedestrians will impede the pedestrian platoon to disperse.

Moreover, after the position distribution of the crossing pedestrians by elapsed green time is expressed using the pedestrian presence probability model, the pedestrian crossing probability of the edge of the conflict-area is estimated (at the entry and exit section of the conflict-area), then estimation of the expected elapsed time necessary to cross the conflict-area by assuming some percentage of crossing pedestrian clearance is computed. After that, to generalize the estimation of EPT an equation is proposed which consider the influence of crosswalk length and bi-directional waiting pedestrian demand.

Finally, the estimated waiting pedestrian presence-time at the conflict-area is compared with estimation procedures of Zhang's method, HCM procedure, JSTE procedures and also the unused time of left turning vehicles, the comparison result showed that the proposed estimation method can represent the waiting pedestrian's presence-time realistically by considering significant influencing factors.

Chapter 5

Left-turning Vehicles Discharge Flow

5.1 Overview

In the previous chapter the crossing pedestrian's characteristics is investigated to understand the effect of pedestrians on left turning vehicles. To effectively estimate the left turn lane capacity, under concurrent signal timing design the proper consideration of crossing pedestrians and left turning vehicles interaction is necessary. Therefore, in this chapter the other main user at signalized intersection i.e. left turning vehicles characteristics will be discussed. After the empirical observation of the left turning vehicles movement it is attempted to model the discharge flow rate of under the presence of crossing pedestrians at different crosswalk layout conditions.

5.2 Observed intersections

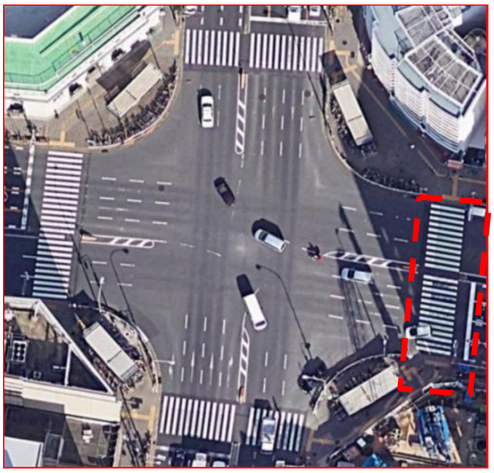
5.2.1 Study sites and traffic conditions

For the left turning vehicles discharge flow rate analysis, six signalized intersections located in Nagoya, Japan are selected and the movements of vehicles are video recorded in the weekdays, considering the peak hour and under sunny weather condition. The intersections have different pedestrian demand situations and crosswalk geometric characteristics. The geometric condition, traffic condition and signal settings of the selected signalized crosswalks are summarized in **Table 5.1** and the layout of the observed crosswalks are shown in **Fig. 5.1**. from (a) to (f).

Table 5.1 Conditions of observed sites

Intersection name			(a) Kanayama	(b) Ueda	(c) Nishi osu	(d) Otsu -dori	(e) Mizuho - kuyakus hyo	(f) Imaike
Subject crosswalk			East	South	North	West	East	East
Signal phase	PG (s)		48-61	47-62	38	37	58	45
	G (s)		57- 71	61-67	52	54	70	60
	PFG (s)		6	8	8	9	4	8
	Cycle length (s)		148-174	144-176	160	160	140	160
Waiting pedestrians (ped/cycle)	Near -side	Range	0-13	0-3	0-4	1-13	3-25	1-6
		Average	5	2	3	6	9	4
	Far- side	Range	0-10	0-4	0-3	2-12	1-5	2-9
		Average	4	2	2	5	3	5
Arriving pedestrians (ped/cycle)	Near -side	Range	0-6	0-2	0-3	0-3	4-9	1-4
		Average	3	1	2	2	7	3
	Far- side	Range	0-3	0-5	0-2	1-6	0-1	1-3
		Average	2	2	1	3	1	2
Crosswalk geometry	Length (m)		16	21	32	34	10	21
	Width (m)		6	5	6	6	6	7
Intersection angle θ (degree)			93	124	78	91	93	100
Setback distance of LTV stop line *ls* (m)			21.4	16.5	17.5	21.5	13.5	28
Setback distance of subject crosswalk *lc* (m)			4.0	20.1	13.1	12.5	1.5	14.5
Number of sample cycles for LTV analysis			14	24	27	19	4	9

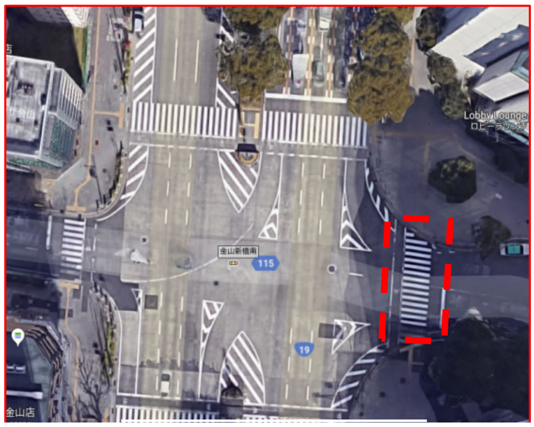
**Mizuho-kuyaushyo hereinafter named as Mizuho.*



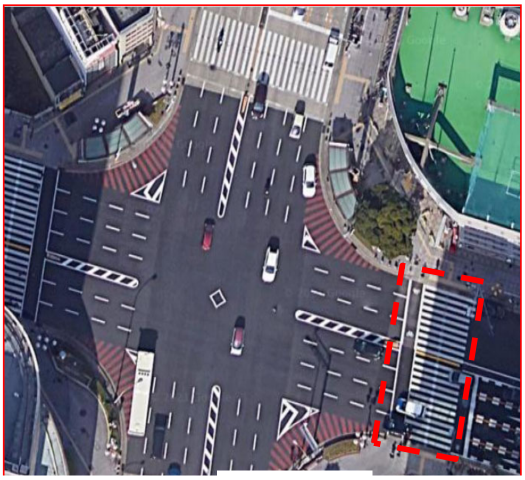
(a) Otsu-dori

(b) Ueda

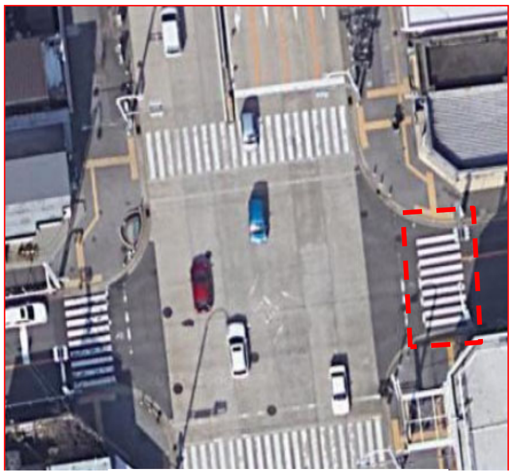
(c) Nishiosu



(d) Kanayama



(e) Imaike



(f) Mizuho-kuyakushyo

Fig.5.1 Layout of observed crosswalks

The positions of pedestrians when they cross along the crosswalk and when the rear wheel of the left turning vehicles start from the stop line until it passes the conflict-area at every 0.1s are manually extracted from observation videos by using the image processing system Traffic Analyzer (Suzuki and Nakamura, 2006) then the coordinates in these images are converted to global coordinate through the projective transformation. Kalman smoothing method is applied to estimate trajectories of left turning vehicles at each time interval. The observed cycles are selected by considering certain constraints that may affect the discharge flow rate i.e. only passenger cars, no impact of bicycles and no impact of heavy vehicles. All observed cycles had continues left turning vehicles flow at the onset of green, with a minimum five number of lined up left turning vehicles.

5.2.2 Observation of left-turning vehicles discharge flow

In this study the discharge flow of left turning vehicles is measured at the edge of the crosswalk located opposite to the x-axis as shown in **Fig.5.2**. The travel time of left turning vehicles from the stop line up to the discharge flow measurement section is affected by stop-line and crosswalk setback distances. As our target is to understand the influence of crossing pedestrians on the left turning vehicles movement, the observation must be after the vehicles pass the shared space.

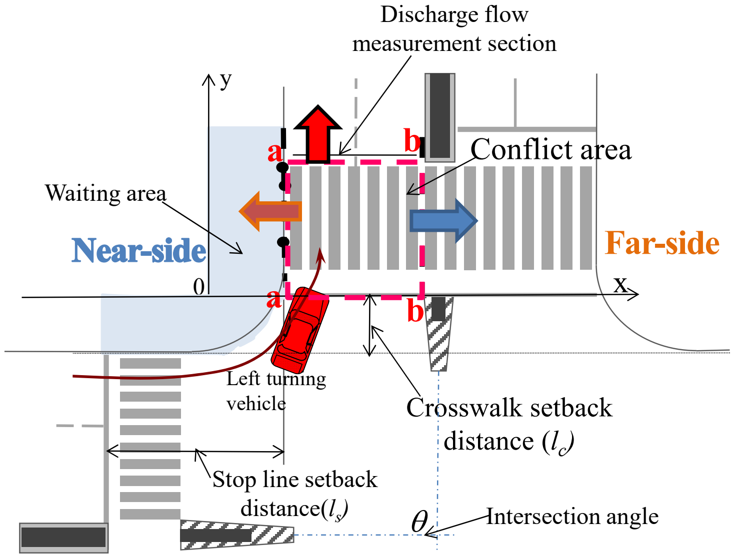


Fig.5.2 Layout of subject crosswalk and definitions of basic terms

In the observation the last left turning vehicle is the last vehicle passing the measurement cross-section in the group of left turning vehicles under continues flow condition and t_n is the measurement cross-section passing time of the last vehicle in the group of continues left turning vehicles that started from the onset of green. Pedestrian green time (PG) is the time interval assigned for crossing pedestrians from both sides of the crosswalk. Pedestrian flash green time (PFG) is the time interval arriving pedestrian are not allowed to enter into the crosswalk, however pedestrians who are still on the crosswalk have to finish crossing within this time interval and Green time (G) is assigned green time for left turning vehicles.

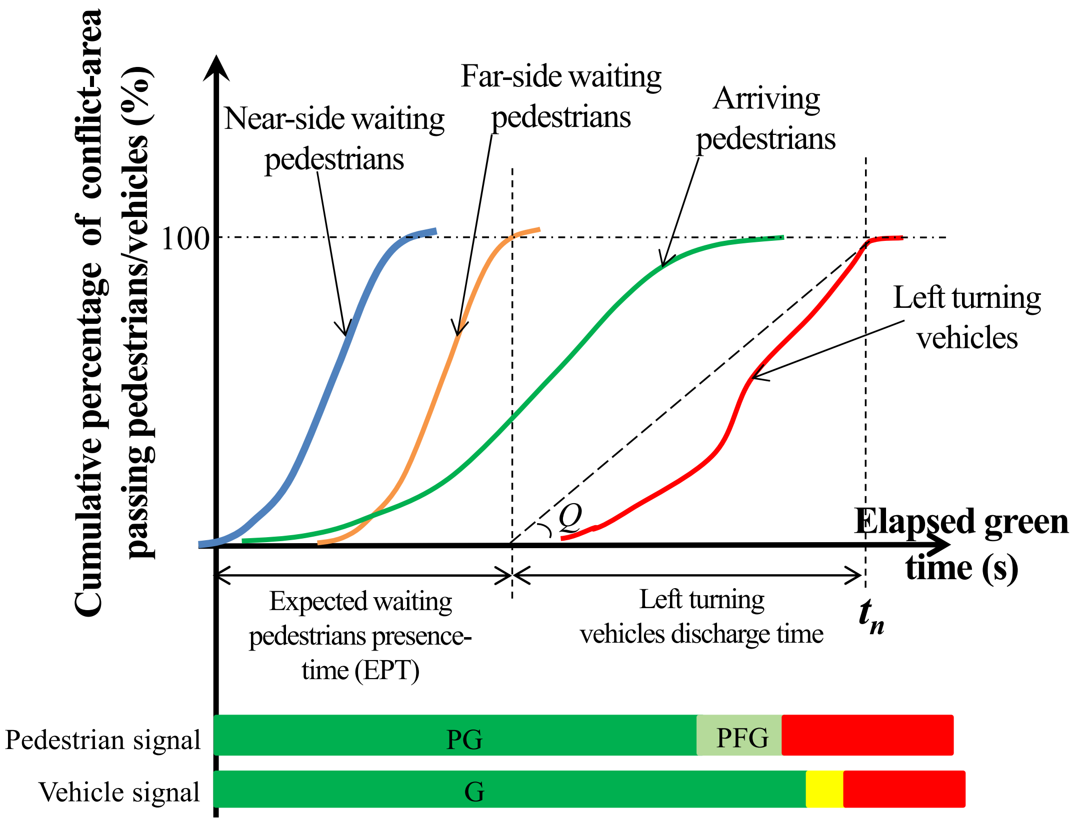


Fig.5.3 Pedestrian and left turning vehicle conflict-area passing distribution

As it is shown in **Fig.5.3** within the assigned concurrent signal timing we may have two portions (EPT and left turning vehicles discharge time) of time intervals. The time intervals are proposed based on the crossing pedestrians and left turning vehicles movement characteristics. At the onset of PG, a group of waiting pedestrians start to pass the conflict-area and when the last pedestrians from near-side/far-side clear from the conflict-area the first time interval portion will finish. Arriving pedestrians can enter the crosswalk at any time after the onset of green until they clear in the PFG time; meanwhile left turning vehicles may pass the crosswalk by searching acceptable gap between arriving pedestrians. Therefore, the second time interval portion is the left turning vehicles discharge flow time by interacting with arriving pedestrians.

Left tuning vehicles discharge time is defined as the expected time after the waiting pedestrian platoon clear from the conflict-area until the last queued left turning vehicle pass the conflict area (the observation section is the edge of the conflict-area along the left turning vehicles movement direction as shown in **Fig.5.2**). In the cumulative distribution of number of left turning vehicles passing the conflict-area as shown in **Fig.5.3**, the slope of the line which connects the end of *EPT* and t_n is the discharge flow rate (Q) of the left turning vehicles movement within available discharge time under the influence of arriving pedestrians. Left turning vehicle discharge flow rate can be influenced by the pedestrian arrival rate and the geometric layout of the crosswalk and intersection, like crosswalk setback distance, intersection angle, stop-line setback distance and crosswalk length. To estimate the discharge flow rate of left turning vehicles, observation of the conflict-area passing time of the last queued vehicle is necessary. Therefore, discharge flow rate for each observed cycle can be estimated using Equation (5.1).

$$Q = 3600 * \left(\frac{n}{t_n - EPT} \right) \tag{5.1}$$

Where, Q is the discharge flow rate (veh/hr), n is the number of discharged left turning vehicles, t_n is crosswalk exit time of the last queued left turning vehicle and *EPT* is the observed waiting pedestrian presence-time (s).

