An - Najah Univ. J. Res. (N. Sc.) Vol. 28, 2014

Consideration of Vehicular and Pedestrian Flows in A Multi-Modal Traffic Signal Optimization Strategy for Isolated Intersections

إستراتيجيات تحسين الإشارات المرورية الضوئية على التقاطعات المنفردة آخذاً بالإعتبار تدفق المركبات والمشاة

Wael Alhajyaseen* & Meng LI**

وائل الحاج ياسين، و منغ لي

*Department of Civil Engineering, Faculty of Engineering, An-Najah National University, Palestine. **Department of Civil Engineering, Tsinghua University, 100084, Beijing, China

*Corresponding Author: Email: wael.alhajyaseen@najah.edu

Received: (19/9/2012), Accepted: (26/1/2014)

Abstract

Current signal control strategies tend to ignore the pedestrian delays that may be imposed by reducing traffic delays. Such an objective is reasonable for motorways and rural roads where vehicular traffic is dominant over pedestrian traffic. However, it is not the case in metropolitan cities with large volume of pedestrian demands. This paper developed a traffic signal optimization strategy that considers both vehicular and pedestrian flows. The objective of the proposed model is to minimize the weighted vehicular and pedestrian delays. The deterministic queuing model is used to calculate vehicular traffic delay and pedestrian delay on sidewalk. Pedestrian delay on crosswalk is calculated based on an empirical pedestrian speed model, which considers interactions of pedestrian platoons and their impacts on average walking speed. A Japanese Intersection is utilized as a numerical case study to evaluate the proposed model. MATLAB is used to solve the optimization problem and to output a set of measures of effectiveness (MOEs). The results show that the proposed model improved average person delay (APRD) by 10% without changing the existing cycle length. Moreover, the model can optimize the cycle length and further improve APRD by as much as 44%. In order to demonstrate the applicability of the proposed model for general cases, this paper also conducted sensitivity analysis. The results showed that the proposed model is most significant and necessary for two circumstances: (1) metropolitan areas with high pedestrian demands and (2) major urban arterials with high pedestrian demands crossing major streets.

Keywords: pedestrian, multi-modal, optimization, signal control, delay

ملخص

هدف هذه الدراسة هو تطوير نموذج لتحسين تصميم الإشارات المرورية الضوئية في التقاطعات أخذاً بالإعتبار التأخير للمركبات والمشاة في آن واحد، علما بأن تأخر المشاة لا يؤخذ عادة بعين الإعتبار في تصميم الإشارات المرورية حسب المواصفات والمراجع المعتمدة عالمياً. وقد أثبتت العديد من الدراسات أن إهمال تأخر المشاة، خاصة في مراكز المدن أو المناطق التي تتميز بكثافة مشاة عالية، قد يؤدي إلى فترة تأخير كبيرة، والتي بدورها ستؤدي إلى عدم التزام المشاة بالإشارة المرورية الضوئية، الأمر الذي قد يهدد سلامة المشاة والأمان على التقاطعات المرورية. تقترح هذه الدراسة نموذجاً يقلل التأخير للمركبات والمشاه حيث تم استخدام نموذج الطابور التحديدي لحساب تأخر المركبات والمشاة على الرصيف، بينما تم الإستعانة بنموذج تجريبي لحساب تأخر المشاه أثناء عبورهم الطريق نتيجة التفاعل بين المارة من الإتجاهين المتعاكسين. لتقييم النموذج المقترح تم إستعمال معلومات لتقاطع مروري يقع في مدينة ناغويا في اليابان، كما تم حل النوذج الرياضي بإستعمال برنامج الماتلاب. أظهرت النتائج أن النموذج المقترح يقلل تأخر الشخص المستخدم للتقاطع سواء كان من المشاة أو من راكبي السيارات بنسبة ١٠% بإفتراض عدم تغير الدورة الزمنية للإشارة الضوئية للتقاطع الذي تم تحليله، وبنسبة ٤٤% إذا تم تغيير الدورة الزمنية. بشكل عام تتلخص نتيجة هذا البحث بأن إدخال تأخر المشاة في تصميم الإشارات المرورية الضوئية أمر فعال وضروري خاصة في حالتين أساسيتين: (١) في مراكز المدن حيث يتركز المشاة بأعداد كبيرة و (٢) على الطرق الشريانية الرئيسة التي تتميز بتدفق كبير للمشاة عابري الطريق.

Introduction

Traffic signal control has long been the most important operational strategy for traffic on urban surface streets. Although the control methods and control systems (hardware and software) are quite different case by case, the objective is always similar. It is to minimize the total vehicular delay and/or number of stops at intersections while securing the safety requirements for all stakeholders, i.e. vehicles and pedestrians in most cases. Current strategies tend to ignore the pedestrian delays that may be imposed by reducing vehicle delays. Such an objective is reasonable for motorways and rural roads where vehicular traffic is dominant over pedestrian traffic. However, it is not the case in metropolitan cities with large volume of pedestrian demands. Such ignorance can lead to unnecessary long delays for pedestrians, dangerous behavior by impatient pedestrians, and potential reductions in pedestrian traffic and transit usages.

The continuous and strong growth in transportation demands has created the worst congestion together with the ever-serious concerns on energy shortage. Over the next 25 years, world demand for liquid fuels and other petroleum is expected to increase more rapidly in the transportation sector than in any other end-use sector. Therefore, the practice of traffic signal control has to consider sustainable strategies for the future. Essentially, traffic signal control should promote more cost- and/or energy-efficient transportation modes, e.g. public transportation, carpool, walking, and riding bicycles. Several works on transit signal priority (TSP) (Li, et al., 2005) aim to improve transit service by providing them prioritized timing treatment at signalized intersections.

Another important stakeholder at signalized intersections is the pedestrian. An improved public transportation system and public acceptance always lead to more pedestrians on streets. However, the existing signal control strategies only focus on safety aspects for pedestrians while fail to pay enough attention on the efficiency aspect, i.e. pedestrians’ delay. Actually, pedestrians’ delay can also be significant when compared with vehicular delays. It happens at intersections with consistent medium-to-high pedestrian demands where these are typical in large cities with good public transportation system, e.g. New York, London, Tokyo, etc. Moreover, the optimized signal timing from the perspective of minimizing vehicular delays usually is not optimal for pedestrians' flow. It is because the directional demand ratios (DDRs) among vehicle flows are not likely to be the same as those for pedestrian flows.

This study aims to develop a traffic signal optimization strategy, which considers the safety and efficiency of both vehicular and pedestrian flows. The remainder of the paper is organized as follows. Firstly, the literature review will be presented followed by the detailed description of the proposed methodology. The third section will demonstrate the performance of the proposed model on a Japanese intersection. The fourth section will discuss the applicability of the proposed model for general cases. The last section will conclude this paper with recommendations for future work.