5.2.3 Left-turning vehicles discharge flow distribution within the green time

Fig.5.4 show example cycles of observed left turning vehicles passing time distribution within available green time for Kanayama crosswalk. The distribution curves located in different horizontal position and also they have different slope (Q), which implies that for the observed cycles the influence of pedestrians and the response of left turning vehicles may significantly fluctuate when we compare observed cycles. As shown in **Fig.5.4**, from distribution curves we can notice that the first left turning vehicle passing time is shifted at least 15s after the onset of green, two factors may contribute for this; the first one is the influence of waiting pedestrians i.e. waiting pedestrians may block the movement of left turning vehicles and the second one is related with geometric layout of the crosswalk i.e. if the position of stop line for the left turning vehicles is far from the crosswalk it takes time to arrive at the subject crosswalk before they pass the conflict-area.

In **Fig.5.4** the red color distribution curve represents an observed cycle without arriving pedestrian presence. As it is shown in the figure, the position of the red curve is in the front position and the curve slope is relatively steep compared with the others. This indicate that, if there are no arriving pedestrians left turning vehicles start to discharge in short time after the waiting pedestrians clear from the conflict-area. The green color distribution curve represents an observed cycle in which arriving pedestrians enter only from near-side direction. Therefore, the position of the curve is shifted to the right from the red curve and the slope is similar with the red curve, because only near-side arriving pedestrians influence the left turning vehicles. The other curves (black and blue) characterize an observed cycle with both directional arriving pedestrians. Their position and slope are different because of the variance in the number of arriving pedestrians per cycle and directional composition of arriving pedestrians.

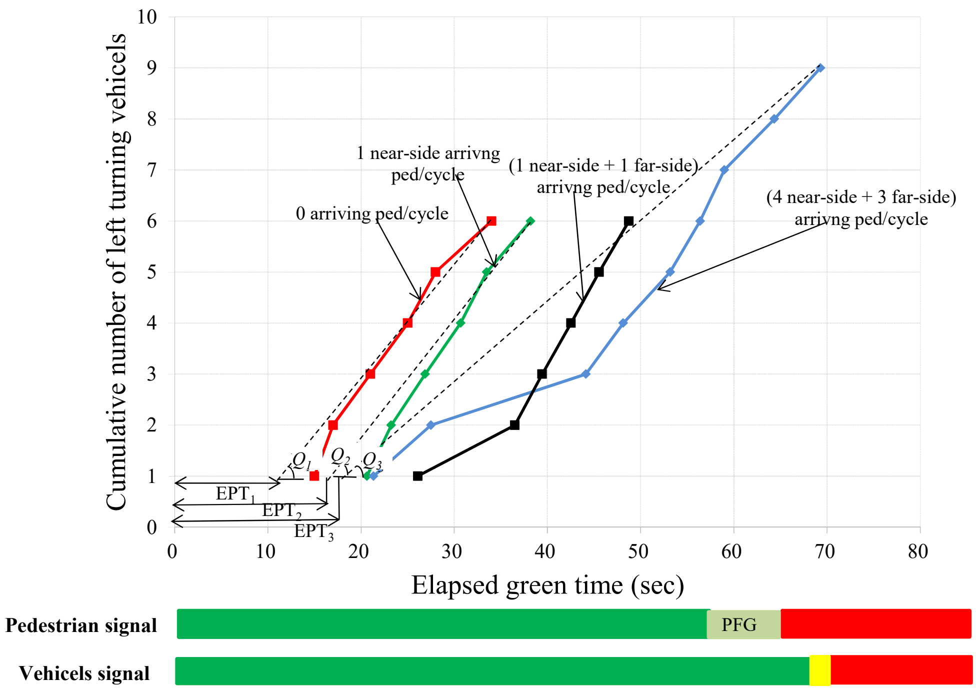


Fig.5.4 Cumulative number of left turning vehicles for Kanayama crosswalk

5.3. Modeling left-turning vehicles discharge flow rate

5.3.1 Influencing factors

After the observation of left turning vehicles passing time distribution the discharge flow rate is computed using Equation (5.1) for all observed cycles. **Fig.5.5** and **Fig.5.6** shows examples of the discharge flow rate related with pedestrian arrival rate of Kanayama and Otsu-dori crosswalks. For both crosswalks, the discharge flow rate has a decreasing trend with increase in near-side and far-side pedestrian arrival rates, as the correlation coefficients *r* are negative. The discharge flow rates of Kanayama crosswalk are located below that of Otsu-dori plots. Left turning vehicles may have difficulty to find acceptable gaps quickly in Kanayama crosswalk because the crosswalk length of Kanayama is shorter than Otsu-dori, consequently pedestrians may arrive at the conflict-area within short time.

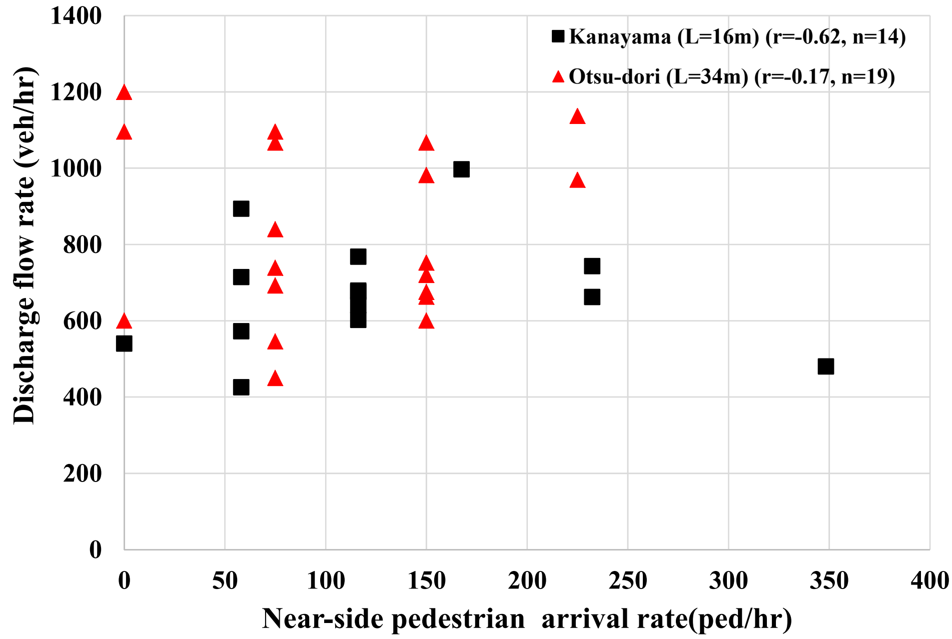


Fig.5.5 Observed discharge flow rate related with near-side pedestrian arrival rate.

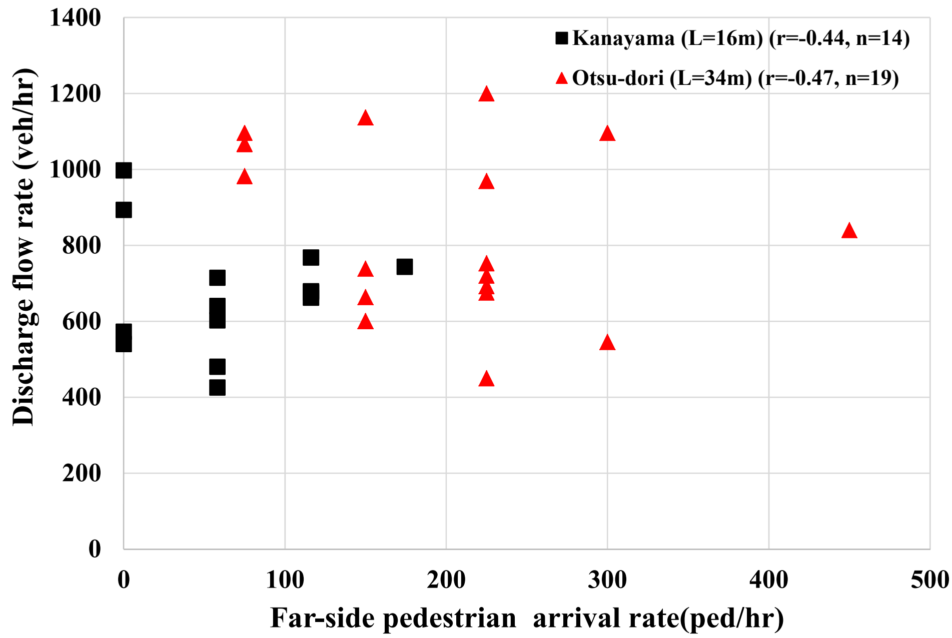


Fig.5.6 Observed discharge flow rate related with near-side pedestrian arrival rate.

In addition, **Fig.5.7** shows the variation of discharge flow rate with increasing crosswalk length, the discharge flow rate is significantly differing by increasing crosswalk length. For shorter crosswalks left turning vehicles may wait for longer time before they pass the conflict-area, consequently discharge flow rate is lower. Because, entering pedestrians may presence on the conflict-area frequently due to the short arriving time in the case of short crosswalks.

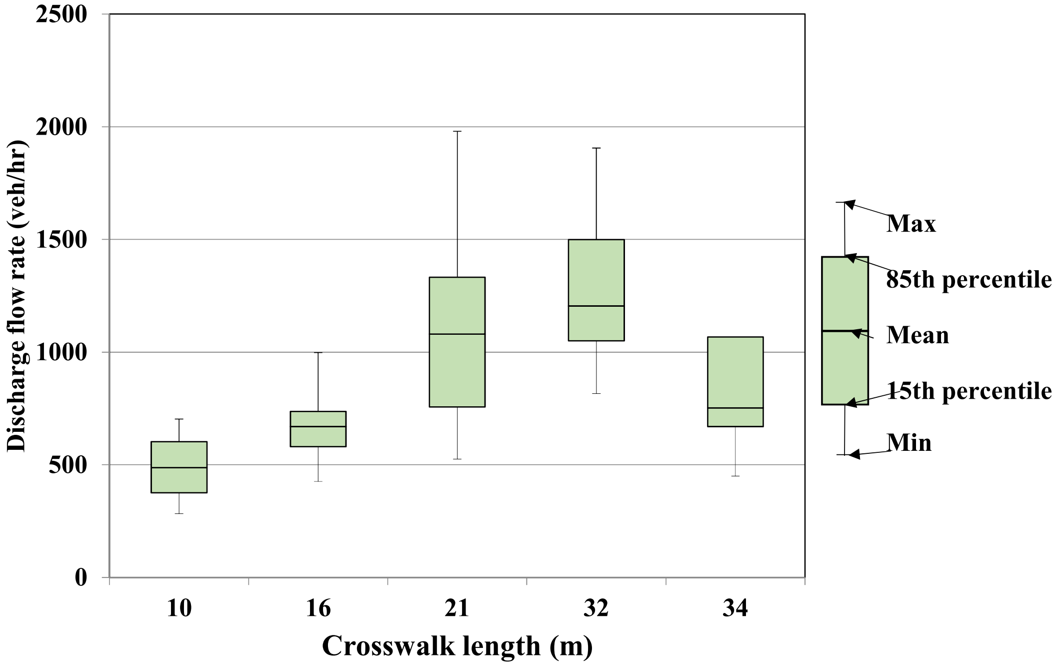


Fig.5.7 Variation of discharge flow rate with increasing crosswalk length

The correlation of possible influencing factors on left turning vehicles discharge flow rate is calculated to identify the significant influencing factors. **Table 5.2** shows the result of the correlation between influencing factors and discharge flow rate. As we observe from the result near-side and far-side pedestrian arrival rates, crosswalk length, stop line setback distance and crosswalk setback distance have a significant correlation with discharge flow rate, therefore considering those factors as the influencing factor of discharge flow rate estimation is reasonable.

Table 5.2 Correlation of influencing factors with discharge flow rate

Influencing factors	Correlation
Near-side pedestrian arrival rate rate(ped/sec)	-0.5655
Far-side pedestrian arrival rate (ped/sec)	-0.3744
Intersection angle	0.1002
Stop line set back distance (m)	-0.4378
Crosswalk set back distance (m)	0.5219
Crosswalk length (m)	0.2833

5.3.2. Discharge flow rate model result and discussion

Based on the trend of the observed discharge flow rates and by considering the significant influencing factors, the linear regression model is adopted for the discharge flow rate modeling as shown in Equation 5.2.

$$Q = (b_1 * q_n) + (b_2 * q_f) + (b_3 * l_c) + (b_4 * l_s) + (b_5 * L) + c \quad (5.2)$$

Where, b_1 to b_5 and c are coefficients of influencing factors, Q is the discharge flow rate (veh/hr), q_n is near-side pedestrian arrival rate (ped/hr), q_f is far-side pedestrian arrival rate (ped/hr), l_s is stop line setback distance (m), l_c is crosswalk set back distance and L is crosswalk length (m).

Table 5.3 Coefficients of discharge flow rate model

Variables	Coefficient	t-Stat	R^2
Intercept	1039	4.99	0.53 (n=97)
Near-side pedestrian arrival rate (ped/hr) q_n	-0.9449	-3.07	
Far-side pedestrian arrival rate (ped/hr) q_f	-1.291	-3.64	
Crosswalk set back distance (m) lc	16.86	2.88	
Stop line set back distance (m) ls	-18.80	-2.20	
Crosswalk length (m) L	12.36	3.06	

Table 5.3 shows the coefficients for the linear regression model of discharge flow rate. Since the observation show a linear trend of discharge flow rate correlated with influencing factors and for simplification reason a linear regression model is applied. Near-side and far-side pedestrian arrival rates have a negative impact to left turning vehicles discharge flow rate, when higher number of pedestrian's presence in the conflict-area the possibility of left turning vehicles to pass the conflict-area will be lower. Stop line set back distance has a negative impact on left turning vehicles discharge flow rate, when the stop line is far from the intersection the arriving time to the crosswalk will be long due to that there will be delay to pass the conflict-area and unused green time may increase. Consequently, with short discharge flow time the total number of left turning vehicles will be decrease. On the other hand, crosswalk set back distance and crosswalk length have a positive impact to discharge flow rate. This implies that when the crosswalk position is near to the intersection the left turning vehicles discharge flow rate will decrease. The reason is may be related with sight distance, left turning vehicles can easily notice the crossing pedestrians at the onset of green and they will approach with lower turning speed, due to that the number of turning vehicles per cycle will decrease. In addition, for shorter crosswalks left turning vehicles may wait for

longer time before they pass the conflict-area. Because, arriving pedestrians may presence on the conflict-area frequently due to short arriving time. In the model, crosswalk setback distance has higher influence on discharge flow rate compared with other variables. This implies that, even if we have lower pedestrian demand on the crosswalk, without proper crosswalk layout design the left turning vehicles discharge flow rate may decrease.