Literature Review

Vehicle delay is perhaps the most important parameter used by transportation professionals to evaluate the performance of signalized intersections. The Highway Capacity Manual (HCM) (Transportation Research Board, 2010) uses the average control delay experienced by vehicles at intersection approaches as a base for determining the level of service. Pedestrian traffic has not been given the same priority as vehicular traffic. However, at many urban areas where large volume of pedestrian exists, it is more rational and reasonable to evaluate the level of service of roadways from a multi-modal perspective. A key goal of multi-modal transportation systems is to minimize delays for all roadway users, including motorized traffic, bicyclists and pedestrians. However, Webster's (1958) and other numerous methods for signal optimization focus on reducing vehicle delays without considering pedestrian flows and delays. Long signal cycle durations from optimizing vehicle flows and signal coordination for vehicles have negative effects on pedestrian movements and may impose large delays on pedestrians (Bayley 1966). Furthermore, long cycles may cause a safety hazard for pedestrians, thus one of the most effective measures to improve pedestrian safety and compliance is by making signals as comfortable as possible, and this is done by minimizing pedestrian waiting time (Garder 1989). Therefore, investigating the rationality of considering pedestrian delays in the optimization of signal control and providing guidelines for the conditions where such a policy should be implemented is very useful and significant.

Few studies have been done to investigate the balance between pedestrian and motorized traffic delays at isolated intersections or the network level. Noland (1996) analyzed the signal timing solutions regarding pedestrians and motorized traffic at isolated intersections with high pedestrian demand. The relative cost of time was used to analyze the performance of signal control, however, the difference between optimized signal parameters considering pedestrian and vehicle delays and those considering vehicle delays only was not shown. Furthermore, general guidelines about the conditions where such control policy is advantageous and reasonable for implementation were missing.

Ishaque, et al. (2005) and Ishaque, et al. (2007) analyzed the trade-offs in pedestrian and vehicle delays in a hypothetical network by considering relative values of time for pedestrians and vehicles. They found that shorter cycle lengths are beneficial for pedestrians. Moreover, the existing policies that are most advantageous to vehicles might be disadvantageous to pedestrians, which do not make the network optimally perform for all road users. They assumed that pedestrian delay is composed only from control delay. Actually with high demands, pedestrians experience significant delays while discharging at the edge of the crosswalk and while crossing the street due to the interaction between opposing pedestrian flows. Furthermore, a discussion about the optimized signal parameters considering pedestrian and vehicle delays was not presented.

Few studies addressed the issue of bi-directional pedestrian flow and its impact on crossing time and speed at signalized crosswalks and the resultant delays. HCM (2010) does not consider the effects of pedestrian demand and crosswalk width on pedestrian crossing time. However, when pedestrian demand increases at both sides of the crosswalk, crossing time increases due to the interaction between conflicting pedestrian flows (Alhajyaseen and Nakamura 2009).

Urbanik, et al. (2000) investigated the effects of different pedestrian phasing schemes based on various left-turning control types and split phasing on pedestrian delays. Wang, et al. (2009) introduced a set of models for calculating pedestrian delays at signalized intersections. The models take into consideration various signal phasing and pedestrian treatment scenarios, especially under two-stage crossing situation. They found that specially designated signal phasing and pedestrian treatments are able to reduce pedestrian delays without affecting vehicle delays significantly. However, no optimization model was developed and no consideration was given to the experienced delay by pedestrians while discharging or crossing at the crosswalk.

Teknomo (2006) proposed a microscopic pedestrian simulation model as a tool to evaluate quantitatively the impacts of a proposed control policy before its implementation on pedestrian behavior at signalized intersections. The developed model was used to demonstrate the effect of bi-directional flow at signalized crosswalks. It was found that at high pedestrian demand with roughly equal flow from each side of the crosswalk, the average crossing speed might drop up to one third compared to the uni-directional flow, which will result in large experienced delays while crossing.

Golani and Damti (2007) proposed a model for estimating crossing time considering start-up lost time, average walking speed, and pedestrian headways as a function of the subject and opposite pedestrian platoons separately. They found that the size of the opposite pedestrian platoon can cause a significant increase in the crossing time of the subject pedestrian platoon especially at high demands. The proposed model relates the impact of bi-directional flow to the headway between pedestrians when they finish crossing. Therefore, it is difficult to see how the interaction is happening and what the resulting speed drop or deceleration is.

Alhajyaseen and Nakamura (2009) developed a theoretical methodology to model total pedestrian crossing time. Pedestrian platoon crossing time was modeled by utilizing the aerodynamic drag force theory to estimate the reduction in crossing speed due to an opposite pedestrian flow. The proposed model was successfully validated from empirical data. In the final formulation, the reduction in crossing speed was estimated as a function of pedestrian demands at both sides of the crosswalk, signal timing parameters and crosswalk geometry. It was found that at high pedestrian demand, a significant reduction in the crossing speed and increase in the crossing time occurs due to the interaction between the bi-directional flows. Therefore, it was concluded that the interactions between opposing pedestrian flows are significant and should be considered in evaluating pedestrian flow at signalized crosswalks.

Methodology

Most existing signal control and planning practices have been focusing on reducing delays for vehicular traffic. The proposed signal control strategy in this paper considers a broader view of intersection delays. In other words, the objective of the "new" strategy is no longer limited to vehicular traffic but to cover other major stakeholders, i.e. vehicular traffic and pedestrians. As shown in Equation (1), the vehicular traffic delay was consolidated with pedestrian delay by giving a weighting factor for each of the two terms. There can be different physical meanings for the weighting factors. For example, the overall objective can be total person delay at an intersection. Thus, the weighting factor is the average occupancy for all traffic or for each vehicle type, e.g. car, bus, commercial vehicle, and taxi. Alternatively, it can be the total economic costs associated with travel delays. Then the weighting factor will be the relative values of time for various modes; car, bus, commercial vehicle, taxi, and pedestrian. Ishaque, et al. (2007) suggested to further split the delays by waiting delays and delays in motion to more accurately represent the various values of time. Finally, the weighting factors can reflect the preferences on vehicular traffic and pedestrians by traffic system managers, traffic planners and operators. So, the proposed strategy can assist them in making trade-offs among different transportation modes. For example, the manager of congested metropolitan areas, e.g. Tokyo, New York, etc., can set a higher preference on pedestrian and transit to promote "green" and more efficient transportation modes.

$$F^{objective} = \sum_{i=1}^{N} \left(W^{Veh} \cdot delay_i^{Veh} + W^{Ped} \cdot delay_i^{Ped} \right) \quad (1)$$

where: W^{Veh} and W^{Ped} are the weighting factors for vehicular traffic delay and pedestrian delay, respectively; $delay_i^{Veh}$ is vehicular delay for phase i; and $delay_i^{Ped}$ is pedestrian delay for phase i; N is the number of phases in a signal cycle.

Model Assumptions

In order to facilitate the model formulation and the latter discussions, few assumptions were made about intersection geometry, traffic demand, and signal settings. However, it is noted that the proposed concept is general so that the model can be applied for most general cases with minor modifications on these assumptions.

Generally speaking, pedestrian trip is the connection trip from either a transit station or parking facility to the final destination. Unlike vehicle trips, pedestrians typically will not cross many intersections in a row with full speed. Thus, it is not critical to consider signal timing coordination for pedestrians. On the other hand, a large portion of signal control in metropolitan areas is fixed-timing control. Therefore, the proposed model focuses on isolated intersections with fixed-timing control. With the expansion of the objective to the network level, the model can be readily modified to consider network coordination for vehicular traffic.

Without considering the network effect, it is assumed that the demands for both vehicular traffic and pedestrian are consistent and uniformly distributed within a certain period of time. Moreover, the procedure in Japanese Manual of Traffic Signal Control (Japan Society of Traffic Engineers 2006) is adopted for the signal timing regulations and requirements, e.g. minimum traffic green, minimum pedestrian green and flashing time, yellow and all-red interval. Japanese Manual of Traffic Signal Control (JMTSC) is quite similar in such settings to the Manual on Uniform Traffic Control Devices MUTCD (FHWA 2009) in the U.S.A.

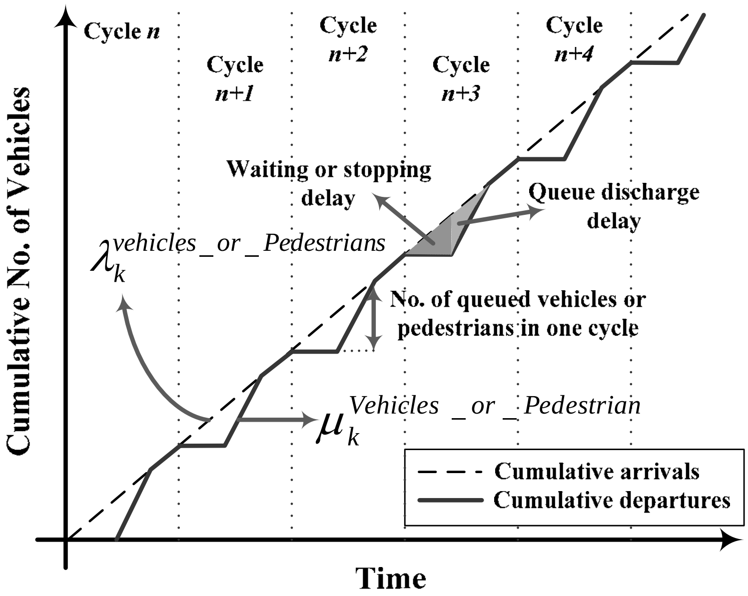


Figure (1): Idealized departure and arrival curves at a signalized intersection.

Model Formulation

Vehicular Traffic Delay

The widely used deterministic queuing model (Equation 2) was chosen to estimate the vehicular delay as shown in Figure 1. The model views traffic as a few uniform streams of arriving vehicles. Traffic signal is a control device that periodically opens its gate to a traffic stream after serving the conflicted streams. As illustrated by Figure 1, the areas between the cumulative arrivals and departures curves represent the total delay incurred by all vehicles on the same approach to cross the intersection. Equation (2) can be derived to calculate the total vehicular delay for all signal phases within a signal cycle. The model assumes instantaneous acceleration and deceleration for all vehicles and all vehicles queue vertically at the stop line, thus the exact number of queued vehicles at a given instant may not be accurate. But the delay estimation is not bias over an entire queue formation and dissipation process (Dion, et al. 2004).

$$delay_i^{Veh} = \sum_{k=1}^{N_i} \frac{\mu_k \cdot \lambda_k}{2(\mu_k \cdot \lambda_k)} (C - g_i) \tag{2}$$

where: $delay_i^{Veh}$ is the total vehicular traffic delay for phase i; N_i is the number of movements for phase i; μ_k and λ_k are saturation flow rate and arrival flow rate for movement k; C is signal cycle length and g_i is effective green for phase i.

Pedestrian Delay

Pedestrian delay is more complicated to compute than vehicular traffic delay because pedestrians are more active and do not form a well-organized queue as vehicular traffic does in their lane. The experienced delay by pedestrians can be divided into two parts. The first part is the experienced delay before stepping down from the sidewalk. It consists of waiting delay for green signal and discharging delay for standing pedestrian queue on the sidewalk. The other part is the experienced delay while crossing the crosswalk. This delay results from the interaction between opposing pedestrian flows on the crosswalk and it is significant when pedestrian demand is high.

The waiting and discharging processes by pedestrians on sidewalk are similar with what happens to vehicular traffic before discharging from the intersection stop-line, as shown in Figure 1. Therefore, the pedestrian delay on sidewalk can be calculated using Equation (3). It is noted that the effective green for pedestrian phase does not include the pedestrian flash warning time because it is assumed that all pedestrians would stop stepping down from the sidewalk when the warning sign starts to flash.

$$delay_i^{PedSW} = \sum_{k=1}^{N_i^{Ped}} \frac{\mu_k^{Ped} \cdot \lambda_k^{Ped}}{2(\mu_k^{Ped} - \lambda_k^{Ped})} (C - g_i^{Ped}) \tag{3}$$

where: $delay_i^{PedSW}$ is the total pedestrian delay on sidewalk for pedestrian phase i; N_i^{Ped} is the number of movements for pedestrian phase i; μ_k^{Ped} and λ_k^{Ped} are saturation flow rate and arrival flow rate for pedestrian movement k *and* g_i^{Ped} is effective green for pedestrian phase i.