5.3.3. Validation of the discharge flow rate model

To validate the discharge flow rate model, the model estimation is compared with the observed discharge flow rates. As the validation shown in **Fig.5.8**, the proposed model can reasonably estimate the left turning vehicles discharge flow rate by considering the influence of pedestrians and different crosswalk layout. The MAPE values of each crosswalk are reasonably low, which implies that the model can estimate the observed discharge flow rate with lower error. However, the MAPE value of Mizuho crosswalk is comparatively higher, which is because of the small number of sample.

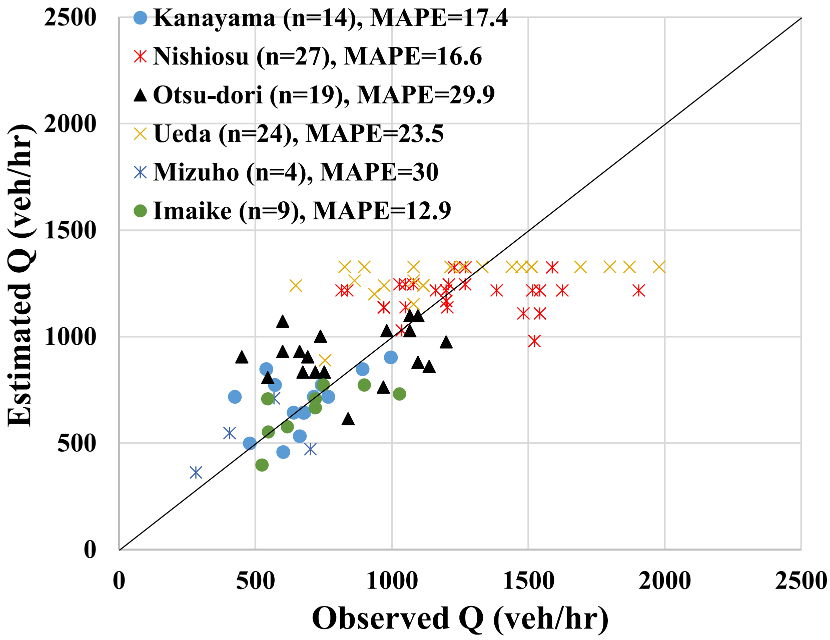


Fig.5.8 Left turning vehicles discharge flow rate model validation

5.4. Summary

In this chapter the left turning vehicles discharge flow rate is observed and then modeled considering the significant influencing factors. Left turning vehicles discharge time is defined as the expected time after the waiting pedestrian platoon clear from the conflict-area until the last queued left turning vehicle passes the edge of the conflict area along the left turning vehicles movement direction. The observed discharge flow rate distribution at different signalized intersections showed that the position and slope are different because of the variance in the number of arriving pedestrians per cycle and directional composition of arriving pedestrians. Therefore, the discharge flow rate is modeled by checking the correlation of possible influencing factors on left turning vehicles discharge flow rate to identify the significant influencing factors.

The discharge flow rate model revealed that, near-side and far-side pedestrian arrival rates have a negative impact to left turning vehicles discharge flow rate, when higher number of pedestrian's presence in the conflict-area the possibility of left turning vehicles to pass the conflict-area will be lower. Stop line set back distance has a negative impact on left turning vehicles discharge flow rate, when the stop line is far from the intersection the arriving time to the crosswalk will be long due to that there will be delay to pass the conflict-area and unused green time may increase. Consequently, with short discharge flow time the total number of left turning vehicles will be decrease. On the other hand, crosswalk set back distance and crosswalk length have a positive impact to discharge flow rate. This implies that when the crosswalk position is near to the intersection the left turning vehicles discharge flow rate will decrease. The reason is may be related with sight distance, left turning vehicles can easily notice the crossing pedestrians at the onset of green and they will approach with lower turning speed, due to that the number of turning vehicles per cycle will decrease. In addition, for shorter crosswalks left turning vehicles may wait for longer time before they pass the conflict-area. Because, arriving pedestrians may presence on the conflict-area frequently due to short arriving time.

Chapter 6

Left-turn Lane Capacity Estimation

6.1 Overview

In the previous chapters, the crossing pedestrian's characteristics and the left turning vehicles discharge flow conditions are investigated. Therefore, after understanding of the crossing pedestrians and left turning vehicles characteristics at the signalized intersections then the next procedure is to estimate the left turn lane capacity by combining the analysis results of chapter 4 and 5. The conceptual procedure of the proposed left turn lane capacity estimation method is discussed in chapter 3, on the proposed method the waiting pedestrian's impact considered as EPT, while the crosswalk layout condition and the arriving pedestrians impact considered in the discharge flow rate model. This chapter tries to explain the left turn lane capacity estimation by assigning some case study conditions. In addition, the validation and sensitivity of the proposed method will be checked. Finally, the application of the proposed method as an adjustment factor for pedestrian and crosswalk layout impact on left turn lane capacity is discussed.

6.2 Validation of the proposed method

6.2.1 Comparison with empirically observed left-turn lane capacity

To validate the estimation of proposed method it is compared with empirically observed capacity and then it is compared with estimation of some selected manuals. The empirically observed capacity is computed by selecting saturated cycles from the observed signalized crosswalks and recording the total number of left turning vehicles when they pass the conflict-area under the condition of crossing pedestrians within the green time. A total of 20 cycles found from the observed six signalized crosswalks which have a continuous left turning vehicles flow conditions.

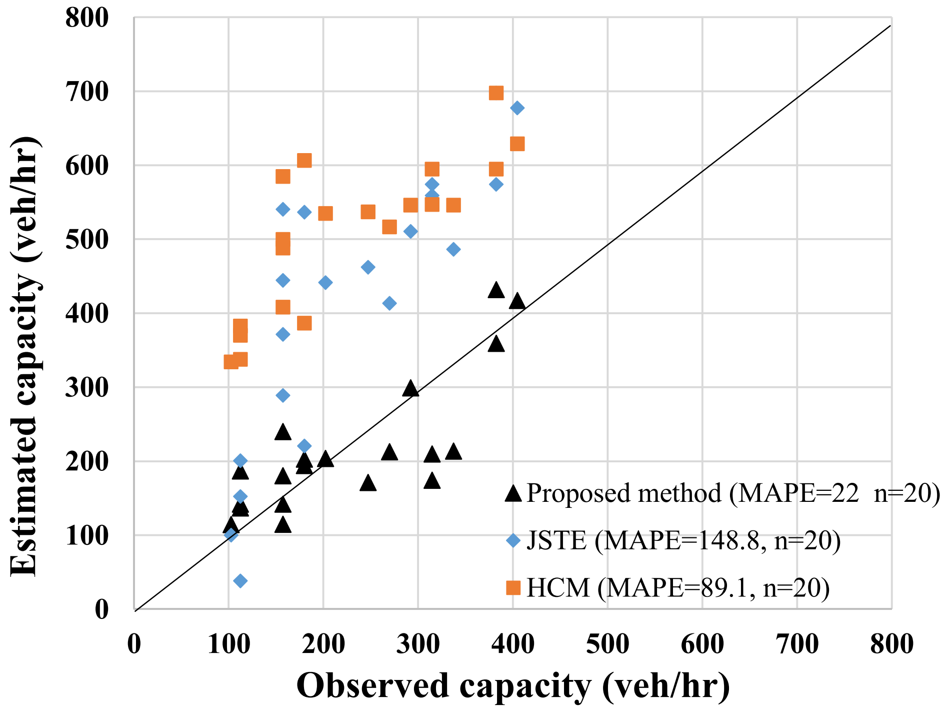


Fig 6.1 validation of capacity estimation

Fig 6.1 shows the comparison of the estimated capacity by the proposed method and the observed capacity at the saturated cycles using a 45-degree plot. As it is shown in the plot the plots are concentrated around the 45-degree line, which implies that the estimated capacity based on the proposed method can reasonably represent the observed capacity conditions under different pedestrian demand and signalized crosswalks. While the estimation by JSTE and HCM are far from the 45-degree line, which implies the estimation procedures may need some modifications to realistically represent the capacity estimation.

6.2.2 Comparison with existing manuals estimations

The proposed capacity estimation method results are compared with two existing capacity estimation procedures, i.e. Guide for Planning, Design and Traffic Signal Control of Japan (JSTE 2018 manual) and Highway capacity manual (HCM 2016). **Table 6.1** shows the summary of the comparison between the estimation methods. In addition, the estimation methods compared with observed capacity as a benchmark, the observed capacity calculated based on observed over saturated cycles of different crosswalks. The HCM method proposes adjustment factors based on the average pedestrian volume as the basic input and then the capacity is estimated by adjusting the base saturation flow rate. While the JSTE method similarly propose adjustment factors which are derived from simulation analysis to adjust the base saturation flow rate, However, the JSTE method consider crosswalk length as an additional factor in the adjustment factors proposals. Based on the comparison results, JSTE and HCM methods overestimate left turn lane capacity, while proposed method capacity estimation is reasonable compared with the others. The reason behind this variation in capacity estimation is related with;

- ✓ Pedestrian platoon distribution (proposed method) vs. number of pedestrian (HCM and JSTE) based estimation.
- ✓ Effect of different bi-directional pedestrian demand ratio.
- ✓ Base saturation flow rate of 1800 pcu/hr (JSTE and HCM) vs. 900 -1500 pcu/hr (proposed method), reduction due to impact of crosswalk layout conditions.

Table 6.1 Comparison of left turn lane capacity estimations

Cycle No.	Crosswalk	Near-side pedestrian volume (Ped/hr)	Far-side pedestrian volume (Ped/hr)	Observed capacity (veh/hr)	Proposed method estimation (veh/hr)	JSTE estimation (veh/hr)	HCM estimation (veh/hr)
1	Nishiosu	45	90	248	171	537	462
2	Nishiosu	0	45	315	174	547	559
3	Nishiosu	45	45	293	299	546	510

4	Nishiosu	68	113	270	213	516	413
5	Nishiosu	23	90	338	213	546	486
				MAPE	**42.65**	**45.7**	**39.6**
				RMSE	**94.35**	**247.2**	**197.7**
6	Otsu-dori	90	473	113	187	338	38
7	Otsu-dori	90	225	158	180	408	289
8	Otsu-dori	68	315	180	194	386	220
9	Otsu-dori	113	338	113	136	370	152
				MAPE	**19.24**	**62.7**	**71,5**
				RMSE	**41.1**	**235.6**	**80.5**
10	Ueda	0	135	315	209	595	574
11	Ueda	0	0	383	432	698	698
12	Ueda	0	23	405	417	629	677
13	Ueda	68	68	383	359	595	574
				MAPE	**17.8**	**40.8**	**40.9**
				RMSE	**59.6**	**261.0**	**263.1**
14	Imaike	135	180	158	141	488	444
15	Imaike	90	135	203	203	535	441
16	Imaike	180	113	158	114	500	371
				MAPE	**16.5**	**66.1**	**58.7**
				RMSE	**26.6**	**334.8**	**248.3**
				Total MAPE	24.3	56.6	51.0
				Total RMSE	61.6	287.5	221.4

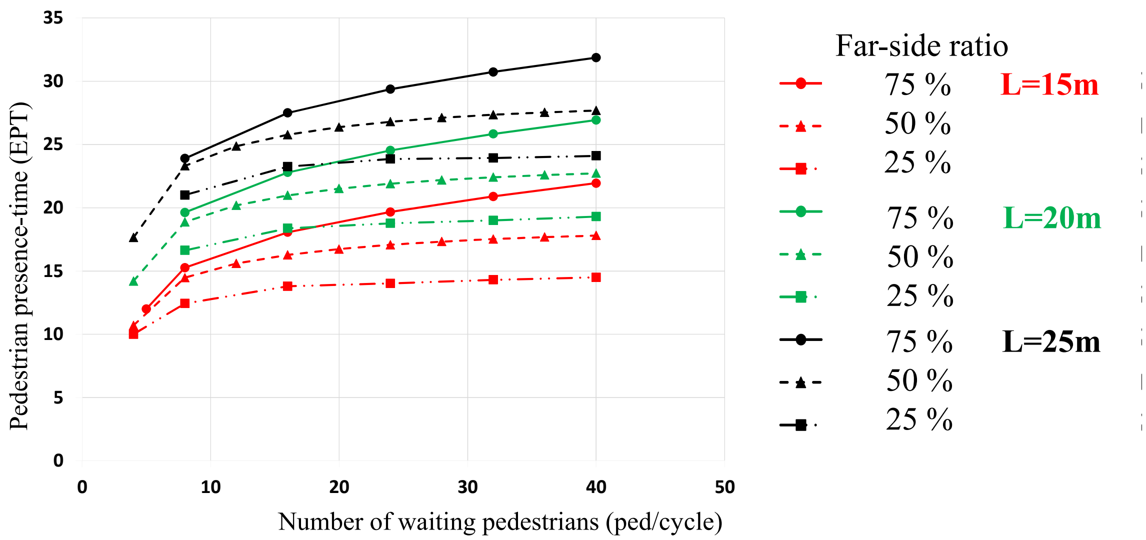
Additionally, in order to quantitatively evaluate the relative margin of estimation errors, Mean Absolute Percentage Error (MAPE) and Root Mean Squared Error (RMSE) are computed for all methods. MAPE are measures that indicate about the mean of the dispersion between predicted and observed value, for each one with the estimation model. While RMSE is a measure of model error, for both cases lower value represents a better prediction model. As it is shown in **Table 6.1**, for all crosswalks the RMSE and MAPE values of the proposed method is lower than that of JSTE and HCM method, this revealed that the proposed method contemplates additional relevant influencing factors to realistically estimate left turn lane capacity under different pedestrian demand and crosswalk layout conditions.