The pedestrian delay on crosswalk is due to the interaction of pedestrian platoon with the opposing pedestrian platoon. According to Alhajyaseen and Nakamura (2009), the pedestrian walking speed can be significantly dropped due to the size of the opposite platoon, crosswalk width and some other factors. In order to simplify the model without losing much of the accuracy, it is assumed that all pedestrians on the same movement walk with an average speed $\bar{v}_k^{Ped}$ for the whole crosswalk. Thus, the pedestrian delay on crosswalk can be calculated using Equation (4). The developed model by Alhajyaseen and Nakamura (2009) is utilized to estimate the average speed of the subject pedestrian flow $\bar{v}_k^{Ped}$ as shown in Equation (6). It shows that the crossing speed of subject pedestrian platoon is a function of crosswalk geometry, pedestrian demand at each side of the crosswalk and free-flow speed of pedestrians.

$$delay_i^{PedCW} = \sum_{k=1}^{N_i^{Ped}} \lambda_k^{Ped} \cdot (C - g_i^{Ped} + t_k^q) \cdot L_i \cdot \left(\frac{1}{\bar{v}_k^{Ped}} - \frac{1}{V_{FF}^{Ped}}\right) \tag{4}$$

$$where: \quad t_k^q = \frac{\lambda_k^{Ped}}{\mu_k^{Ped} - \lambda_k^{Ped}} (C - g_i^{Ped}) \tag{5}$$

where: $delay_i^{PedCW}$ is the total pedestrian delay on crosswalk for pedestrian in phase i; t_k^q is the queue discharging time for pedestrian in phase i and it is estimated according to Equation (5); L_i is the length of crosswalk for pedestrian phase i; $\bar{v}_k^{Ped}$ is the average walking speed for pedestrian movement k; V_{FF}^{Ped} is the free flow walking speed and is assumed as 1.45*m/s* in this study.

$$\bar{v}_k^{Ped} = \sqrt{\left(V_{FF}^{Ped}\right)^2 - \frac{0.02P_o\left(\frac{P_k}{P_k + P_o}\right)^{0.791}\left(V_{FF}^{Ped}\right)^2 L_i}{W_k}} \tag{6}$$

$$\text{where: } P_k = \lambda_k^{Ped}(C - g_i^{Ped} + t_k^q) \quad and \quad P_o = \lambda_o^{Ped}(C - g_i^{Ped} + t_o^q) \tag{7}$$

where L_i is crosswalk length in phase i; W_k is crosswalk width for pedestrian movement k; P_k and P_o are the subject and opposite pedestrian demands on the same crosswalk in phase i.

Model Constraints

In order to generate reasonable signal timings, the model has to satisfy some constraints. Generally, traffic signal timings have the constraints on minimum green and maximum saturation degrees. The same constraints are applied on pedestrian green. According to JMTSC (2006) and Japanese practices, the minimum pedestrian time $T_i^{Ped\,\min}$ is defined as the sum of pedestrian green g_i^{Ped} and flash warning time F_i^{Ped}, as shown by Equation (8). $T_i^{Ped\,\min}$ is a function of pedestrian free flow walking time, pedestrian demand P_k, saturation flow μ_k^{Ped} and crosswalk width W_k for movement k. JMTSC (2006) defines F_i^{Ped} as the walking time for half of a crosswalk. It is because pedestrians who fail to pass the mid-point of crosswalk when warning starts to flash suppose to come back, although very few people actually follow this rule. Given Equations (5) ~ (10), the minimum pedestrian green can be obtained by Equation (11) with the flow parameter ρ_k^{Ped} defined by Equation (12).

$$g_i^{Ped} + F_i^{Ped} \geq T_i^{Ped\,\min} = \frac{L_k}{V_{FF}^{Ped}} + \frac{P_k}{\mu_k^{Ped} \times W_k} \tag{8}$$

$$F_i^{Ped} = \frac{L_k}{2V_{pw}} \tag{9}$$

$$P_k = \lambda_k^{Ped}(C - g_i^{Ped} + t_k^q) \tag{10}$$

$$g_i^{Ped} \geq \frac{1}{W_k + \rho_k^{Ped}} \left(\frac{W_k \cdot L_k}{2V_{FF}^{Ped}} - \rho_k^{Ped} C \right) \tag{11}$$

$$\rho_k^{Ped} = \frac{\lambda_k^{Ped}}{\mu_k^{Ped} + \lambda_k^{Ped}} \tag{12}$$

Another constraint of the model is the relationship between vehicular traffic green and pedestrian green. As shown by Equation (13), the green plus flashing warning time for pedestrian phase *i* should not be longer than the duration of the corresponding vehicular traffic through phase *i*. Lastly, the sum of all vehicular traffic greens together with the yellows and all-reds is the cycle length *C* (Equation 14).

$$g_i \geq g_i^{Ped} + F_i^{Ped} \tag{13}$$

$$\sum_{i}^{N_i} (g_i + Y_i + AR_i) = C \tag{14}$$

Measures of Effectiveness (MOEs)

The definition of measures of effectiveness (MOEs) is essential when evaluating the system performance. It can also represent the preference of the system to designer and managers. In this study, three major MOEs are defined: average vehicular delay AVD (*sec/veh*), average pedestrian delay APD (*sec/ped*), and average person delay APRD (*sec/per*) which are presented in Equations (15)~(17). All these parameters were estimated in time interval of one signal cycle. It is noted that APRD is a special case of weighted average intersection delay when the weighting factor for vehicular traffic W^{Veh} is assumed to be equal to the average vehicle occupancy N_{occ}^{Veh}. In this study, N_{occ}^{Veh} is assumed as 1.2*per/veh* and that for pedestrian as 1.0.

$$AVD = \sum_{k=1}^{N_i} \frac{\mu_k}{2(\mu_k - \lambda_k)} (1 - g_i / C) \tag{15}$$

$$APD = \sum_{k=1}^{N_i^{Ped}} \frac{\mu_k^{Ped}}{2(\mu_k^{Ped} - \lambda_k^{Ped})}(1 - g_i^{Ped}/C) + \sum_{k=1}^{N_i^{Ped}} (1 - g_i^{Ped}/C + t_k^q/C) \cdot L_i \cdot (\frac{1}{\bar{v}_k^{Ped}} - \quad (16)$$

$$APRD = \frac{\sum_{i=1}^{N} (delay_i^{Veh} + delay_i^{Ped})}{C \cdot \sum_{i=1}^{N} (\lambda_i \cdot N_{OCC}^{Veh} + \lambda_i^{Ped})} \quad (17)$$

Computation Procedure

The objective function of the proposed optimization model is complicated. The vehicular delay and pedestrian delay on sidewalk are quadratic. The pedestrian delay on crosswalk has complex exponential terms. As a result, solving this problem mathematically or developing an optimization algorithm is very difficult and time consuming. Therefore, MATLAB, the most popular and powerful numerical computing environment, was selected to solve the problem. Since all the constraints in the model are in linear form, the nonlinear programming function *fmincon()* from MATLAB's optimization toolbox is chosen to solve the problem.

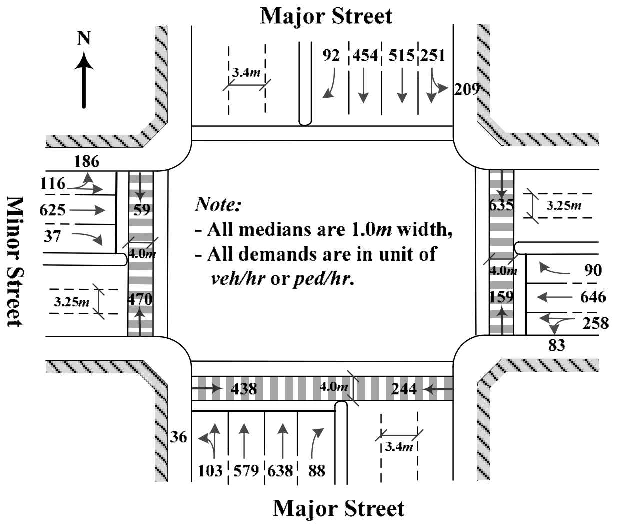
Numerical Case Study

The proposed model was firstly applied on a real Japanese signalized intersection for demonstration purpose. Japanese signalized intersections are generally characterized by unreasonable long cycles (140sec ~ 200sec) regardless of the size, complexity or vehicle demand, which impose high delays on all users. Such long cycles are referred to the high vehicle demand; however, at some signalized intersections where vehicle demands are not high, still cycle lengths are very long. Moreover, urban Japanese signalized intersections are often characterized by medium to high pedestrian demands. Some field data were collected at a key multilane intersection with high vehicle and medium pedestrian demands. This intersection is called Chikatetsu Horita and is close to downtown Nagoya, Japan. Video data were collected for both vehicles and pedestrians on Thursday (18 of June, 2009) in the morning peak hour. The geometric characteristics, vehicle and pedestrian demands as well as phasing and signal timings are presented in Figure 2. The intersection has a dedicated right-turn lane and pedestrian crosswalks on three approaches. The signal timing follows what is so called lag-lag sequence where the through phases 1 and 3 are followed by right-turn phases 2 and 4. The pedestrian crossing is traditional (Figure 2(b)). There is neither exclusive pedestrian phase nor staggering crossing with pedestrian median refuge. The pedestrian phases start together with the non-conflict through traffic phase 1 and 3 and end before the end of the vehicular traffic phases.

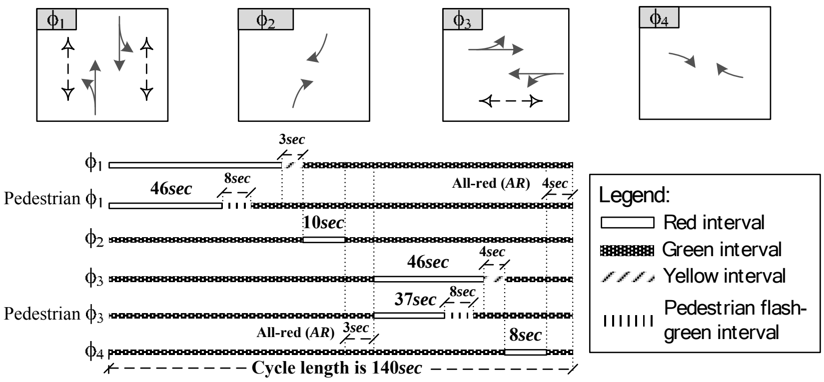
Existing signal timings measured from Chikatetsu Horita intersection were compared with the optimized timing generated by the proposed model. Firstly, the proposed model optimized the timings without changing the existing cycle length (140*sec*). Then the proposed model tried to further improve the performance of signal timings by optimizing the cycle length. The reduction in cycle based on AVD, APD and APRD are presented in Figure 3.

At cycle length 140*sec*, Figure 3(a) shows that the optimized signal timings are able to reduce APRD from 39 to 35 *sec/cycle/per* by 10%. The improvements came from the reduction of APD from 43 to 35 *sec/cycle/ped* by 19% and also the reduction of AVD from 38 to 35 *sec/cycle/veh* by 8%. The proposed model was able to reduce both APD and AVD because the existing timings at the Japanese intersection were not optimized for either of them. The proposed model suggests reversing the green splits for phase 1 and phase 3 as shown in Figure 3(b). It is due to the higher per lane vehicle and pedestrian demands for phase 3. For right-turn phases 2 and 4, the new model suggests no changes because both of them run close to their saturation.

It is noted that the signal cycles in Japan is generally long. Thus, the proposed optimization model tried shorter cycle lengths from 140 seconds to 70 seconds. As shown in Figure 3(a), all of AVD, APD, and APRD can be significantly reduced by decreasing the cycle length. For example, APRD can be reduced from 39 to 22 *sec/cycle/per* by 44% when cycle length was 70*sec*. This suggests that existing long cycle lengths at Japanese intersections are not quite rational. Therefore, adopting shorter cycles could result in a drastic reduction in the experienced delays for all users, which will lead to significant improvements in the overall mobility levels of road networks and further it will contribute to safety improvements.



(a) Intersection geometry, vehicle and pedestrian demands.

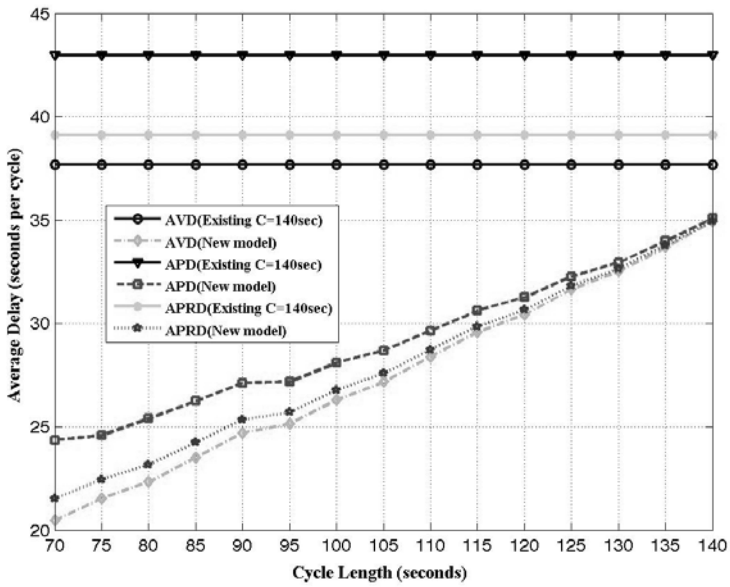


(b) Phasing and signal timing.