6.3 Sensitivity of the proposed method

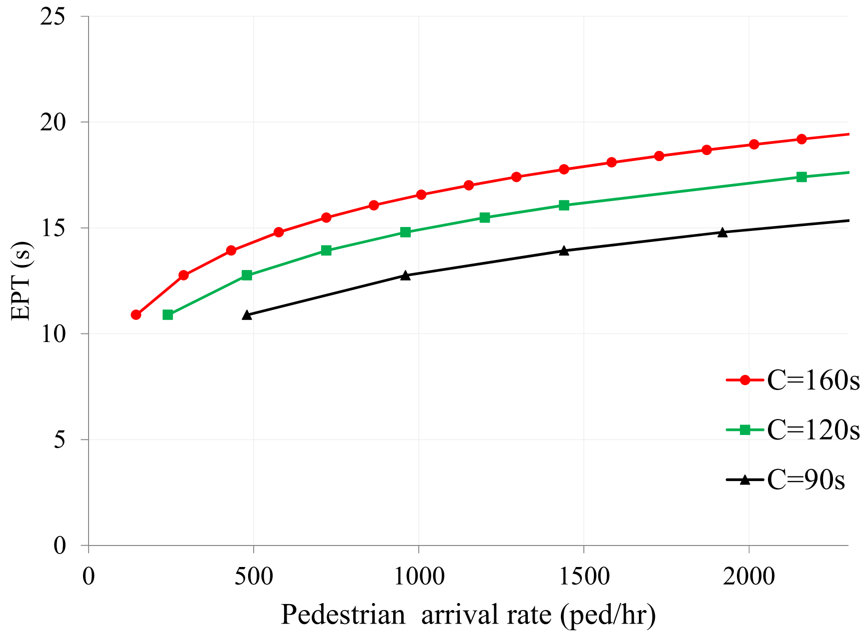
Chapter 4 and 5 tried to summarize the influence of crossing pedestrians and geometric layout on the left turning vehicles movement, accordingly their influence on the left turn lane capacity can be generalized using Equation (3.6). The two main input models for the proposed capacity estimation equation are EPT and discharge flow (Q).

Sensitivity of EPT

The influencing time interval of waiting pedestrians on the left turning vehicles is generalized by EPT as it is discussed in chapter 4, the EPT can be influenced by change in bi-directional pedestrian flow and crosswalk length. **Fig.6.2** shows the sensitivity of the EPT for change in bi-directional pedestrian flow and crosswalk length for the case of G=65s and PG=60s, as it is noticed from **Fig.6.2** (a) EPT has an increasing trend with increase in crosswalk length, that implies the impact of long crossing time for far-side pedestrians and higher far-side pedestrian ratio resulted increasing in EPT, because with increase in pedestrian demand the far-side pedestrians platoon distribution and crossing time will increase. **Fig 6.2 (b)** also shows the sensitivity of EPT to change in cycle length for the case of L=15m, G=65s, PG=60s and far-side ratio=50/50, when the cycle length increase EPT shows an increasing trend under the same pedestrian arrival rate.



a) Sensitivity of EPT for far-side ratio and L



b) Sensitivity of EPT for cycle length

Fig.6.2 Sensitivity of EPT

The proposed capacity estimation method has six basic input parameters i.e. crosswalk length, pedestrian demand, bi-directional pedestrian demand ratio, crosswalk set back distance, stop line set back distance and signal setting. By changing different input values of the basic components the sensitivity of the proposed method can be computed. **Table 6.2** shows the summary of input case study conditions for the sensitivity analysis of the proposed capacity estimation method and **Fig 6.3** shows the flow chart of the proposed method capacity estimation. The capacity estimation is computed by combining different values of the case study conditions.

Table 6.2 Case study input conditions

Variable	Conditions
Crosswalk length (m)	15, 20, 25, 30, 35
Pedestrian demand (ped/hr)	0 – 1000 ped/hr
Near-side/Far-side ratio	100/0, 75/25, 50/50, 25/75,0/100
Crosswalk setback distance (m)	5, 10, 15
Stop line setback distance (m)	5, 10, 15, 20
Signal setting (s)	G=65s, 60s, 55s, EPR= 5s, 10s, 15s, C= 90s, 120s, 160s

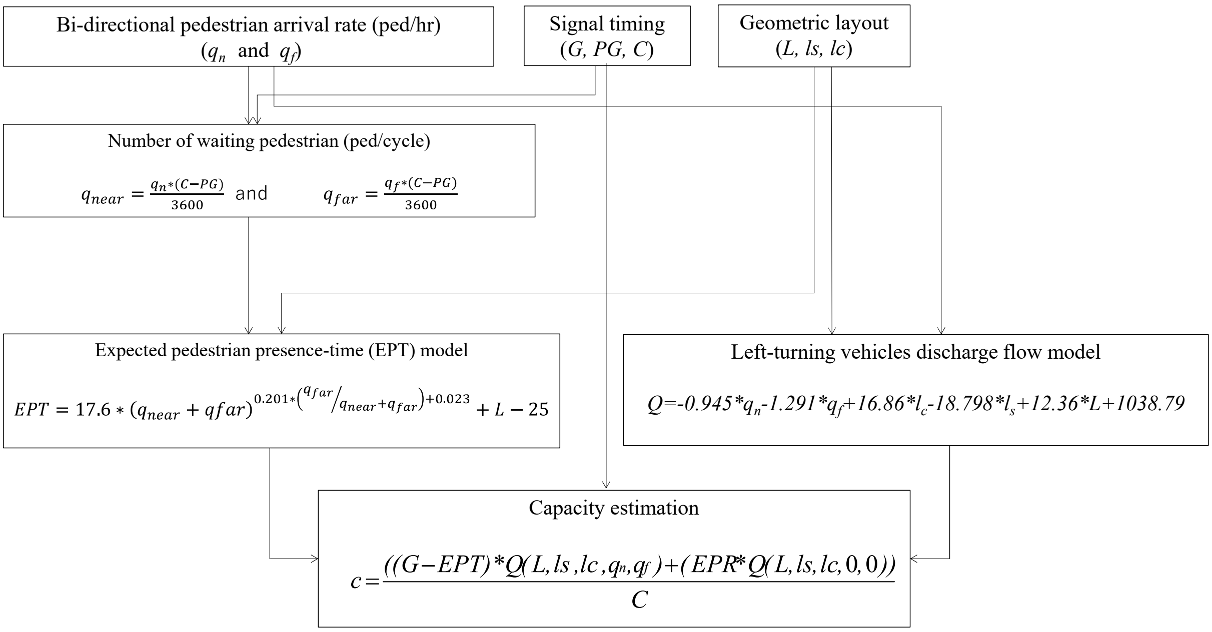


Fig 6.3 Flow chart of proposed method capacity estimation

6.3.1 Sensitivity for crosswalk layout

The proposed method has three crosswalk layout parameters i.e. crosswalk length, crosswalk set back distance and stop line set back distance. Crosswalk length may simultaneously influence the crossing pedestrian's behavior and the left turning vehicles movement. Based on the models proposed in chapter 4 and 5 the crosswalk length has negative impact on the pedestrian presence probability and it has a positive impact on the discharge flow rate of left turning vehicles. While the crosswalk set back distance and stop line set back distance have an impact on the left turning vehicles movement. **Fig. 6.4** shows the variation in left turn lane capacity at different crosswalk length conditions with increasing pedestrian demand under the same condition of C=160, EPR=5s, G=65, l_s=5m, l_c=5m and Near/Far ratio =50/50.

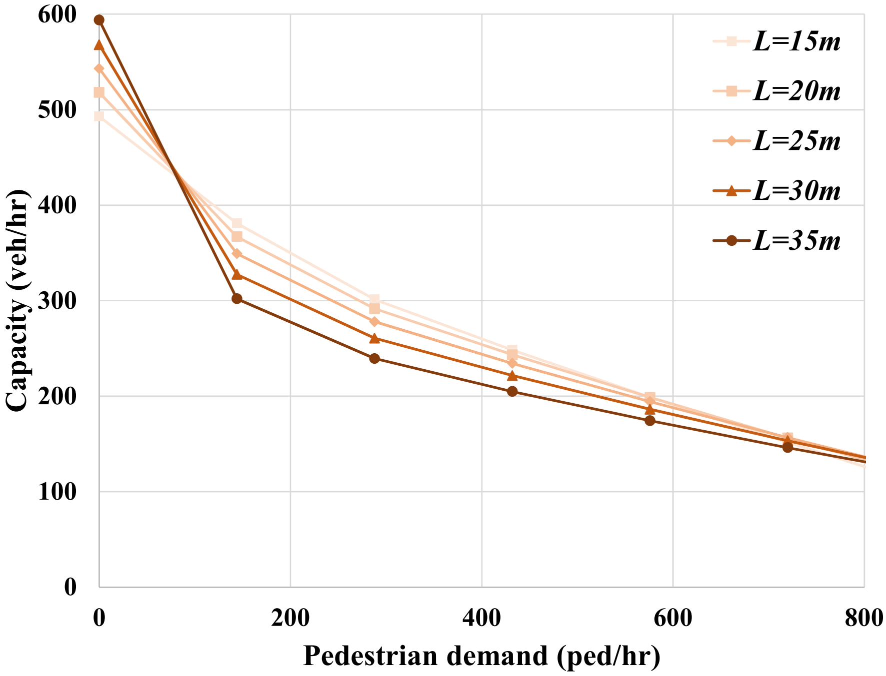


Fig 6.4 Sensitivity of capacity for crosswalk length

The estimation curves plotted in **Fig 6.4** can be divided into two portions based on the order of the increase in crosswalk length conditions. The first portion is from 0 ped/hr up to 100 ped/hr, and in this portion longer crosswalks have higher capacity, because the possibility to observe bi-direction pedestrian flow is lower and even if there are too small number of crossing pedestrians left turning vehicles may accept short gaps since the conflict-area will be wide at long crosswalks. In addition, since crosswalk length has positive impact on discharge flow rate at 0 ped/hr case longer crosswalks will have higher discharge flow rate, resulting in higher capacity. In the second portion from after 100 ped/hr up to 800 ped/hr shorter crosswalks have higher capacity, because the possibility to have unbalanced bi-directional pedestrian flow is higher within this interval, so that due to the longer crossing time of the far-side pedestrians the left turning vehicles may wait for long time by giving priority for crossing pedestrians in case of longer crosswalks. Due to the limitation of the observed pedestrian demand range less than 800 ped/hr, it is not possible to plot the curves after 800 ped/hr. For greater than 800 ped/hr cases, it is expected that due to the extremely higher pedestrian flow condition there will be continuous flow of pedestrian along the crosswalk and it is very difficult for left turning vehicles to find acceptable gaps, vehicles only can get a chance to pass the conflict area in the clearance time after the end of PFG.

The sensitivity of the left turn lane capacity for change in crosswalk set back distance is shown in **Fig 6.5** under L=20m, Near/Far ratio= 50/50, EPR=5s, G=65s, C=160s. The increase in crosswalk set back distance by 5m will raise the left turn lane capacity approximately by 12% under all pedestrian demand conditions. This implies that under the same pedestrian demand condition longer crosswalk set back distance may result higher left turn lane capacity, the possible reason is that when the crosswalk position is near to the intersection (short crosswalk set back distance) the left turning vehicles discharge flow rate will decrease. Because of the sight distance range left turning vehicles can easily notice the crossing pedestrians at the onset of green and they will approach with lower turning speed, due to that the number of turning vehicles per cycle may decrease.

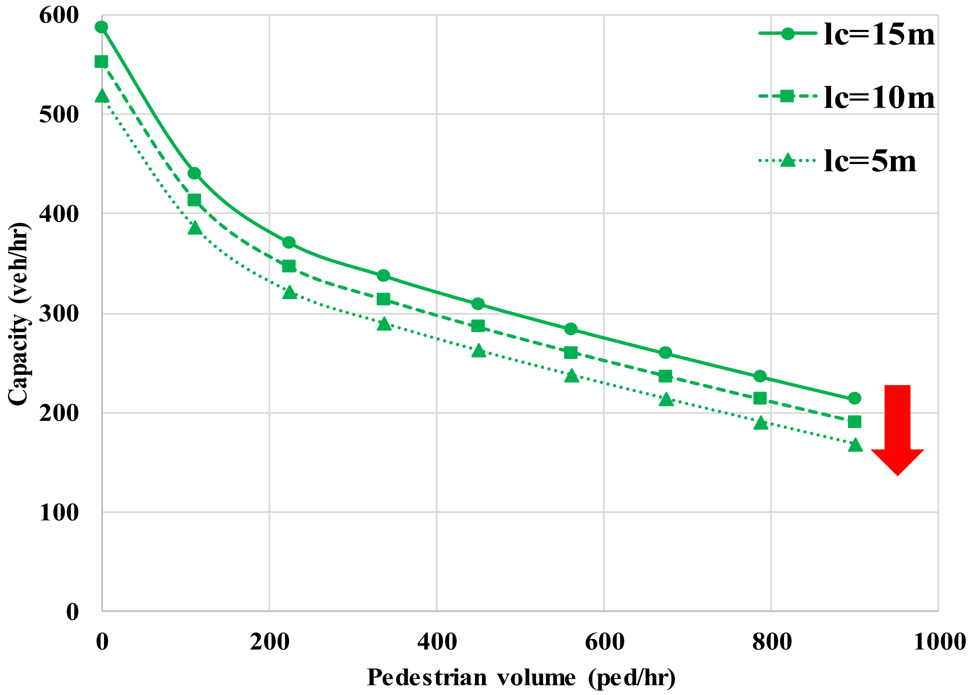


Fig 6.5 Sensitivity of capacity for crosswalk setback distance

Fig. 6.6 shows the sensitivity of the estimated capacity for stop line set back distance. The increase in stop line set back distance by 5m will decrease the left turn lane capacity approximately by 17% under all pedestrian demand conditions. The possible reason is related with the arrival time of left turning vehicles from the stop line to the subject crosswalk. For longer stop line set back distance turning vehicles may take long time before they pass the conflict-area.

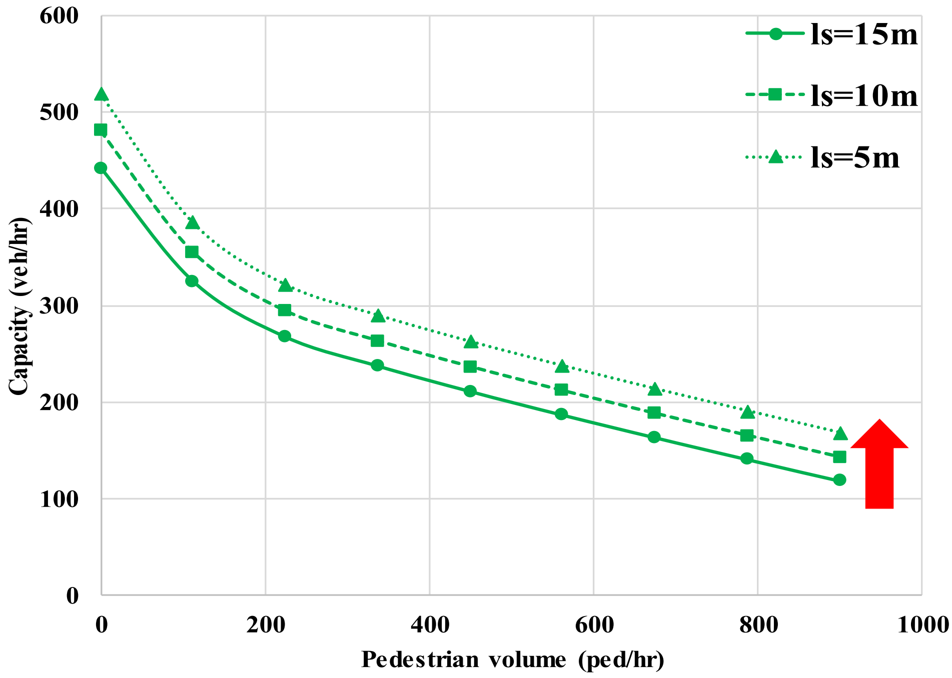


Fig 6.6 Sensitivity of capacity for stop line setback distance

Fig 6.7 shows one example of the sensitivity of capacity estimation under the combined change in crosswalk setback distance and stop line set back distance with the following common conditions, L=25m, Near/Far ratio= 50/50, EPR=5s, G=65s, C=160s and pedestrian demand=450 ped/sec.

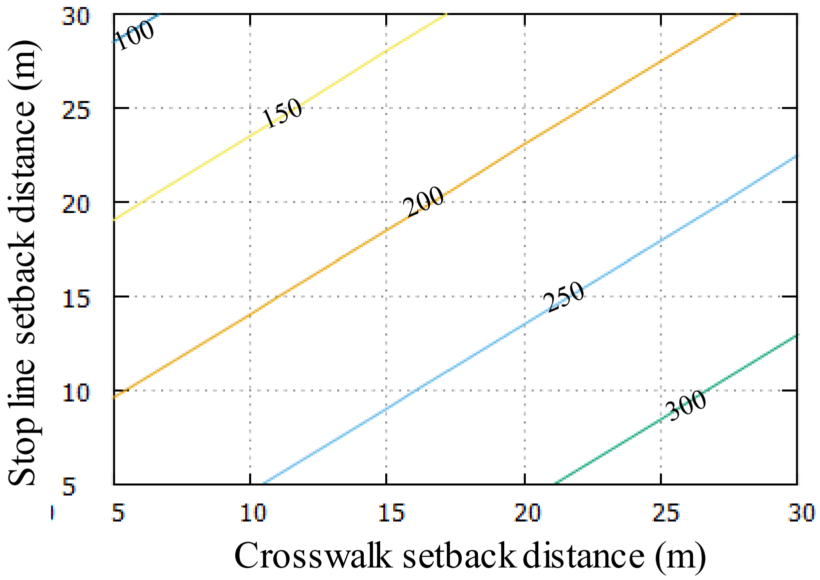


Fig 6.7 Sensitivity of capacity for combined change in crosswalk and stop line setback distance

As it is shown in **Fig. 6.7** the left turn lane capacity can be varied under different combination of crosswalk and stop line setback distances. Increasing the setback distance of the crosswalk without changing the position of stop line will result the left turn lane capacity to increase, while increasing the stop line distance may have a decreasing effect on the left turn lane capacity. This implies that the balanced location of the stop line and crosswalk set back distance must be checked at the planning stage of signalized crosswalks to minimize the impact on left turn lane capacity.

6.3.2 Sensitivity for bi-directional pedestrian demand ratio

The other main influencing factor in the proposed capacity estimation method is the bi-directional pedestrian demand. **Fig. 6.8** shows a case study estimation of left turn lane capacity under different near-side/far-side pedestrian flow proportion cases, with increasing pedestrian volume. For all conditions, when pedestrian demand increases the left turn lane capacity has a decreasing trend. These implies that the higher the crossing pedestrian's presence on the crosswalk, there will be a significant impact on left turning vehicles discharge flow, so that the application of concurrent signal timing may not be favorable at higher pedestrian demand conditions.

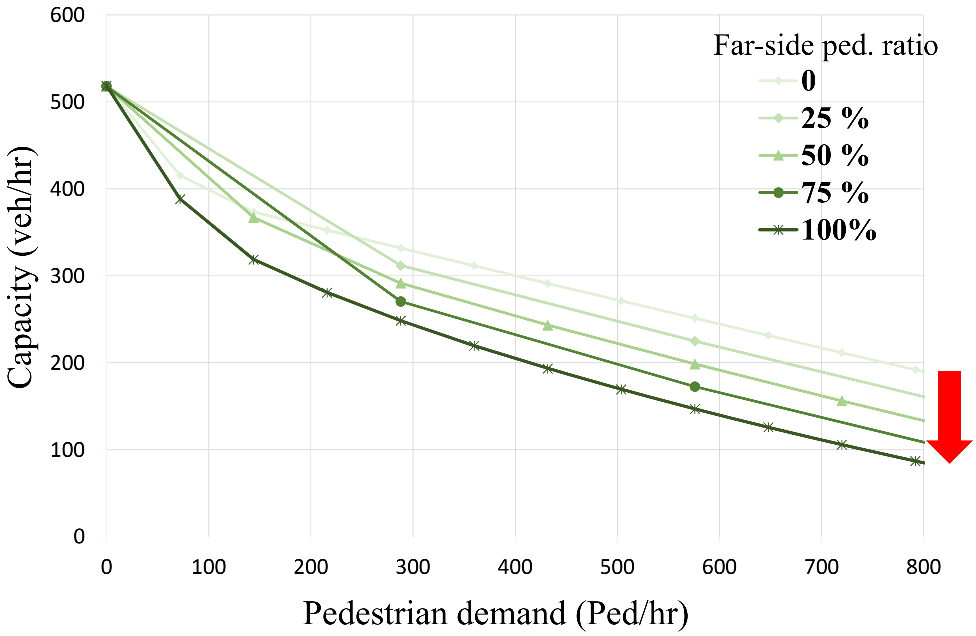


Fig.6.8 sensitivity of capacity for far-side pedestrian ratio

As shown in **Fig.6.8,** the distributions of the curves have decreasing trend with increase in far-side pedestrian ratio. This implies that, in bi-directional pedestrian flow the opposing higher pedestrian demand will affect the crossing pedestrian's movement, thus left turning vehicles waiting time will increase. Consequently, left turn lane capacity may decrease; from the curves we can notice that there will be around 24% reduction of capacity due to the increase in far-side pedestrian ratio.

Fig. 6.9 shows the sensitivity of the estimated left turn lane capacity for bi-directional pedestrian demand with crosswalk set back distance of 20m, crosswalk length of 30m and stop line set back distance of 5m. With increase in both near-side and far-side pedestrian demand the left turn capacity shows a decreasing trend, since the pedestrian presence-time in the conflict area will be higher for higher pedestrian demand situations consequently the left tuning vehicles may have long waiting time before passing the crosswalk.

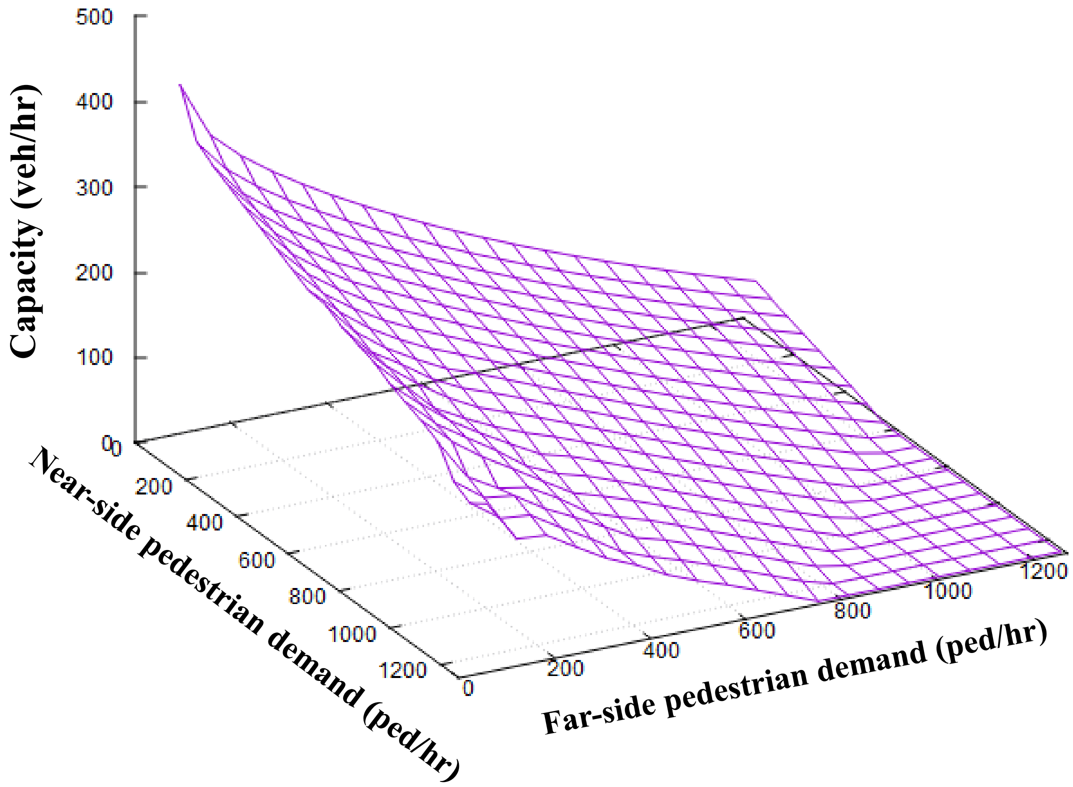


Fig 6.9 Sensitivity of capacity for bi-directional pedestrian demand

From **Fig. 6.9** we can notice that the zero capacity starting pedestrian demand point for near-side direction and far-side direction is different i.e. 600ped/hr and 800 ped/hr, respectively. This implies that the far-side pedestrian flow has higher influence on the left turn lane capacity estimation due to the longer crossing time of far-side pedestrians to pass the conflict area of the subject crosswalk and due to the opposing higher pedestrian demand circumstance; capacity may deteriorate even at lower near-side pedestrian demand conditions.

6.3.3 Sensitivity for signal timing

Sensitivity to EPR

There are different types of signal timing phase strategies for consideration of pedestrian-vehicle interaction, such as concurrent pedestrian phase, Exclusive Pedestrian Phase, Leading Pedestrian Interval and Early Pedestrian Red time. The performance of each strategy is dependent on the proper consideration of pedestrian-vehicle interaction at the signalized crosswalk. In this study, left turn lane capacity is studied under the case of concurrent signal timing design. Thus, settings of the pedestrian green time and the effective green time will influence the capacity estimation. Particularly, the setting of the exclusive pedestrian red time (EPR) may have significant impact on the discharge flow characteristics of left turning vehicles, because EPR is the clearance time interval for turning vehicles without the direct influence of crossing pedestrians. **Fig. 6.10** shows the sensitivity of the capacity estimation under different conditions of signal timing settings (EPR and G) and with common settings of Near/far ped. Ratio =50/50, L=20m, ls=5m, lc=5m, C=160.

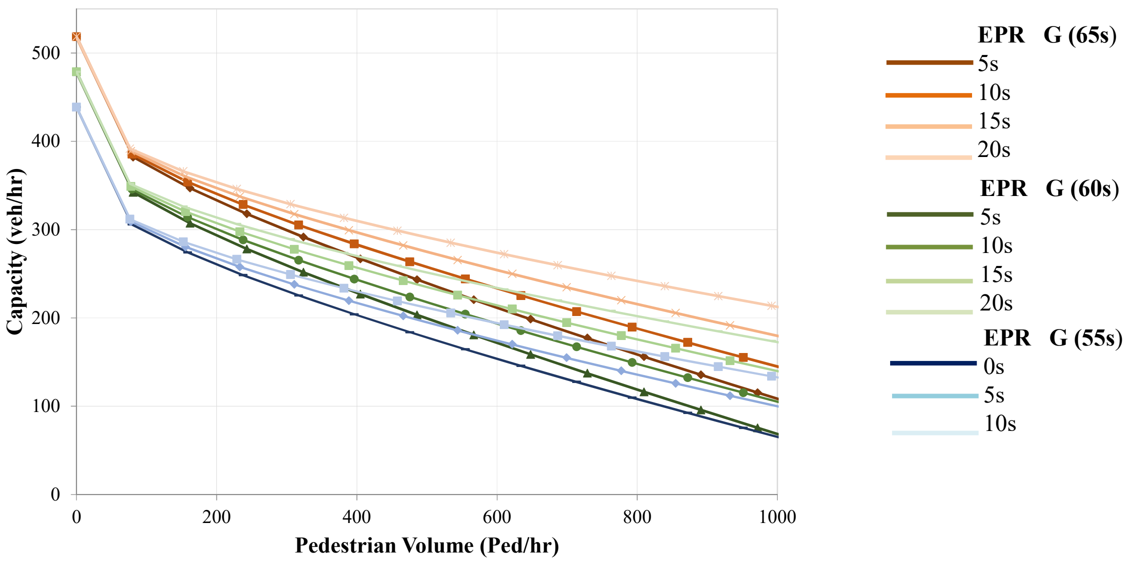


Fig 6.10 Sensitivity of capacity for signal timing settings

As shown in **Fig 6.10** when G increases the left turn lane capacities also have higher increasing possibility, since the left turning vehicles will have long available green time for possible discharge flow. In the same way, for short EPR interval cases the estimated capacity is lower due to significant impact of crossing pedestrians with higher blockage probability for left turning vehicles.

Sensitivity to Cycle length

The other signal timing parameter in the proposed method is cycle length. **Fig.6.11** shows the sensitivity of left turn lane capacity for change in cycle length for the case of G=65s, PG=60s, l_s=5m, l_c=5m, L=20m and Near/far ratio=50/50. As it is noticed from **Fig 6.11,** with increase in cycle length the estimated capacity showed a decreasing trend; since the available green time per hour will decrease with increase in cycle length then left turning vehicles have higher turning possibilities in shorter cycle lengths.