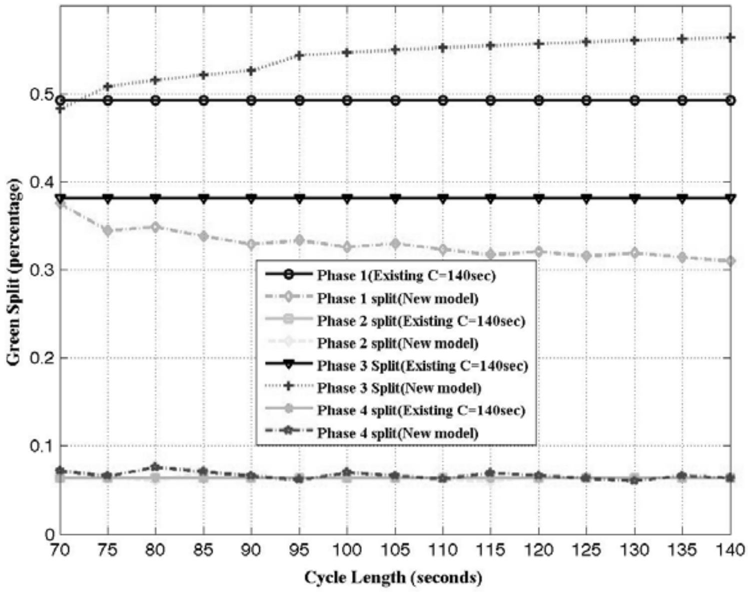
Figure (2): Characteristics of Chikatetsu Horita intersection (Nagoya city).

Discussion for General Cases

Although it is proved that the proposed strategy can significantly improve the existing timings for a typical Japanese intersection, it does not mean the model can improve the existing signal timings for any general case. In other words, there should be certain circumstances that the proposed strategy is necessary and significant. Here a sensitivity analysis is conducted to reveal such scenarios. The purpose of the sensitivity analysis is to evaluate the potential costs and benefits of the proposed model at (1) different combinations of directional demands; and (2) different combinations of pedestrian and vehicular demands. Two directional demand ratios (DDRs) are defined. For vehicular traffic, the directional demand ratio DDR^{Veh} is defined by Equation (18) as the average per lane traffic on the major street over that on the minor street. For pedestrian, the directional demand ratio DDR^{Ped} is defined by Equation (19) as the average per movement pedestrians moving along the major street over that along the minor street. It is noted that the major street is defined by the street with relatively higher vehicular demands than the crossing street, which is named minor street.



(a) Comparison of AVD, APD and APRD.



(b) Comparison of green splits.

Figure (3): Comparisons between the proposed strategy and existing timings.

$$DDR^{Veh} = \frac{\lambda_{major}^{Veh} / N_{major}^{Lane}}{\lambda_{minor}^{Veh} / N_{minor}^{Lane}} \tag{18}$$

$$DDR^{Ped} = \lambda_{major}^{Ped} / \lambda_{minor}^{Ped} \tag{19}$$

where DDR^{Veh} and DDR^{Ped} are directional demand ratios for vehicular traffic and pedestrian, respectively; λ_{major}^{Veh} and λ_{minor}^{Veh} are vehicular demands along major street and minor street; N_{major}^{Lane} and N_{minor}^{Lane} are number of lanes for major and minor streets; λ_{major}^{Ped} and λ_{minor}^{Ped} are pedestrian demands walking along major street and minor street.

As shown in Table 1, three demand scenarios are also defined. In scenario 1, both of the demand levels for vehicles and pedestrians are medium. In scenario 2, the demand level for vehicles is low while that for pedestrian is high. Scenario 3 is the reverse of scenario 2 with high vehicular demand and low pedestrian demand. Scenario 1 represents a typical intersection in metropolitan areas with medium pedestrian demands together with medium vehicular traffic. Scenario 2 represents the center of major metropolitan areas with significantly high pedestrian demands, such as downtown New York or Tokyo. Scenario 3 represents rural or suburban areas with low pedestrian demands. The scenario of very high pedestrian and vehicular demands is not considered, since in such case the intersection will be operating near capacity with long cycle lengths and high associated pedestrian and vehicle delays. Therefore, the consideration of pedestrian delays in such case will lead to oversaturation or very long impractical cycle lengths.

Table (1): Assumed demand scenarios.

Demand scenarios			Vehicular demand *veh/hr*		Pedestrian demand and crosswalk width	
			Major street		Crossing the major street*	
No.	Demand level					
	Vehicle	Pedestrian	Through	Right turning	Demand *ped/hr*	Crosswalk width *m*
1	Medium	Medium	400	100	900	6.0
2	Low	High	200	50	1440	8.0
3	High	Low	550	150	360	4.0

*pedestrian demand at both sides of the crosswalk with directional split ratio of 0.5.

A typical Japanese intersection layout with one particular phase sequence is adopted in this analysis. The assumed intersection layout and phasing scheme are identical to that of the previous case study, which was presented in Figure 2.

In the analysis, the results of the proposed optimization model, which considers pedestrian delays, are compared with the similar optimization model without considering pedestrian delays. Vehicular DDR from 1 to 4 and pedestrian DDR from 0.2 to 2 are analyzed. Figures 4 and 5 illustrate the comparison of APRD, APD, and AVD for demand scenario 1. The difference of APRD, AVD and APD are defined by the values for the proposed model minus those values for the model without considering pedestrian delays. When pedestrian DDR is larger than 1, the pedestrians walking along the major street are more than those walking along the minor street. According to Figure 4, there is no significant difference for APRD when the DDR for pedestrian is larger than 1. In other words, when both vehicular demand and pedestrian demand are at medium level and they share the same major movement direction, the signal timing optimization models with and without considerations of pedestrian delays can reach similar results. When pedestrian demand ratio is close to or smaller than 1, the majority of pedestrian demands are in conflict with those of vehicular demands. In such situations, the proposed strategy started to surpass the traditional model without considering pedestrian delays. Furthermore as shown in Figure 5, at a fixed pedestrian DDR of 0.4 for example, and by varying the vehicle DDR, it is found that as vehicle DDR increases the difference between the proposed and existing model increases also. This is because as vehicle DDR increases the traditional model (existing) will assign longer greens to the major vehicular traffic phase (Phase 1), while shorter greens will be assigned to the minor vehicular traffic phase (Phase 3), although the pedestrian DDR is 0.4, which means that, the majority of pedestrians are crossing the minor vehicular flow along with the major vehicular flow (Phase 3).

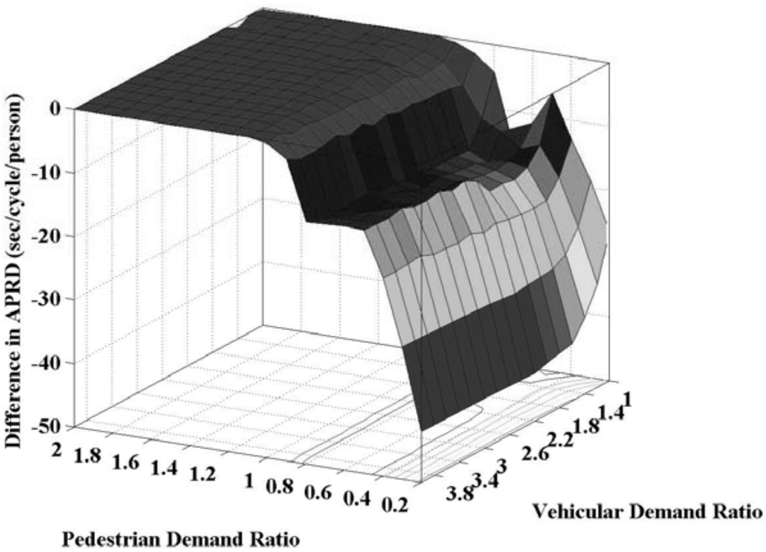


Figure (4): Comparison of APRD between the existing and the proposed models.