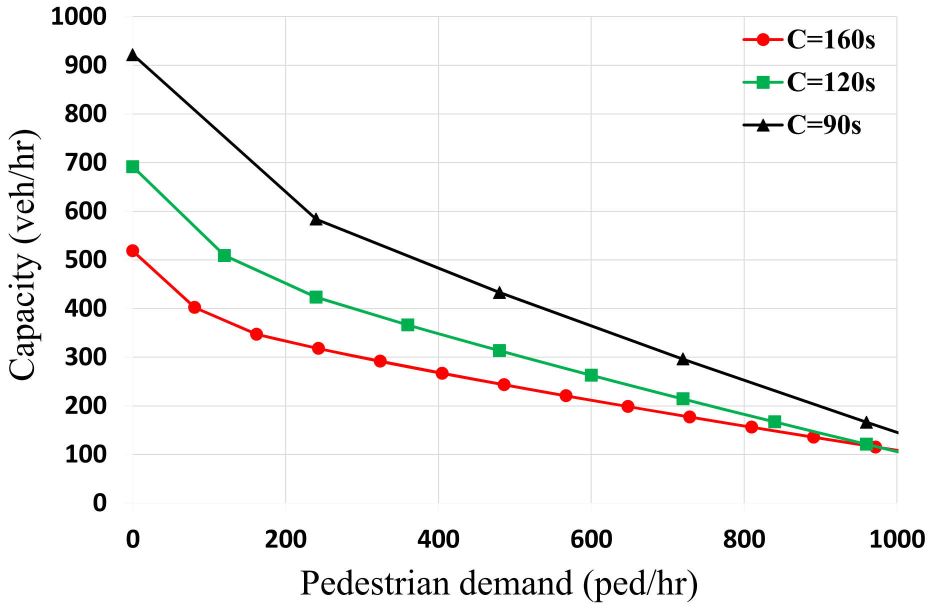


Fig. 6.11 Sensitivity of capacity for cycle length

Fig. 6.12 also show the sensitivity of capacity for cycle length with the same green ratio, the plotted curves in **Fig. 6.12** have the same input conditions of EPR=5s, l_s=5m, l_c=5m, L=20m, Near/Far ped. ratio =50/50 and green ratio of 0.4, however even if they have the same green ratio and basic input parameters the estimated capacity showed different results for different cycle lengths. The estimated capacity has a decreasing trend with decrease in cycle length. Because, the pedestrian arrival rate will increase for short green time (the case of C=90s) at the same pedestrian demand case with the others, that will result in a decrease in discharge flow rate of left turning vehicles. The difference in estimated capacity is minimized with increase in pedestrian demand; this implies that in the case of higher pedestrian demand regardless of the cycle length the available green time will be occupied by crossing pedestrians and left turning vehicles will use only the EPR time interval.

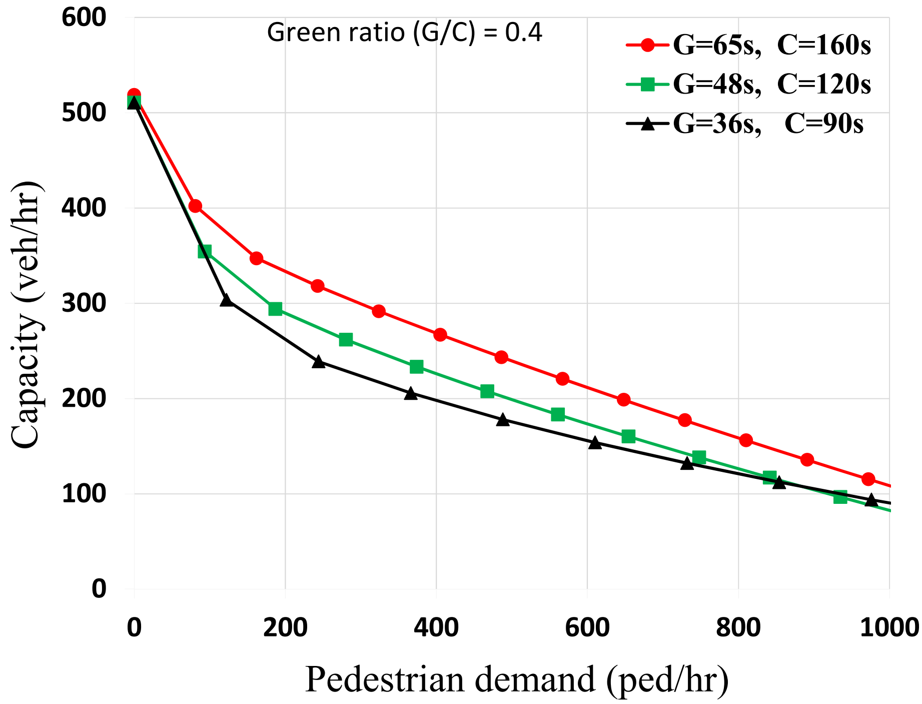


Fig.6.12. Sensitivity of capacity for cycle length with the same green ratio

6.4 Application of the proposed method

6.4.1 Adjustment factors

Most researches and existing manuals recommend different types of adjustment factors to consider the impact of influencing factors on the base traffic flow. For example, the highway capacity manual (HCM 2016) considers that there are 11 factors that affect saturation flow estimates. These factors are: lane width, heavy vehicles in traffic stream, approach grade, existence of a parking lane and parking activity adjacent to lane group, blocking effect of local buses that stop within intersection area, area type, lane utilization, left-turn vehicle presence in a lane group, right-turn vehicle presence in a lane group, pedestrian for left-turn groups, and pedestrian for right-turn groups. The accuracy of the saturation flow rate estimation and capacity estimation may highly depend on the proper consideration of the influencing factors. Therefore, to have realistic capacity estimation the effective computation of adjustment factors of the influencing factors is necessary.

6.4.2 Adjustment factors to consider pedestrian and crosswalk layout influence on left turn lane capacity

As it is discussed in detail from chapter 3 up to chapter 5 about the empirical observation and modeling of crossing pedestrians and left turning vehicles characteristics, in this study the impact of bi-directional pedestrian flow and the crosswalk layout on the left turning vehicles discharge flow is investigated, so that the realistic left turn lane capacity by considering the presence of bi-directional pedestrian flow at different crosswalk layout conditions can be proposed. To apply the result of the proposed capacity estimation procedure in the performance evaluation procedure of signalized intersections and in the signal timing design procedures, the development of adjustment factors is essential. The adjustment factors are suggested based on the following procedures by considering the analysis discussed for crossing pedestrians and left turning vehicles in chapter 4 and 5, respectively.

Step 1: Estimation of the pedestrian presence probability (PPP) based on the modified PPP model. As it is discussed in chapter 4, the pedestrian presence along the crosswalk by elapsed time for different crosswalk layout conditions can be estimated by the developed PPP model.

Step 2: Computation of waiting pedestrian presence-time at the conflict-area of signalized crosswalk by utilizing the PPP model. As it is summarized in chapter 4, by setting the time thresholds for first/last pedestrians in the bi-directional waiting pedestrian's flows occupancy of the conflict-area by the bi-directional waiting pedestrian group movements can be computed.

Step 3: The discharge flow rate of the left turning vehicles by considering the presence of bi-directional arriving pedestrians at different crosswalk layout conditions can be estimated based on the discharge flow model proposed in chapter 5.

Step 4: The left turn lane capacity can be estimated by combining the estimated waiting pedestrian presence-time and the discharge flow of left turning vehicles, the basic equation is introduced in chapter 3. In the proposed left turn lane capacity estimation equation, the capacity that is influenced by crossing pedestrians and without the influence of crossing pedestrians must be estimated separately and finally add up together to have the total left turn lane capacity.

Step 5: Adjustment factors are computed using Equation 6.1, the proportional change in estimated capacity under crossing bi-directional pedestrian influence is compared with the base capacity without crossing pedestrians influence.

$$f_p = 1 - \left(\frac{\left(\frac{G * Q(0, X_c)}{C} \right) - \left(\frac{(G - EPT) * Q(X_p, X_C)}{C} \right)}{\left(\frac{G * Q(0, X_C)}{C} \right)} \right) \tag{6.1}$$

Where, G is left turning vehicle green time (s), EPT is expected waiting pedestrian-presence time (s), Q is discharge flow rate (veh/hr), Xp is the influence of pedestrians, Xc is the influence of crosswalk layout and C is cycle length (s).

After following the above steps, the adjustment factors to consider the impact of bi-directional pedestrian flow and crosswalk layout to estimate the left turn lane capacity can be generated. **Fig 6.13** shows one example of proposed adjustment factors for 20m crosswalk length, 5m crosswalk setback distance and 5m stop line setback distance conditions.

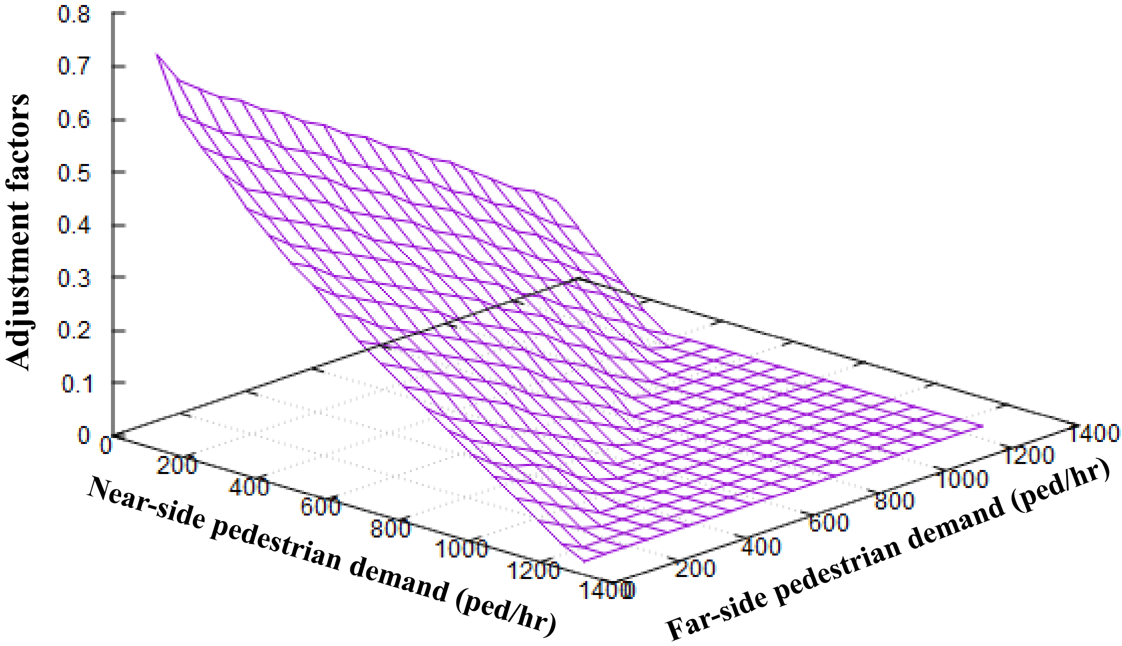
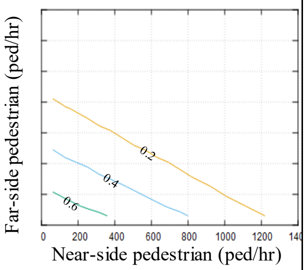
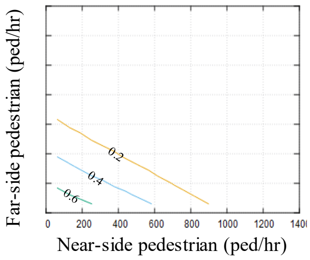
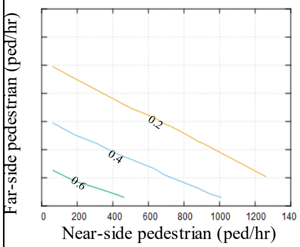
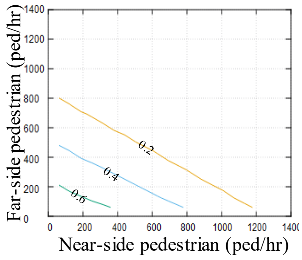
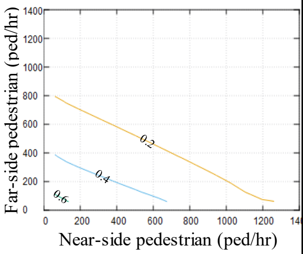
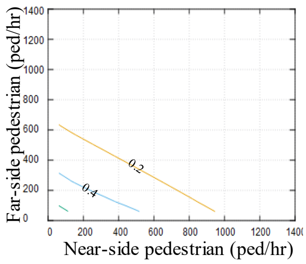
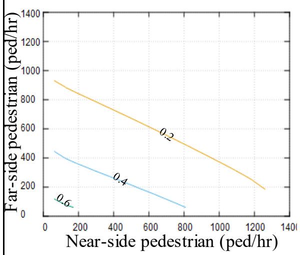
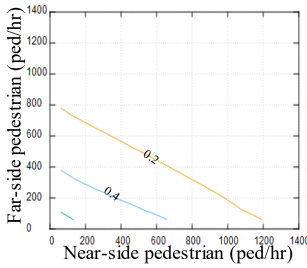


Fig. 6.13 Adjustment factors for bi-directional pedestrian flow

Table 6.3 summarize the contour plots of the adjustment factors under selected cases of different crosswalk length, crosswalk set back distance and stop line set back distance situations of the bi-directional pedestrian flows with assigned C=160s, G=65s and EPR=5s signal timing conditions. Using the table, we can easily pick the required adjustment factor to contemplate the impact of bi-directional pedestrian flow and crosswalk layout in the left turn lane capacity estimation, consequently the signalized intersections performance evaluation and signal timing design procedures will have realistic estimated left turn lane capacity values.

Table 6.3 Contour plots of adjustment factors

Cross walk length	Crosswalk setback = **5m**		Crosswalk setback = **20m**	
	Stop line setback = **5m**	Stop line setback = **20m**	Stop line setback = **5m**	Stop line setback = **20m**
15 m				
25 m				

In general, the contour plots show that adjustment factors have a decreasing trend with increase in bi-directional pedestrian demand and we can easily select the possible adjustment factor for bi-directional pedestrian demand situation under different crosswalk layout conditions.

Base capacity

The combination of the base capacity and adjustment factors are used to estimate the capacity under the influence of bi-directional pedestrian demand and crosswalk layout. The base capacity is computed by setting the bi-directional pedestrian demand as zero in the proposed capacity estimation equation (Equation 3.6) as shown in equation 6.2. After the estimation of the adjustment factors, the base capacity is multiplied by the adjustment factors to estimate the left turn lane capacity under different bi-directional pedestrian demand and crosswalk layout conditions.

$$c_b = \frac{(G * Q(0, X_c))}{C} \tag{6.2}$$

Where, c_b is base capacity (veh/hr), G is left turning vehicle green time (s), Q is discharge flow rate (veh/hr), Xc is the influence of crosswalk layout and C is cycle length (s).

6.4.3 Adjustment factor equation

For simplified application of the proposed method it is better to generalize the adjustment factor estimation by proposing an equation. Thus, adjustment factor (f) equation is proposed to consider the impact of bi-directional pedestrian flows and crosswalk layout, based on the results of the proposed capacity estimation procedures. Accordingly, practitioners can apply the proposed left turn lane capacity estimation method by substituting the influencing factors in the generalized proposed equation and multiplying with base discharge flow rates. Different equations are checked to fit the results of the proposed method estimation values and then the best fitted option is adopted. Equation 6.3 and Equation 6.4 shows the proposed equations as linear and exponential curves, respectively. The adjustment factor (f) is also computed using equation 6.5. The coefficients of the proposed equation are shown in **Table 6.4**. and **Table 6.5**.

Option 1

$$c = a_1 * q_n + a_2 * q_f + a_3 * l_c + a_4 * l_s + a_5 * L + a_6 * \left(\frac{EPR}{G}\right) + a_7 * \left(\frac{G}{C}\right) + b \tag{6.3}$$

Option 2

$$c = b * e^{\left(a_1 * q_n + a_2 * q_f + a_3 * l_c + a_4 * l_s + a_5 * L + a_6 * \left(\frac{EPR}{G}\right) + a_7 * \left(\frac{G}{C}\right)\right)} \tag{6.4}$$

$$f = 1 - \left(\frac{c(0, X_c, X_s) - c(X_p, X_c, X_s)}{c(0, X_c, X_s)}\right) \tag{6.5}$$

Where, c is left-turn lane capacity (veh/hr), f is adjustment factor, q_n is near-side pedestrian arrival rate (ped/hr), q_f is far-side pedestrian arrival rate (ped/hr), l_c is crosswalk setback distance (m), l_s is stop line set back distance (m), L is crosswalk length (m), EPR is exclusive pedestrian red time (s), C is cycle length (s), X_c is impact of crosswalk layout, X_s is impact of signal timing, X_p is impact of pedestrian demand and G is left turning vehicles green time (s).

Fig 6.14 shows the plots of the estimated capacity for different cycle lengths based on the proposed method by combining the different crosswalk layout, bi-directional pedestrian demand and signal timing cases. From the plot of the proposed capacity estimation and the pattern on the sensitivity analysis we can notice that the plotted curves have one bending point moving vertically under different input conditions, and this bending point is shifting horizontally with decrease in cycle length. The threshold value of option 1 equation is estimated by checking the fittings of the two lines at different threshold locations and the threshold location that resulted better fitted lines is selected i.e. threshold value which resulted a lower estimation error for the trend lines is selected.

For different cycle length the threshold value location is shifted horizontally, **Fig. 6.16** shows the location of the threshold points (bending points) for different cycle length cases. The bending points (intersection of the two slope lines) are shifting horizontality with decrease in cycle length. Consequently, the threshold pedestrian demand for the piecewise linear equation is correlated with cycle length(C) as:

$$\text{Pedestrian demand (ped/hr)} = -2.33*C + 528.5$$

Thus, the option-1 linear equation can be estimated by dividing the plots in to two piecewise linear equations. While option-2 equation is one continuous curve equation by combining all points.

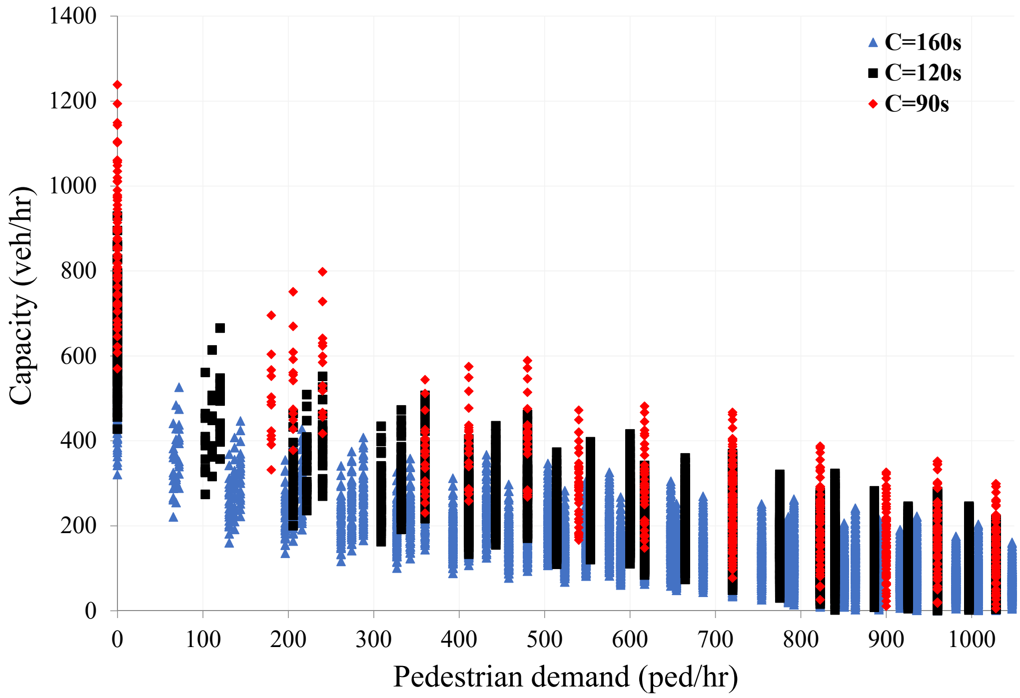


Fig 6.14 Proposed method capacity estimation

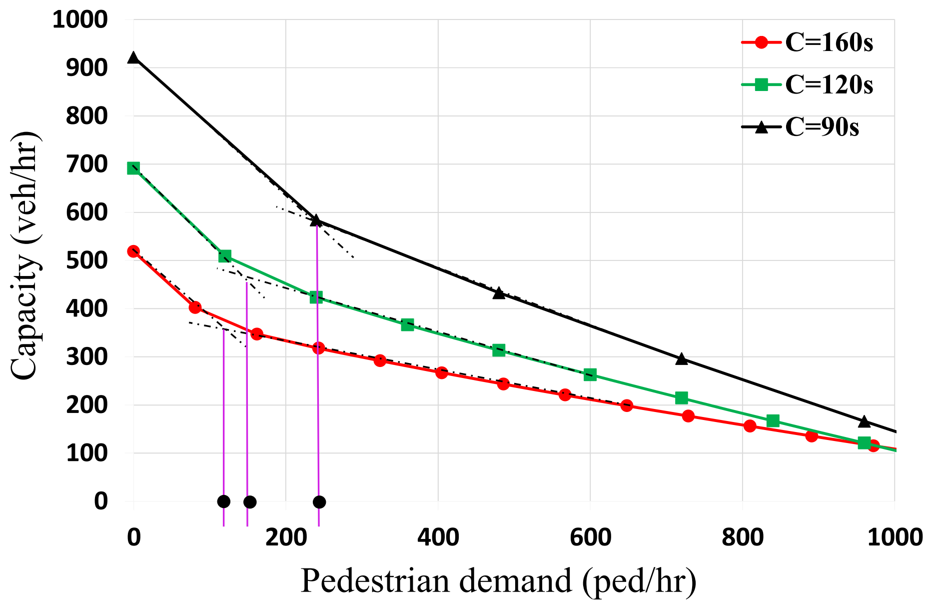


Fig 6.15 Pedestrian demand threshold points at different cycle length

Consequently, the coefficients and other statistics data of the proposed option-1 and option-2 equations are summarized in **Table 6.4.** and **Table 6.5.**

Table 6.4 Option 1 equation coefficients

Variable	Total pedestrian demand < (-2.23*C+528.5) ped/hr		Total pedestrian demand ≥ (-2.23*C+528.5) ped/hr	
	Coefficients	**t-stat**	**Coefficients**	**t-stat**
Constant	-124.8	-2.51	231.4	23.7
Near side pedestrian demand q_n(ped/hr)	-1.131	-8.58	-0.2655	-104
Far side pedestrian demand q_f (ped/hr)	-1.470	-11.2	-0.3853	-148
Crosswalk setback distance l_c (m)	6.675	17.7	4.182	59.2
Stop line setback distance l_s (m)	-7.440	-19.8	-4.663	-65.9
Crosswalk length L (m)	0.8456	2.12	-0.4774	-6.34
EPR ratio (EPR/G)	1112	2.15	-469.2	-4.80

Green time ratio (*G/C*)	1356	53.3	520.9	80.9
R^2	0.95		0.81	
n	360		8767	

Table 6.5 Option 2 equation coefficients

Variables	Coefficients	t-stat
B	231.8	94.34
Near side pedestrian demand q_n **(**ped/hr)	-0.001331	-88.11
Far side pedestrian demand q_f *ped/hr*	-0.002361	-154.1
Crosswalk setback distance l_c **(***cm***)**	3.308	79.26
Stop line setback distance l_s **(***cm*)	-3.667	-87.87
Crosswalk length *L***(***cm***)**	0.3081	6.931
EPR ratio (*EPR/G*)	-3.467	-6.015
Green time ratio (*G/C*)	2.375	62.55
R^2	0.77	
N	8767	

As it is summarized from the coefficients result, the bi-directional pedestrian demand, stop line setback distance, crosswalk length and signal timing ratio have a negative impact on the adjustment factors, while crosswalk setback distance has a positive impact on the adjustment factors.

To validate the proposed equations, one case of comparison between the equation estimation and the proposed method estimation is computed under the condition of L=20m, l_s=5m, l_c=5m, Near/Far ped.ratio=50/50, EPR=5s, G=65s and C=90s, 120s and 160s. **Fig. 6.16** shows the comparison of the original method and the simplified equation estimation of the left-turn lane capacity. As it is clearly noticed from the graphs the option 1 equation prediction is more reasonable to generalize the original method results. In addition, under different settings of crosswalk layout, signal setting and bi-directional pedestrian demands the 45-degree validation plot is shown in **Fig 6.17**, the validation plot also revealed that the option1 equation can successfully predict the estimation results of the original method. Even if the option 2 equation estimation accuracy is less than option 1 equation estimations, however the option 2 equation estimation is simpler to apply since it is written as one equation and the plotted curves are smooth.

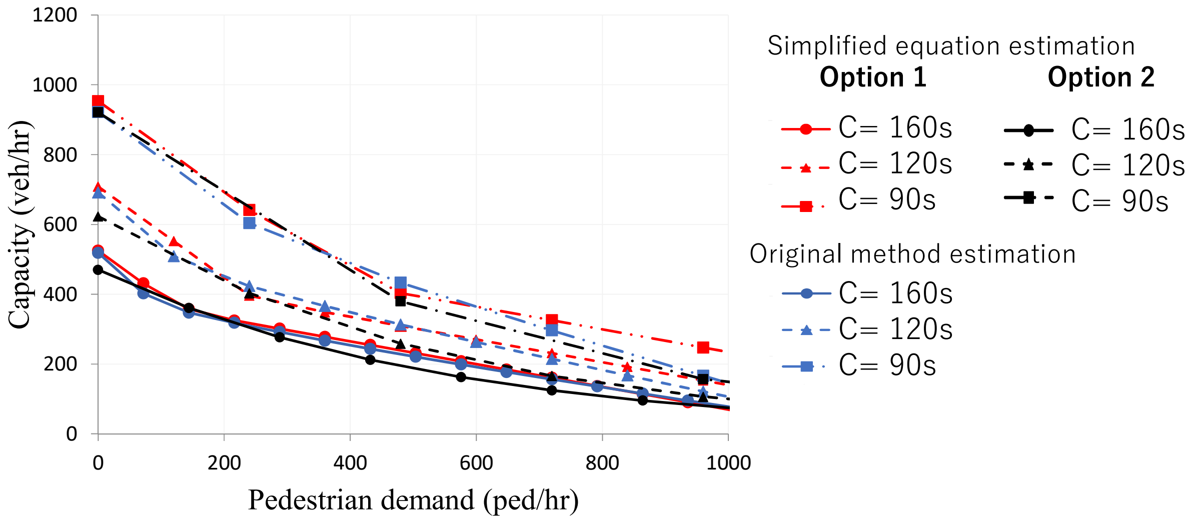


Fig 6.16 Comparison of capacity estimation

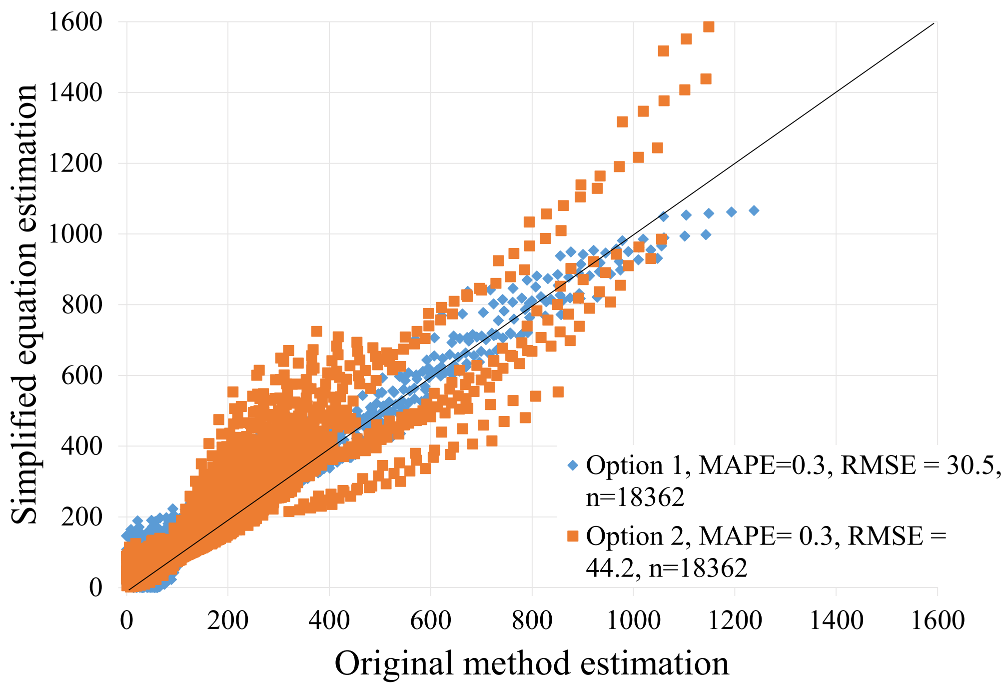


Fig 6.17 Validation of equation estimations

As a result, based on the validation results the option 1 equation can be adopted to generalize the estimation results of the original method. The adjustment factor (*f*) equation that can be easily usable by practitioners can be applied following the flow chart shown in **Fig 6.18**.