As illustrated by Table 2, the proposed new model can improve the APRD for the regular traffic model by 14% when vehicular traffic demand is 1.4 and pedestrian DDR is 0.8. The conflict between majority pedestrian demands and majority vehicular demands got more severe when the gap of the two demands ratios grew. In this scenario, the most improvement on APRD is 42% when pedestrian DDR is 0.2 and vehicular DDR is 4. It is noted that the differences between the two models are more sensitive with pedestrian DDR than the vehicular DDR. It is because the increment of pedestrian demand had more significant impacts on APRD due to the non-linear pedestrian speed model.

Table 2 illustrates the results for all the three demand scenarios. It is not a surprise that the proposed model performs the best in scenario 2 with low vehicular demand and high pedestrian demand. It is because the regular signal-timing model fails to pay enough attentions on pedestrian delays that are also important contributors to the overall APRD. For example, the new model can improve the APRD by as much as 55%.

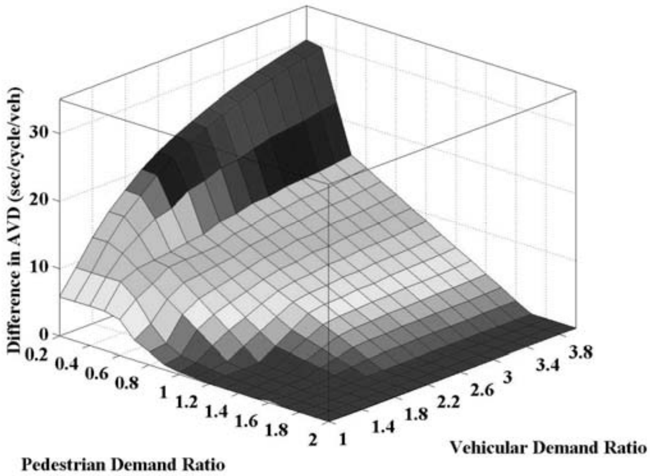
When traffic and pedestrian demands are both balanced with ratio 1, the new model achieved more improvement on APRD for demand scenario 2 than for demand scenario 1. In scenario 3, the benefits of the new model are less than those in scenarios 1 and 2. For instance, there is absolutely no change on signal timings when pedestrian and vehicular demands are balanced. When the DDRs are conflicted with each other, e.g. 1.4 for vehicles and 0.8, the APRD are the same for the two models although APD, AVD, and the timings are slightly different.

Table (2): Results of sensitivity analysis.

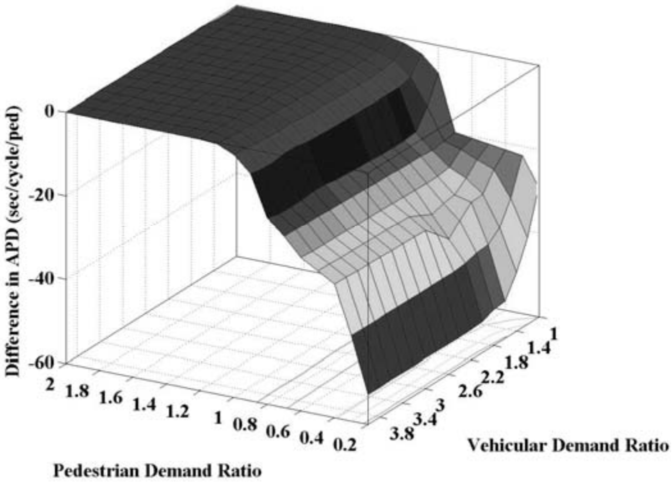
Demand Scenarios									
	1:Medium(vehicle) Medium(ped)			**2: Low(vehicle) High(ped)**			**3: High(vehicle) Low(ped)**		
	Existing Model	**New Model**	**Difference**	**Existing Model**	**New Model**	**Difference**	**Existing Model**	**New Model**	**Difference**
Demand Ratio for Vehicular Traffic: 1.0									
Demand Ratio for Pedestrian: 1.0									
APRD	38.2	37.9	-0.3 (-1%)	37.8	36.5	-1.3 (-3%)	42.7	42.7	0 (0%)
APD	42.3	38.2	-4.1 (-8%)	41.6	39.3	-2.3 (-6%)	40.1	40.1	0 (0%)
AVD	35.4	35.8	+0.4 (1%)	29.4	30.3	0.9 (3%)	43.2	43.2	0 (0%)
Phase 1 Green	71*sec*	62 *sec*	-9 (-13%)	75*sec*	59*sec*	-16 (-21%)	54*sec*	54 *sec*	0 (0%)
Ped 1 Green	61*sec*	51 *sec*	-10 (16%)	65*sec*	49*sec*	-16 (-25%)	44*sec*	44 *sec*	0 (0%)
Phae 3 Green	38*sec*	47 *sec*	9 (24%)	43*sec*	59*sec*	16 (37%)	48*sec*	48 *sec*	0 (0%)

… Continue table (2)