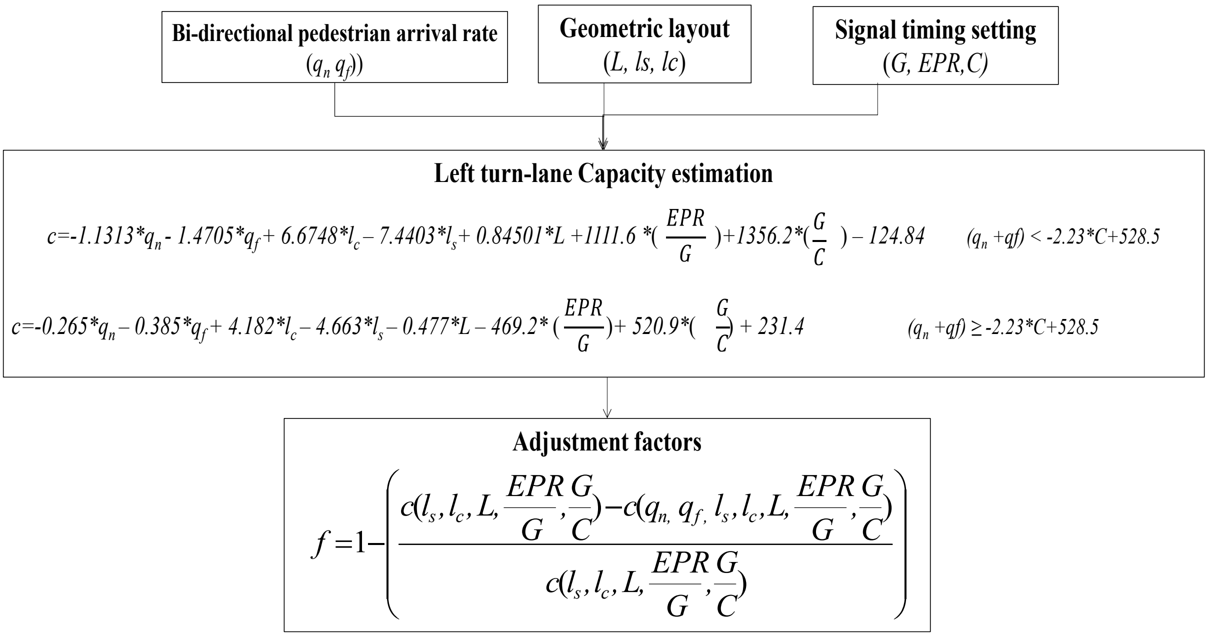


Fig 6.18 Flow chart for adjustment factors estimation

6.4.4 Comparison of adjustment factor estimations with HCM

As it is discussed in Chapter 2 the exiting manuals propose an adjustment factors to consider the impact of pedestrians on the left turning vehicles flow and capacity is estimated based on the proposed adjustment factors. The proposed adjustment factors of this study are compared with widely usable method of HCM adjustment factors under the same conditions of L=20m, ls=5m, lc=5m, C=160, EPR=5s and G=65s. **Fig 6.19 s**hows the adjustment factors contour plots for the HCM method and the proposed method. As it is discussed in Chapter 2, for the adjustment factor estimation based on HCM procedure, only the pedestrian demand is necessary. Therefore, for the HCM case the adjustment factors are computed by combining the near-side and far-side pedestrian demands, while for the proposed method case the adjustment factors are computed under different combinations of near-side and far-side pedestrian demands.

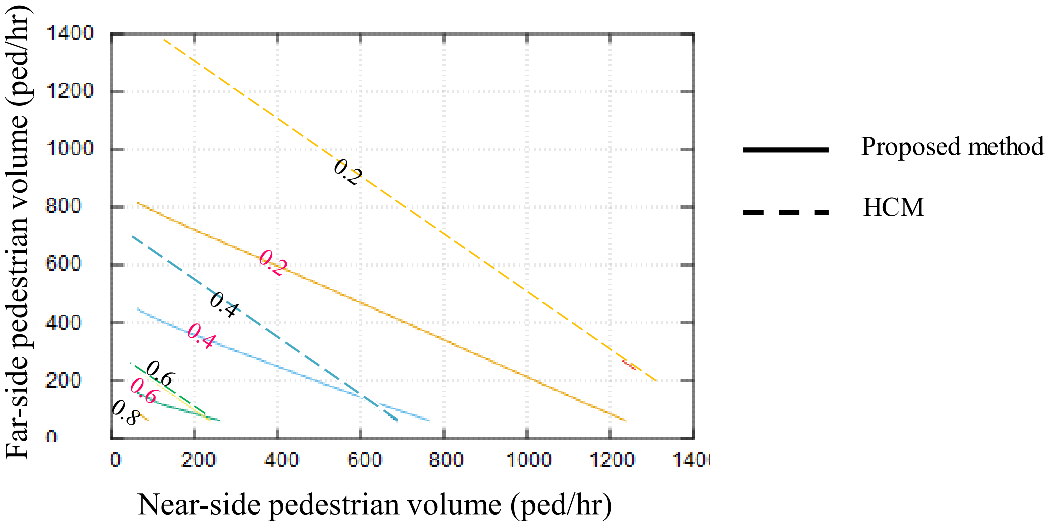


Fig 6.19 Adjustment factors contour plots based on HCM method and proposed method

The comparison of adjustment factors from **Fig 6.19** shows that the HCM method have higher adjustment factors under the same condition with the proposed method, thus HCM method overestimate the capacity estimation when the adjustment factors multiplied by base saturation flow rate, because bi-directional pedestrian interaction and combined impact of crosswalk layout is not considered. However, in the proposed method of this study left turn lane capacity is estimated by considering the bi-directional pedestrian flow interaction under different crosswalk layout conditions, thus realistic impacts of crossing pedestrians can be captured in the proposed adjustment factors.

6.5 Summary

In this chapter, left turn lane capacity is estimated by considering the discussions in chapter 4 and 5 about the detailed investigation of crossing pedestrians and left turning vehicles characteristics. The validation of the proposed estimation procedure is evaluated by two ways; first the proposed method capacity estimation is compared with the observed capacity estimation at selected saturated cycles of the signalized crosswalks, and next the proposed method capacity estimation is compared with the estimation results of selected existing manuals. For both comparisons the proposed estimation method performs better. Because, in the proposed capacity estimation the realistic bi-directional pedestrians flow and the influence of crosswalk layouts are considered by following some detailed empirical observations, in which those factors are not covered in the existing methods.

In addition, the sensitivity of the proposed method for change in crosswalk layout parameters, signal timing setting and bi-directional pedestrian flows are investigated. The results showed that longer crosswalks have higher capacity at lower pedestrian demand conditions because the possibility to observe bi-direction pedestrian flow is lower and even if there are too small number of crossing pedestrians left turning vehicles may accept short gaps since the conflict-area will be wide at long crosswalks. Moreover, the increase in crosswalk set back distance will raise the left turn lane capacity approximately by 17% under all pedestrian demand conditions and the sensitivity to the bi-directional pedestrian flow shows that the far-side pedestrian flow has higher influence on the left turn lane capacity estimation and due to the opposing higher pedestrian demand circumstance; capacity may deteriorate even at lower near-side pedestrian demand conditions.

Finally, the adjustment factors to consider the influence of bi-directional pedestrian flow and crosswalk layout on the left turn lane capacity are proposed. The adjustment factor estimation procedure summarizes the overall discussion of this study about the crossing pedestrians and left tuning vehicles characteristics. Therefore, using the adjustment factors we can easily understand and apply the impact of crossing pedestrians and crosswalk layout in the signalized intersection performance evaluation and signal timing design procedures.

Chapter 7

Conclusion and Future works

7.1 Conclusions

In this study, a methodology for the exclusive left turn lane capacity estimation is proposed by considering the influence of crossing pedestrians and crosswalk layout conditions under concurrent signal timing designed signalized intersections. The main conclusion points of this study are summarized below.

7.1.1 Consideration of bi-directional pedestrian flow

In realistic condition of crossing pedestrians flow there are two directional flows from the two opposite sides of the crosswalks; therefore, in the study of pedestrian flow analysis this condition of crossing pedestrian's characteristics must be studied. In this study, first the bi-directional waiting pedestrian flow is empirically investigated and based on the observation it is noticed that due to the bi-directional pedestrian flow the crossing time of pedestrians is significantly varied, this implies the opposing pedestrian flow interaction can influence the speed of crossing pedestrians. Moreover, the waiting pedestrian platoon dispersion characteristics showed significant change under different combination of bi-directional pedestrian flow interactions especially at higher pedestrian demand crosswalks. Consequently, the waiting pedestrian presence probability by time is modeled considering the bi-directional pedestrian demand conditions. Most of existing methodologies represent the crossing pedestrian flows along the crosswalk by simulators, it is not appropriate to utilize for everyone and calibration is necessary before the application that consumes considerable time. Therefore, by utilizing the modified pedestrian presence probability model proposed in these study practitioners can easily estimate the crossing pedestrian distribution along the crosswalk by elapsed time by entering only the bi-directional pedestrian demand and crosswalk length of the signalized crosswalks.

In the same way, the impact of bi-directional arriving pedestrian flow on the discharge flow of left turning vehicles is investigated from empirical observations of signalized crosswalks. The results showed that the arriving pedestrians entering from far-side direction have higher impact on left turning vehicles discharge flow rate compared with arriving pedestrians entering from near-side direction, because left turning vehicles are highly conscious for pedestrians coming towards their position and also far-side pedestrians need longer time to pass the conflict area due to the crosswalk length.

7.1.2 Waiting pedestrian’s presence time at the conflict-area of signalized crosswalks

Turning vehicles are conscious to crossing pedestrians when they are present inside the conflict-area. As it is discussed in Chapter 4, the bi-directional crossing pedestrian’s behavior can be influenced by different factors and the position distribution of the crossing pedestrians by elapsed green time can be expressed using the pedestrian presence probability model. After pedestrian crossing probability of the edge of the conflict-area is estimated (at the entry and exit section of the conflict-area), then the necessary expected elapsed time to cross the conflict-area is computed by assuming the clearance time thresholds of first and last pedestrians in the bi-directional waiting pedestrian groups. The result implies that pedestrian presence probability model can be used to efficiently estimate the expected pedestrian presence-time of crossing pedestrians at the conflict-area of signalized crosswalks. The analysis of expected pedestrian presence-time shows that when crosswalk length and pedestrian demand increase, pedestrian presence-time in the conflict –area will increase. In addition, in this study a generalized model which is useful to estimate the expected pedestrian presence-time (EPT) of waiting pedestrians by considering the bi-directional pedestrian demand and crosswalk length is proposed. Therefore, based on the model we can easily estimate the influencing time interval of waiting pedestrians at the conflict-area of signalized crosswalks by considering bi-directional pedestrian flow and crosswalk length. Furthermore, the comparison of the estimated pedestrian presence-time with observed crossing time between first and last pedestrians in waiting pedestrian platoon show similar results for all observed crosswalks. The estimated pedestrian presence-time showed a better performance compared with existing estimation procedures.

When pedestrian presence-time in the conflict-area increases, consequently the effect of the pedestrians on left turning vehicles discharge flow will be significant and this must be considered in a proper way during signal timing design strategies. If the assigned green time for turning vehicle is highly influenced by crossing pedestrians in the case of concurrent signal timing, then the capacity of left turn lane may significantly decrease. In addition, when turning vehicles wait for long time to yield crossing pedestrians they may show impatient driving behavior, which causes safety problem for crossing pedestrians.

7.1.3 Crosswalk layout

In this study different crosswalk layout parameters are explored and the significant ones are correlated with crossing pedestrians/left turning vehicles flows. Three influencing factors that represent the crosswalk layout are applied in this study i.e. crosswalk length, crosswalk set back distance and stop line set back distance. Crosswalk length showed a significant impact on both pedestrians and left turning vehicle flows. It indicated a negative impact on the pedestrian presence probability distribution, while it has a positive impact on the left turning vehicles discharge flows. Crosswalk and stop line set back distances are used to characterize the influence on the discharge flow rate of left turning vehicles. Stop line set back distance showed a negative impact on left turning vehicles discharge flow rate, when the stop line is far from the intersection the arriving time to the crosswalk will be long due to that there will be delay to pass the conflict-area and unused green time may increase. On the other hand, crosswalk set back distance showed a positive impact to discharge flow rate. This implies that when the crosswalk position is near to the intersection the left turning vehicles discharge flow rate will decrease. The reason is related with sight distance, left turning vehicles can easily notice the crossing pedestrians at the onset of green in the case of short crosswalk set back so that they will approach with lower turning speed, due to that the number of turning vehicles per cycle will decrease. The result showed that stop line and crosswalk setback distance have opposite impact on the left turn vehicles discharge flow rate. Therefore, we need to have a balanced location of stop line and crosswalk at the planning stage of signalized crosswalks to minimize the impact on left turn lane capacity estimation.

7.1.4 Adjustment factors

To make the application of the proposed left-turn lane capacity estimation procedure more easy and user friendly, capacity estimation and adjustment factors equation is generated based on the estimation of the proposed original left turn lane capacity estimation procedure under the combined influence of crosswalk layout and bi-directional pedestrian flows by combining different possible sensitivity cases. Therefore, using the required adjustment factors to contemplate the impact of bi-directional pedestrian flow and crosswalk layout in the left turn lane capacity estimation, consequently the signalized intersections performance evaluation and signal timing design procedures will have realistic estimated left turn lane capacity values.

7.2 Future works and limitation of the study

For this study, in the observed intersections most of road users are ordinary adults. The elderly and children are seldom observed. Consequently, the pedestrian presence probability distribution cannot represent the distribution of all-inclusive pedestrians; elderly and children pedestrians along the crosswalk may have extremely different behaviors. Therefore, in the future the composition of the observed pedestrian position distribution must be explored.

This study assumed the left turning vehicles and crossing pedestrians strictly obey the priority of pedestrians in the shared space, i.e. in PG time interval pedestrians has priority while left turners has priority in EPR time interval, however if there are significant number of jaywalkers and non-yielding drivers the capacity estimation procedure may need further analysis.

Regarding the conflict in the shared space of the signalized crosswalks only the impact of bi-directional pedestrians on the left turning vehicles discharge flow is analyzed. However, bicycles and other non-motorized vehicles are also important users who may influence the movement of left turning vehicles at the conflict-area, therefore the analysis of bicycles at the signalized intersection must also be further investigated by considering the difference of the cyclist behavior by the operation types of bicycle lanes. In addition, the combined effect of crossing pedestrians and cyclists must be investigated.

For the case of left turning vehicles, in this study only the discharge flow of passenger cars is analyzed, however there are other road users with a mixed traffic flow conditions i.e. in the future the movement of heavy vehicles, buses, motor bicycles etc. combined with passenger cars must be studied. The discharge flow characteristics of the mixed traffic condition will be different and the correlation with crossing pedestrians and geometric layout of the crosswalk is also another dimension of variations.

In this study, only concurrent signal timing setting is examined, however the impact of crossing pedestrians and crosswalk layout at different signal timing conditions like leading pedestrian interval (LPI) setting need further investigation. In addition, the shared left-turn lane is not covered in this study, it may be difficult to apply the proposed method for a shared left-turn lane since the left tuning vehicles characteristics in a shared lane is different from an exclusive left-turn lane and in the future the proposed method can be modified to consider the shared left-turn lanes.

The observed signalized intersections are within a limited range of geometric layout and pedestrian demand conditions, i.e. bi-directional pedestrian demand rang of 0 – 1000 ped/hr , crosswalk length of 15m - 35m, crosswalk width of 5m – 10m, stop line setback distance of 13m – 28m, crosswalk setback distance of 2m – 20m. Consequently, the application range will be limited and in the future by observing additional database of different range of signalized crosswalks the proposed models and analysis can be further tuned.

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