Ped 3 Green	25 *sec*	34 *sec*	9 (36%)	30 *sec*	45 *sec*	15 (50%)	35 *sec*	35 *sec*	0 (0%)
Demand Ratio for Vehicular Traffic: 1.4									
Demand Ratio for Pedestrian: 0.8									
APRD	43.6	37.3	-6.3 (-14%)	49.4	38.3	-11.1 (-22%)	35.7	35.7	0 (0%)
APD	59.1	42	-17.1 (-29%)	57.7	40.4	-17.3 (-30%)	42.1	40.5	-1.6 (-4%)
AVD	29.9	33.1	-3.2 (-11%)	26	32.2	6.2 (24%)	34.1	35.2	1.1 (3%)
Phase 1 Green	87 *sec*	60 *sec*	-27 (-31%)	89 *sec*	48 *sec*	-41 (-46%)	75 *sec*	68 *sec*	-7 (-9%)
Ped 1 Green	77 *sec*	50 *sec*	-27 (-35%)	79 *sec*	38 *sec*	-41 (-52%)	65 *sec*	57 *sec*	-8 (-12%)
Phae 3 Green	26 *sec*	54 *sec*	28 (108%)	29 *sec*	70 *sec*	41 (141%)	34 *sec*	41 *sec*	7 (21%)
Ped 3 Green	13 *sec*	40 *sec*	27 (208%)	16 *sec*	57 *sec*	41 (256%)	21 *sec*	28 *sec*	7 (33%)
Demand Ratio for Vehicular Traffic: 4.0									
Demand Ratio for Pedestrian: 0.2									
APRD	77.7	45.1	-32.6 (-42%)	92.6	41.5	-51.1 (-55%)	51.8	45	-6.8 (-13%)
APD	97.3	44.4	-52.9 (-54%)	100.4	40.4	-60 (-60%)	88.8	54.7	-34.1 (-38%)
AVD	17	47.1	30.1 (177%)	16	52.1	36.1 (226%)	18.4	36.1	17.7 (96%)
Phase 1 Green	96 *sec*	34 *sec*	-62 (-65%)	95 *sec*	10 *sec*	-76 (80%)	95 *sec*	56 *sec*	-39 (-41%)
Ped 1 Green	86 *sec*	24 *sec*	-62 (-72%)	85 *sec*	8 *sec*	-77 (-91%)	85 *sec*	46 *sec*	-39 (-46%)
Phae 3 Green	22 *sec*	84 *sec*	62 (282%)	23 *sec*	99 *sec*	76 (330%)	23 *sec*	62 *sec*	39 (170%)
Ped 3 Green	9 *sec*	71 *sec*	62 (689%)	10 *sec*	86 *sec*	76 (760%)	10 *sec*	49 *sec*	39 (390%)



(a) Difference in average vehicle delay AVD.



(b) Difference in average pedestrian delay APD.

Figure (5): Comparisons of AVD and APD between the existing and the proposed models.

It is noted that the weighting factor of the vehicular delay is assumed to be equal to the average occupancy of vehicles with a value of 1.2 *per/veh* in order to estimate the APRD. Such parameter is customizable and the model results can vary accordingly. As aforementioned, the weighting factor can also be the preference of traffic system managers, planners and operators. Therefore, the results can be weighted total delay rather than the average person delay presented in this section.

In summary, the proposed model is most significant and necessary for two circumstances: (1) metropolitan areas with high pedestrian demands and (2) major urban arterials with high pedestrian demands crossing the major streets. For major urban arterials, transit stations are located along the sides of major streets. Most pedestrians get off transit stations and cross major streets to reach their final destinations or transfer stations.

Conclusion

This paper developed a traffic signal optimization strategy that considers both vehicular and pedestrian flows. The results show that the proposed model improved average person delay (APRD) by 10% without changing the cycle length for the existing timings. Moreover, the model can optimize the cycle length and further improve APRD by as much as 44%. The sensitivity analysis, which was performed on various combinations of vehicular and pedestrian demands and different combinations of directional demands, show that the proposed model is most significant and necessary for two circumstances: (1) metropolitan areas with high pedestrian demands and (2) major urban arterials with high pedestrian demands crossing the major streets.

As a future step, efforts can be made to expand the optimization model to the network level. The proposed model was only applied on an intersection with typical stage based signal control and basic phasing scheme. Thus investigations on various phasing schemes, signal control systems (such as movement based signal control) and on different pedestrian crossing scenarios (such as two-stage crossing) are also significant and needed. Moreover, since the model is dealing with person delay, it is also important to consider public transportation in the objective function. Another important dimension which was not considered in this study is to investigate the environmental impacts of promoting non motorized traffic such as pedestrians in traffic operations.

Acknowledgements

The authors would like to acknowledge Professor Nakamura Hideki and ***Interchange*** Nakamura Lab at Nagoya University for the early discussions on this topic and the data collection efforts in Nagoya, Japan.

References

- Alhajyaseen, W. & Nakamura, H. (2009). "A methodology for modeling pedestrian platoon discharge and crossing times at signalized crosswalks". Proceedings of the 88th Transportation Research Board Annual Meeting. Washington. D. C.

- Bayley, J.M. (1966). "Pedestrian and traffic control". Traffic Engineering and Control 8. 311-312.

- Dion, F. Rakha, H. & Kang, Y.S. (2004). "Comparison of delay estimates at under-saturated and over-saturated pre-timed signalized intersections". Transportation Research Part B (38). 99-122.

- Federal Highway Administration FHWA. (2009). Manual on Uniform Traffic Control Devices for Streets and Highways MUTCD.Washington. D.C. U.S.A.

- Garder, P. (1989). "Pedestrian safety at traffic signals: a study carried out with the help of a traffic conflicts technique". Accident Analysis and Prevention 21. 435-444.

- Golani, A. & Damti, H. (2007). "Model for estimating crossing times at high occupancy crosswalks". Proceedings of the 86th Transportation Research Board Annual Meeting. Washington. D. C.

- Ishaque, M.M. & Noland, R.B. (2005). "Micro-simulation of vehicle and pedestrian signal timings: minimizing multi-modal travel times". Transportation Research Record. (1939). Transportation Research Board of the National Academics. Washington. D.C. pp.107–122.

- Ishaque, M.M. & Noland, R.B. (2007). "Trade-offs between vehicular and pedestrian traffic using micro-simulation methods". Transportation Policy. (14). 124-138.

- Japan Society of Traffic Engineers. (2006). Manual of Traffic Signal Control**.** Tokyo. Japan.

- Li, M. Yin, Y.F. & Zhang, W.B. (2005). "Adaptive Transit Signal Priority on Actuated Signalized Corridors". Proceedings of the 84th Transportation Research Board Annual Meeting. Washington. D.C.

- Noland, R.B. (1996). "Pedestrian travel times and motor vehicle traffic signals". Transportation Research Record: Journal of the Transportation Research Board. (1553). Transportation Research Board of the National Academics. Washington. D.C. 28-33.

- Teknomo, K. (2006). "Application of microscopic pedestrian simulation model". Transportation Research. Part F 9. 15-27.

- Transportation Research Board. (2010). Highway Capacity Manual HCM. National Research Council. Washington. D.C. U.S.A.

- Urbanik, T. Tian, Z. & Engelbrecht, R. (2000). "Pedestrian Timing with Split Phasing Operations". Proceedings of the 70th ITE Annual Meeting. Nashville. Tenn. Aug. 6–9.

- Wang, X. Tian, Z. Ohene, F. A. & Koonce, P. (2009). "Pedestrian delay models at signalized intersections considering signal phasing and pedestrian treatment alternatives". Proceedings of the 88th Transportation Research Board Annual Meeting. Washington. D. C.

- Webster. F.V. (1958). "Traffic signal settings". Road Research Technical. Paper no. 39. Transport Research Laboratory. Berkshire.